2. Penstock

2. PENSTOCK

2.1 Stability Analysis

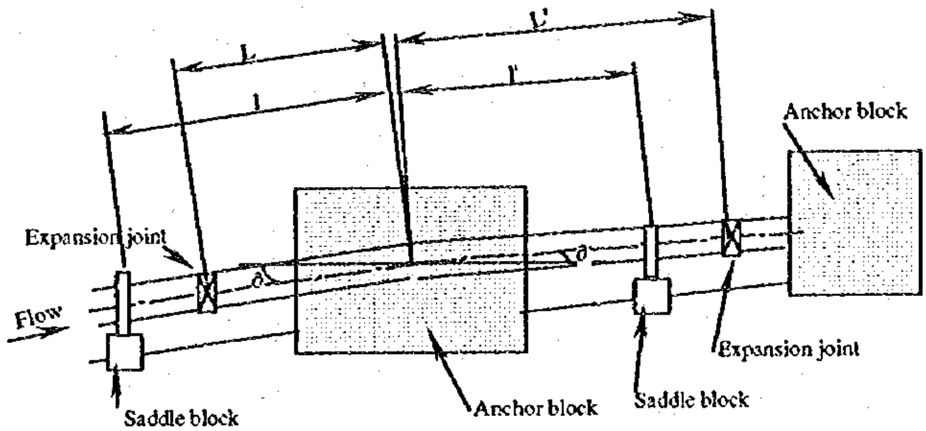
The anchor blocks No. 1-3 and 2-3 are to resist the centrifugal force caused by horizontal bend of Severino Penstock line and the resultant force of water pressure acting on the pipe. Stability calculations are therefore made for the Anchor Blocks No. 1-3 and 2-3.

Stability calculations of anchor blocks of IP-1, IP-2 and IP-4 are neglected since they are embedded in underground/ encased in mass concrete and small thrust forces.

Figure 1 shows the dimensions of Anchor Block No. 1-3 and 2-3.

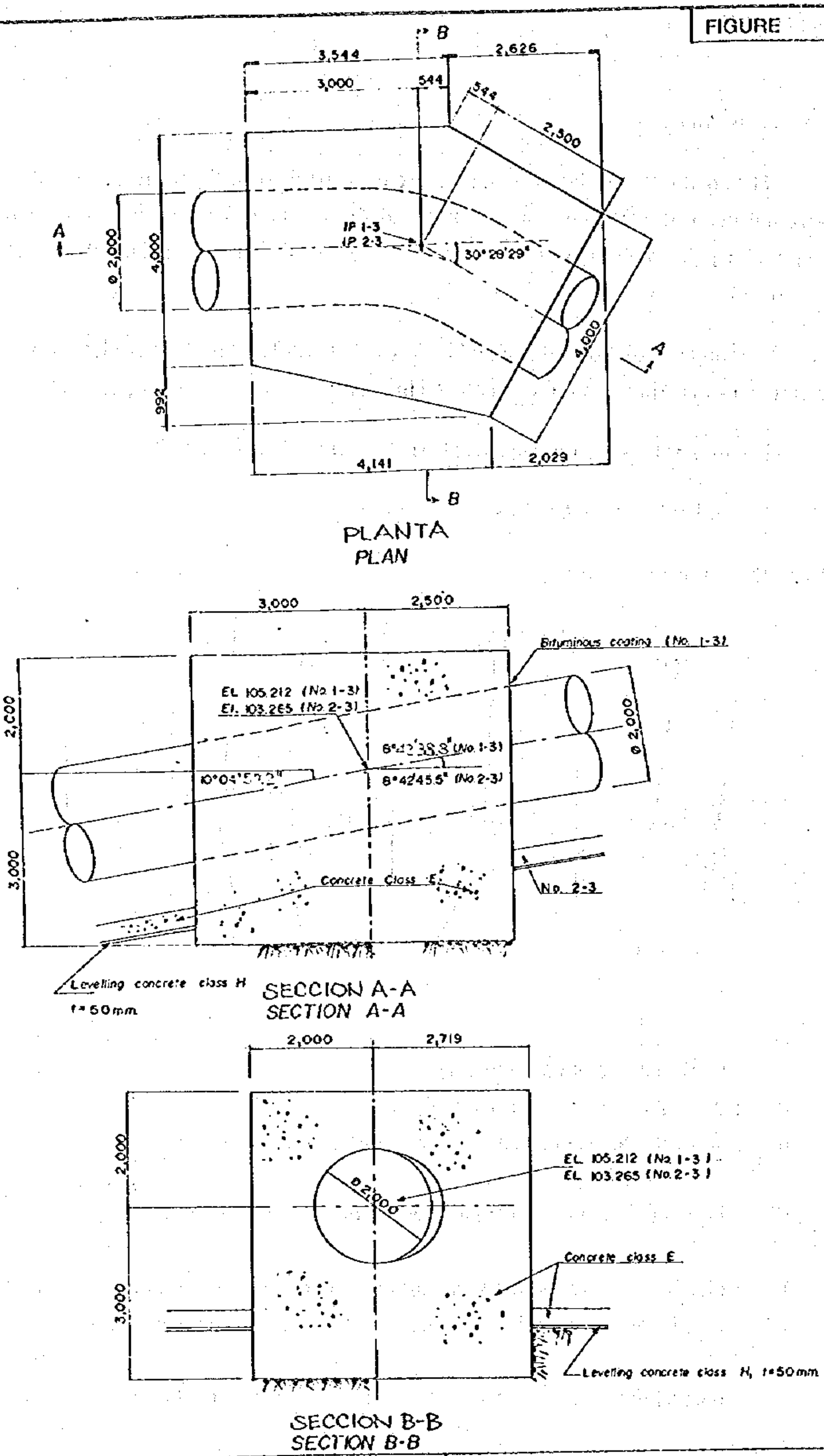
(1) Method of Stability Analysis

(A) Definition of variables



∂	: vertical angle of upstream pipe axis	(degree)
∂'	: vertical angle of downstream pipe axis	(degree)
$\varnothing$	: vertical intersection angle between upstream and downstream pipe axes $\varnothing = \partial - \partial'$	(degree)
θ	: horizontal intersection angle between upstream and downstream pipe axes	(degree)
L	: pipe length between I.P of anchor block and upstream expansion joint	(m)
L'	: pipe length between I.P of anchor block and downstream expansion joint	(m)
l	: pipe length between I.P of anchor block and upstream adjacent saddle pier	(m)

FIGURE 2.1.1



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JAPAN INTERNATIONAL COOPERATION AGENCY

TITLE

Anchor Blocks

l'	: pipe length between I.P of anchor block and downstream adjacent saddle pier	(m)
D	: inside diameter of penstock pipe	(m)
A	: inside sectional area of penstock pipe	(m^2)
t	: thickness of upstream penstock pipe	(m)
t'	: thickness of downstream penstock pipe	(m)
H	: design head at I.P of anchor block	(m)
He	: design head at upstream expansion joint	(m)
He'	: design head at downstream expansion joint	(m)
Q	: maximum pumping discharge	(m^3/sec)
s	: weight of upstream penstock pipe shell per 1 m $s = \pi \cdot D \cdot t \cdot rs$	(t/m)
s'	: weight of downstream penstock pipe shell per 1 m $s' = \pi \cdot D \cdot t' \cdot rs$	(t/m)
rs	: density of steel	=7.85 (t/m^3)
w	: weight of contained water in pipe per 1 m	(t/m)
c	: friction coefficient between pipe shell and saddle	(=0.25)
f	: friction coefficient between water and steel	(=0.02)
fe	: friction force of expansion joint per 1 m	= 0.7 (t/m)
wc	: unit weight of plain concrete	=2.3 (t/m^3)

(B) Acting Force on Anchor Block

(i) Thrust perpendicular to pipe axis due to dead weight of pipe and water in pipe

for upstream pipe $W = 1/2 \cdot (w+s) \cdot l \cdot \cos\partial$

for downstream pipe $W' = 1/2 \cdot (w+s') \cdot l' \cdot \cos\partial'$

component force

Force	x-direction	y-direction	z-direction
W	$W \cdot \sin\partial$	0	$-W \cdot \cos\partial$
W'	$W' \cdot \sin\partial' \cdot \cos\theta$	$-W' \cdot \sin\partial' \cdot \sin\theta$	$-W' \cdot \cos\partial'$

(ii) Thrust along pipe axis due to dead weight of pipe

for upstream pipe $P_1 = s \cdot L \cdot \sin\partial$

for downstream pipe $P_1' = s' \cdot L' \cdot \sin\partial'$

component force

Force	x-direction	y-direction	z-direction
P_1	$-P_1 \cdot \cos\partial$	0	$-P_1 \cdot \sin\partial$

P_1'	$-P_1' \cdot \cos\partial' \cdot \cos\theta$	$-P_1' \cdot \cos\partial' \cdot \sin\theta$	$-P_1' \cdot \sin\partial'$

refer to Figure 2.1.2

(iii) Thrust due to friction of water in pipe

for upstream pipe $P_2 = (2 \cdot f \cdot Q^2 / g \cdot \pi \cdot D^3) \cdot L$

for downstream pipe $P_2' = (2 \cdot f \cdot Q^2 / g \cdot \pi \cdot D^3) \cdot L'$

component force

Force	x-direction	y-direction	z-direction
P_2	$P_2 \cdot \cos\partial$	0	$P_2 \cdot \sin\partial$
P_2'	$P_2' \cdot \cos\partial' \cdot \cos\theta$	$-P_2' \cdot \cos\partial' \cdot \sin\theta$	$P_2' \cdot \sin\partial'$

refer to Figure 2.1.2

(iv) Centrifugal force acting on bend point

due to vertical bend $P_v = 2 \cdot v^2 / g \cdot A \cdot \sin(\phi/2)$

due to horizontal bend $P_h = 2 \cdot v^2 / g \cdot A \cdot \sin(\theta/2)$

component force

Force	x-direction	y-direction	z-direction
P_v	$-P_v \cdot \sin(\phi/2)$	0	$P_v \cdot \cos(\phi/2)$
P_h	$P_h \cdot \sin(\theta/2)$	$P_h \cdot \cos(\theta/2)$	0

refer to Figure 2.1.2

(v) Thrust due to inner pressure acting on expansion joint

for upstream pipe $P_3 = He \cdot \pi \cdot D \cdot t$

for downstream pipe $P_3' = He' \cdot \pi \cdot D \cdot t'$

component force

Force	x-direction	y-direction	z-direction
P_3	$P_3 \cdot \cos\partial$	0	$P_3 \cdot \sin\partial$
P_3'	$P_3' \cdot \cos\partial' \cdot \cos\theta$	$-P_3' \cdot \cos\partial' \cdot \sin\theta$	$P_3' \cdot \sin\partial'$

refer to Figure 2.1.2

(vi) Unbalanced force due to water pressure acting on bend point

due to vertical bend $P_{rv} = 2 \cdot H \cdot A \cdot \sin(\phi/2)$

due to horizontal bend $P_{rh} = 2 \cdot H \cdot A \cdot \sin(\theta/2)$

component force

Force	x-direction	y-direction	z-direction
P_{rv}	$-P_{rv} \cdot \sin(\alpha/2)$	0	$P_{rv}\cos(\alpha/2)$
P_{rh}	$P_{rh} \cdot \sin(\theta/2)$	$P_{rh} \cdot \cos(\theta/2)$	0

refer to Figure 2.1.2

(vii) Thrust due to temperature change

for upstream pipe $F = F_1 + F_2$
for downstream pipe $F' = F_1' + F_2'$

- Thrust due to friction of supporting point

for upstream pipe $F_1 = c \cdot (w+s) \cdot (L-l/2) \cdot \cos\partial$
for downstream pipe $F_1' = c \cdot (w+s') \cdot (L'-l'/2) \cdot \cos\partial'$

- Thrust due to friction of expansion joint

for upstream pipe $F_2 = fe \cdot \pi \cdot (D+2t)$
for downstream pipe $F_2' = fe \cdot \pi \cdot (D+2t')$
component force

Force	x-direction	y-direction	z-direction
F	$F \cdot \cos\partial$	0	$-F \cdot \sin\partial$
F'	$F' \cdot \cos\partial' \cdot \cos\theta$	$-F' \cdot \cos\partial' \cdot \sin\theta$	$F' \cdot \sin\partial'$

refer to Figure 2.1.2

(viii) Dead weight of anchor block

$W_A = wc \cdot V$ V : concrete volume of anchor block

(iv) Seismic force

$F = F_{WA} + F_p$
$F_{WA} = K_h \cdot W_A$ $F_p = K_h \cdot [(w+s) \cdot l/2 + (w+s') \cdot l'/2]$

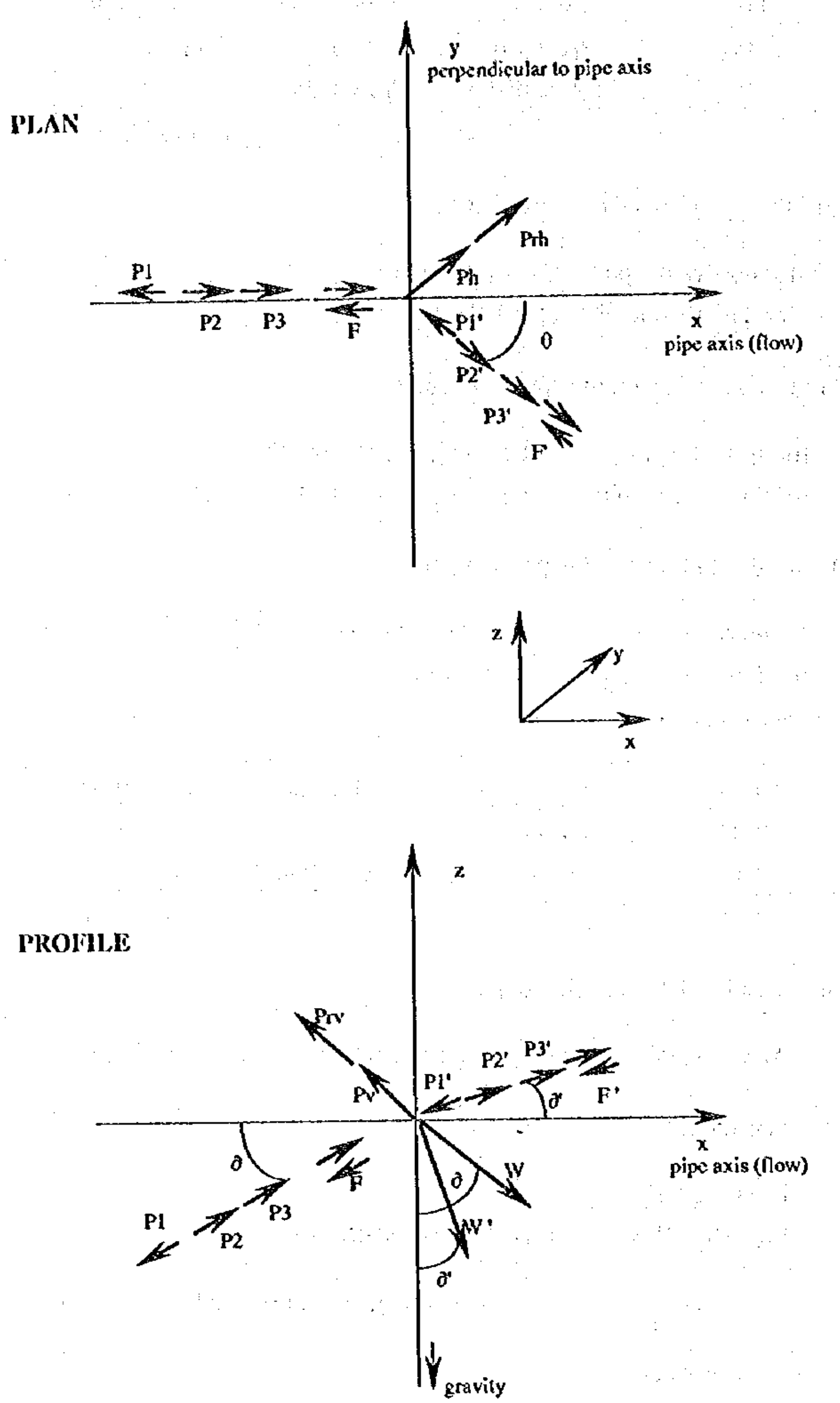
K_h : coefficient of horizontal earthquake (=0.15)

(C) Check of Safety

(i) Safety for overturning

The safety for overturning can be confirmed by the following equation.

FIGURE 2.1.2



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$$e = |B/2 - \sum M / \sum V| < B/6$$

where, e : eccentricity of resultant force on base measured from center of base

B : base length of anchor block

$\sum M$: total moment

$\sum V$: total vertical force

(ii) Safety for sliding

$$Fs = \sum V \cdot \lambda / \sum H > 2.0$$

where, Fs : safety factor

$\sum H$: total horizontal force

λ : friction coefficient of concrete and foundation = 0.65

(iii) Safety for bearing capacity

$$\sigma max = \sum V/A \cdot (1 \pm 6 \cdot e/B) < qa$$

where, σmax : maximum compressive stress

A : area of base

qa : bearing capacity of foundation

(2) Stability Analysis

(A) Cases for stability analysis

Direction of thrust forces due to temperature change (F and F') varies depending on expansion and shrinkage of penstock pipe. Other thrust forces (sum of other thrust forces indicated as "P") acts on a anchor block with a constant direction. Therefore, the following 4 cases are analyzed for stability of the anchor blocks considering the combination of the thrust forces due to temperature change and other thrust forces.

Case	Combination of Forces
1	P+F+F'
2	P+F-F'
3	P-F+F'
4	P-F-F'

The analysis was conducted on 2 plains, i.e., x-z plain (flow direction) and y-z plain (perpendicular to flow direction). Moreover, earthquake force is considered acting on the blocks in both ways for each plain, i.e., x and -x directions for x-z plain, and y and -y directions for y-z plain.

The values of component forces to x, y and z directions (Px, Py and Pz) are calculated and shown in Table 1 with input data.

(B) Dead weight of anchor block, seismic force and center position of gravity of anchor block

(a) W_A : Dead weight of anchor block

$$W_A = 2.3 \times \{(4.992 \times 6.17 - 1/2 \times 2.626 \times 1.545 - 1/2 \times 3.447 \times 2.029 - 1/2 \times 0.992 \times 4.141) \times 5.0 - \pi/4 \times 2.0^2 \times 5.5\}$$
$$= 227.302 \text{ ton}$$

(b) F_{WA} : Seismic force for dead weight of anchor block

$$F_{WA} = 0.15 \times 227.302 = 34.095 \text{ ton}$$

(c) F_P : Seismic force for dead weight of penstock pipe and water in pipe

$$F_P = 0.15 \times (3.142 + 0.395) \times (10 + 12.062) / 2 = 5.852 \text{ ton}$$

(d) Distance to gravity center of anchor block

x-z plain

$$x = \sum A \cdot x / \sum A$$

$$\sum A \cdot x = 4.992 \times 6.17 \times 6.17 / 2 - 1/2 \times 0.992 \times 4.141 \times 1/3 \times 4.141 - 1/2 \times 2.626 \times 1.545 \times (3.544 + 2/3 \times 2.626) - 1/2 \times 2.029 \times 3.447 \times (4.141 + 2/3 \times 2.029)$$
$$= 62.233 \text{ m}^3$$

$$\sum A = 4.992 \times 6.17 \times -1/2 \times 1.545 \times 2.626 - 1/2 \times 2.029 \times 3.447 - 1/2 \times 0.992 \times 4.141$$
$$= 23.221 \text{ m}^2$$

therefore, $x = 62.233 / 23.221 = 2.680$ m

y-z plain

$$y = \sum A \cdot y / \sum A$$

2.8

Table 2.1.1 Thrust Forces

Anchor Block No.1-3

Input Data

∂=		10.0821067 (deg.)
∂'=		10.0821067 (deg.)
φ=	∂-∂'=	0 (deg.)
θ=		30.4747222 (deg.)
L=		5.188 (m)
L'=		0.000 (m)
l=		7.188 (m)
l'=		0.000 (m)
D=		2.000 (m)
A=		3.142 (m2)
t=		0.009 (m)
t'=		0.009 (m)
H=		15.500 (m)
He=		17.500 (m)
He'=		0.000 (m)
Q=		9.6 (m3/sec)
s=	π•D•t•rs=	0.444 (t/m)
s'=	π•D•t'•rs=	0.444 (t/m)
rs=		7.85 (t/m3)
w=	π•D^2/4=	3.142 (t/m)
c=		0.25
f=		0.02
fe=		0.7
wc=		2.3 (t/m3)

		Force	x-direction	component y-direction	z-direction
	W	12.687	2.221	0.000	-12.491
	W'	0.000	0.000	0.000	0.000
	P1	0.403	-0.397	0.000	-0.071
	P1'	0.000	0.000	0.000	0.000
	P2	0.078	0.076	0.000	0.014
	P2'	0.000	0.000	0.000	0.000
	Pv	0.000	0.000	0.000	0.000
	Ph	1.573	0.414	1.518	0.000
	P3	0.990	0.974	0.000	0.173
	P3'	0.000	0.000	0.000	0.000
	Prv	0.000	0.000	0.000	0.000
	Prh	25.596	6.727	24.696	0.000
Total of above					
	P	41.327	10.015	26.214	-12.375
	F1	1.407			
	F1'	0.000			
	F2	4.438			
	F2'	4.438			
	F=F1+F2	5.845	5.754	0.000	-1.023
	F'=F1'+F2'	4.438	3.766	-2.216	0.777
			Px=	Py=	Pz=
Case	Total				
1	P+F+F'	51.609	19.535	23.998	-12.621
2	P+F-F'	42.734	12.004	28.430	-14.175
3	P-F+F'	39.920	8.027	23.998	-10.575
4	P-F-F'	31.044	0.495	28.430	-12.129

Anchor Block No.2-3

Input Data

∂=		10.0821067 (deg.)
∂'=		10.0821067 (deg.)
φ=	∂-∂'=	0 (deg.)
θ=		30.4747222 (deg.)
L=		5.070 (m)
L'=		5.564 (m)
l=		8.070 (m)
l'=		10.022 (m)
D=		2.000 (m)
A=		3.142 (m2)
t=		0.009 (m)
t'=		0.009 (m)
H=		18.750 (m)
He=		20.500 (m)
He'=		17.500 (m)
Q=		9.6 (m3/sec)
s=	π•D•t•rs=	0.444 (t/m)
s'=	π•D•t'•rs=	0.444 (t/m)
rs=		7.85 (t/m3)
w=	π•D^2/4=	3.142 (t/m)
c=		0.25
f=		0.02
fe=		0.7
wc=		2.3 (t/m3)

		Force	x-direction	component y-direction	z-direction
	W	14.244	2.494	0.000	-14.024
	W'	17.689	2.669	-1.571	-17.416
	P1	0.394	-0.388	0.000	-0.069
	P1'	0.432	-0.367	-0.216	-0.076
	P2	0.076	0.075	0.000	0.013
	P2'	0.083	0.071	0.042	0.015
	Pv	0.000	0.000	0.000	0.000
	Ph	1.573	0.414	1.518	0.000
	P3	1.159	1.141	0.000	0.203
	P3'	0.990	0.840	-0.494	0.173
	Prv	0.000	0.000	0.000	0.000
	Prh	30.963	8.138	29.874	0.000
Total of above					
	P	67.604	15.085	29.153	-31.181
	F1	0.913			
	F1'	0.488			
	F2	4.438			
	F2'	4.438			
	F=F1+F2	5.351	5.269	0.000	-0.937
	F'=F1'+F2'	4.926	4.180	-2.460	0.862
			Px=	Py=	Pz=
Case	Total				
1	P+F+F'	77.881	24.534	26.694	-31.256
2	P+F-F'	68.029	16.174	31.613	-32.980
3	P-F+F'	67.179	13.996	26.694	-29.382
4	P-F-F'	57.327	5.637	31.613	-31.107

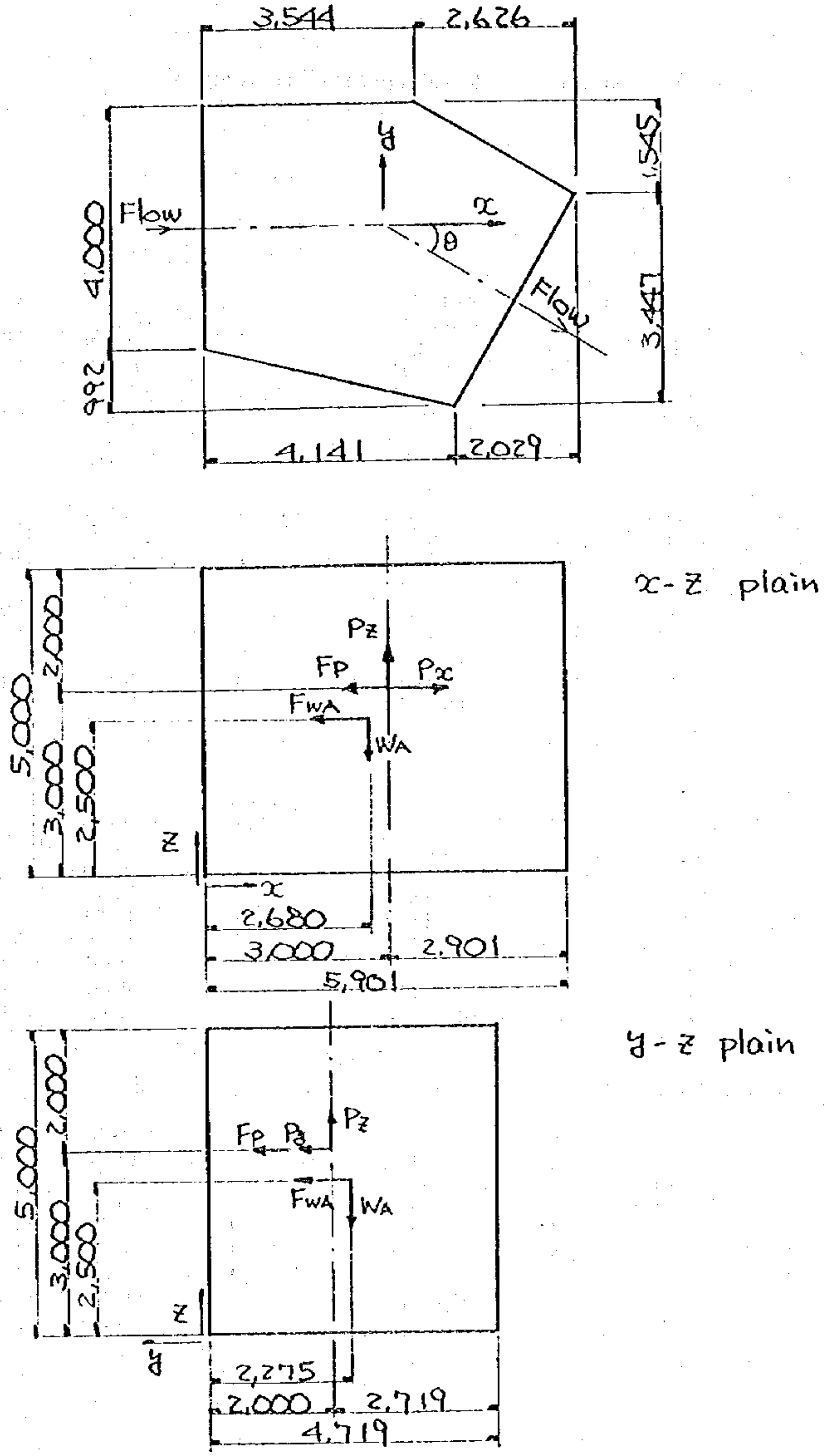
$$\begin{aligned}\Sigma A \cdot y &= 4.992 \times 6.17 \times 2.496 - 1/2 \times 1.545 \times 2.626 \times 1/3 \times 1.545 \\ &\quad - 1/2 \times 2.029 \times 3.447 \times (1.545 + 2/3 \times 3.447) \\ &\quad - 1/2 \times 0.992 \times 4.141 \times (4.0 + 2/3 \times 0.992) \\ &= 52.821 \text{ m}^3\end{aligned}$$

therefore, $y = 52.821 / 23.221 = 2.275$ m

External forces acting on the anchor blocks are illustrated in Figure 2.1.3.

(C) Results of analysis

The summary of the calculation results are shown in Table 2.1.1 and calculation sheets are attached.



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JAPAN INTERNATIONAL COOPERATION AGENCY

TITLE

External Forces

Table 2.1.2 Summary of Results of Stability Analysis

Anchor Block No. 1-3

1. x-z plain

1.1 earthquake -x-direction

Case	safety for overturning e (m)	safety for sliding Fs	safety for bearing capacity ÓÓmax (t/m2)
1	0.389	9.46	14.42
2	0.480	6.53	15.47
3	0.538	5.52	15.85
4	0.628	4.38	16.90
	<B/6= 0.984	> 2.0	<qa =100 t/m2

1.2 earthquake x-direction

Case	safety for overturning e (m)	safety for sliding Fs	safety for bearing capacity ÓÓmax (t/m2)
1	0.370	2.81	14.22
2	0.274	3.27	13.30
3	0.228	3.51	12.62
4	0.132	4.26	11.70
	<B/6= 0.984	> 2.0	<qa =100 t/m2

2. y-z plain

2.1 earthquake -y-direction

Case	safety for overturning e (m)	safety for sliding Fs	safety for bearing capacity ÓÓmax (t/m2)
1	0.178	12.96	12.67
2	0.124	20.66	12.04
3	0.177	12.85	12.55
4	0.122	20.48	11.92
	<B/6= 0.787	> 2.0	<qa =100 t/m2

2.2 earthquake y-direction

Case	safety for overturning e (m)	safety for sliding Fs	safety for bearing capacity ÓÓmax (t/m2)
1	0.581	2.60	17.96
2	0.630	2.44	18.72
3	0.589	2.58	17.91
4	0.638	2.41	18.68
	<B/6= 0.787	> 2.0	<qa =100 t/m2

Anchor Block No. 2-3

1. x-z plain

1.1 earthquake -x-direction

Case	safety for overturning e (m)	safety for sliding Fs	safety for bearing capacity ÓÓmax (t/m2)
1	0.333	11.65	14.91
2	0.427	7.42	16.08
3	0.459	6.68	16.22
4	0.553	5.04	17.38
	<B/6= 0.984	> 2.0	<qa =100 t/m2

1.2 earthquake x-direction

Case	safety for overturning e (m)	safety for sliding Fs	safety for bearing capacity ÓÓmax (t/m2)
1	0.439	2.65	16.10
2	0.340	3.07	15.08
3	0.319	3.15	14.64
4	0.220	3.77	13.62
	<B/6= 0.984	> 2.0	<qa =100 t/m2

2. y-z plain

2.1 earthquake -y-direction

Case	safety for overturning e (m)	safety for sliding Fs	safety for bearing capacity ÓÓmax (t/m2)
1	0.194	13.70	13.88
2	0.139	23.03	13.18
3	0.193	13.60	13.77
4	0.137	22.86	13.07
	<B/6= 0.787	> 2.0	<qa =100 t/m2

2.2 earthquake y-direction

Case	safety for overturning e (m)	safety for sliding Fs	safety for bearing capacity ÓÓmax (t/m2)
1	0.578	2.56	19.32
2	0.629	2.40	20.17
3	0.585	2.54	19.28
4	0.636	2.38	20.12
	<B/6= 0.787	> 2.0	<qa =100 t/m2

Anchor Block No.1-3 Calculation Sheet

Stability Analysis

x-z plain B= 5.901 m

Case-1 earthquake : -x-direction

Vertical	Force (ton)	x (m)	Moment (ton•m)
WA	-227.302	2.680	-609.169
Pz	-12.621	3.000	-37.864
ΣV=	-239.923	ΣV•x=	-647.034

Horizontal	Force (ton)	z (m)	Moment (ton•m)
FwA	-34.095	2.500	-85.238
Fp	-1.933	3.000	-5.799
Px	19.535	3.000	58.606
ΣH=	-16.493	ΣH•z=	-32.431

ΣM= ΣV•x-ΣH•z= -614.603

Safety for overturning

e= 0.389 < 0.984

Safety for sliding

Fs = 9.46 > 2.0

Safety for bearing capacity

ÓÓmax= 14.42 < 100 t/m2

Stability Analysis

x-z plain B= 5.901 m

Case-2 earthquake : -x-direction

Vertical	Force (ton)	x (m)	Moment (ton•m)
WA	-227.302	2.680	-609.169
Pz	-14.175	3.000	-42.525
ΣV=	-241.477	ΣV•x=	-651.695

Horizontal	Force (ton)	z (m)	Moment (ton•m)
FwA	-34.095	2.500	-85.238
Fp	-1.933	3.000	-5.799
Px	12.004	3.000	36.012
ΣH=	-24.024	ΣH•z=	-55.025

ΣM= ΣV•x-ΣH•z= -596.670

Safety for overturning

e= 0.480 < 0.984

Safety for sliding

Fs = 6.53 > 2.0

Safety for bearing capacity

ÓÓmax= 15.47 < 100 t/m2

Stability Analysis

x-z plain B= 5.901 m

Case-3 earthquake : -x-direction

Vertical	Force (ton)	x (m)	Moment (ton•m)
WA	-227.302	2.680	-609.169
Pz	-10.575	3.000	-31.725
ΣV=	-237.877	ΣV•x=	-640.895

Horizontal	Force (ton)	z (m)	Moment (ton•m)
FwA	-34.095	2.500	-85.238
Fp	-1.933	3.000	-5.799
Px	8.027	3.000	24.080
ΣH=	-28.001	ΣH•z=	-66.957

ΣM= ΣV•x-ΣH•z= -573.938

Safety for overturning

e= 0.538 < 0.984

Safety for sliding

Fs = 5.52 > 2.0

Safety for bearing capacity

ÓÓmax= 15.85 < 100 t/m2

Stability Analysis

x-z plain B= 5.901 m

Case-4 earthquake : -x-direction

Vertical	Force (ton)	x (m)	Moment (ton•m)
WA	-227.302	2.680	-609.169
Pz	-12.129	3.000	-36.387
ΣV=	-239.431	ΣV•x=	-645.556

Horizontal	Force (ton)	z (m)	Moment (ton•m)
FwA	-34.095	2.500	-85.238
Fp	-1.933	3.000	-5.799
Px	0.495	3.000	1.486
ΣH=	-35.533	ΣH•z=	-89.551

ΣM= ΣV•x-ΣH•z= -556.005

Safety for overturning

e= 0.628 < 0.984

Safety for sliding

Fs = 4.38 > 2.0

Safety for bearing capacity

ÓÓmax= 16.90 < 100 t/m2

Stability Analysis

x-z plain B= 5.901 m

Case-1 earthquake : x-direction

Vertical	Force (ton)	x (m)	Moment (ton·m)
WA	-227.302	2.680	-609.169
Pz	-12.621	3.000	-37.864
ΣV=	-239.923	ΣV·x=	-647.034

Horizontal	Force (ton)	z (m)	Moment (ton·m)
FwA	34.095	2.500	85.238
Fp	1.933	3.000	5.799
Px	19.535	3.000	58.606
ΣH=	55.564	ΣH·z=	149.643

ΣM= ΣV·x-ΣH·z= -796.677

Safety for overturning
e= 0.370 < 0.984

Safety for sliding
Fs = 2.81 > 2.0

Safety for bearing capacity
σmax= 14.22 < 100 t/m2

Stability Analysis

x-z plain B= 5.901 m

Case-2 earthquake : x-direction

Vertical	Force (ton)	x (m)	Moment (ton·m)
WA	-227.302	2.680	-609.169
Pz	-14.175	3.000	-42.525
ΣV=	-241.477	ΣV·x=	-651.695

Horizontal	Force (ton)	z (m)	Moment (ton·m)
FwA	34.095	2.500	85.238
Fp	1.933	3.000	5.799
Px	12.004	3.000	36.012
ΣH=	48.032	ΣH·z=	127.049

ΣM= ΣV·x-ΣH·z= -778.744

Safety for overturning
e= 0.274 < 0.984

Safety for sliding
Fs = 3.27 > 2.0

Safety for bearing capacity
σmax= 13.30 < 100 t/m2

Stability Analysis

x-z plain B= 5.901 m

Case-3 earthquake : x-direction

Vertical	Force (ton)	x (m)	Moment (ton·m)
WA	-227.302	2.680	-609.169
Pz	-10.575	3.000	-31.725
ΣV=	-237.877	ΣV·x=	-640.895

Horizontal	Force (ton)	z (m)	Moment (ton·m)
FwA	34.095	2.500	85.238
Fp	1.933	3.000	5.799
Px	8.027	3.000	24.080
ΣH=	44.055	ΣH·z=	115.117

ΣM= ΣV·x-ΣH·z= -756.012

Safety for overturning
e= 0.228 < 0.984

Safety for sliding
Fs = 3.51 > 2.0

Safety for bearing capacity
σmax= 12.62 < 100 t/m2

Stability Analysis

x-z plain B= 5.901 m

Case-4 earthquake : x-direction

Vertical	Force (ton)	x (m)	Moment (ton·m)
WA	-227.302	2.680	-609.169
Pz	-12.129	3.000	-36.387
ΣV=	-239.431	ΣV·x=	-645.556

Horizontal	Force (ton)	z (m)	Moment (ton·m)
FwA	34.095	2.500	85.238
Fp	1.933	3.000	5.799
Px	0.495	3.000	1.486
ΣH=	36.524	ΣH·z=	92.523

ΣM= ΣV·x-ΣH·z= -738.079

Safety for overturning
e= 0.132 < 0.984

Safety for sliding
Fs = 4.26 > 2.0

Safety for bearing capacity
σmax= 11.70 < 100 t/m2

Stability Analysis
y-z plain B= 4.719 m
Case-1 earthquake : -y-direction

Vertical	Force (ton)	y (m)	Moment (ton•m)
WA	-227.302	2.275	-517.112
Pz	-12.621	2.000	-25.243
ΣV=	-239.923	ΣV•x=	-542.355

Horizontal	Force (ton)	z (m)	Moment (ton•m)
FwA	-34.095	2.500	-85.238
Fp	-1.933	3.000	-5.799
Py	23.998	3.000	71.994
ΣH=	-12.030	ΣH•z=	-19.043

ΣM= ΣV•x-ΣH•z= -523.312

Safety for overturning
e= 0.178 < 0.787

Safety for sliding
Fs = 12.96 > 2.0

Safety for bearing capacity
σmax= 12.67 < 100 t/m2

Stability Analysis
y-z plain B= 4.719 m
Case-2 earthquake : -y-direction

Vertical	Force (ton)	y (m)	Moment (ton•m)
WA	-227.302	2.275	-517.112
Pz	-14.175	2.000	-28.350
ΣV=	-241.477	ΣV•x=	-545.462

Horizontal	Force (ton)	z (m)	Moment (ton•m)
FwA	-34.095	2.500	-85.238
Fp	-1.933	3.000	-5.799
Py	28.430	3.000	85.290
ΣH=	-7.598	ΣH•z=	-5.747

ΣM= ΣV•x-ΣH•z= -539.715

Safety for overturning
e= 0.124 < 0.787

Safety for sliding
Fs = 20.66 > 2.0

Safety for bearing capacity
σmax= 12.04 < 100 t/m2

Stability Analysis
y-z plain B= 4.719 m
Case-3 earthquake : -y-direction

Vertical	Force (ton)	y (m)	Moment (ton•m)
WA	-227.302	2.275	-517.112
Pz	-10.575	2.000	-21.150
ΣV=	-237.877	ΣV•x=	-538.262

Horizontal	Force (ton)	z (m)	Moment (ton•m)
FwA	-34.095	2.500	-85.238
Fp	-1.933	3.000	-5.799
Py	23.998	3.000	71.994
ΣH=	-12.030	ΣH•z=	-19.043

ΣM= ΣV•x-ΣH•z= -519.220

Safety for overturning
e= 0.177 < 0.787

Safety for sliding
Fs = 12.85 > 2.0

Safety for bearing capacity

Stability Analysis
y-z plain B= 4.719 m
Case-4 earthquake : -y-direction

Vertical	Force (ton)	y (m)	Moment (ton•m)
WA	-227.302	2.275	-517.112
Pz	-12.129	2.000	-24.258
ΣV=	-239.431	ΣV•x=	-541.370

Horizontal	Force (ton)	z (m)	Moment (ton•m)
FwA	-34.095	2.500	-85.238
Fp	-1.933	3.000	-5.799
Py	28.430	3.000	85.290
ΣH=	-7.598	ΣH•z=	-5.747

ΣM= ΣV•x-ΣH•z= -535.623

Safety for overturning
e= 0.122 < 0.787

Safety for sliding
Fs = 20.48 > 2.0

Safety for bearing capacity

2-15

Stability Analysis

y-z plain B= 4.719 m

Case-1 earthquake : y-direction

Vertical	Force (ton)	y (m)	Moment (ton·m)
WA	-227.302	2.275	-517.112
Pz	-12.621	2.000	-25.243
ΣV=	-239.923	ΣV·x=	-542.355

Horizontal	Force (ton)	z (m)	Moment (ton·m)
FwA	34.095	2.500	85.238
Fp	1.933	3.000	5.799
Py	23.998	3.000	71.994
ΣH=	60.026	ΣH·z=	163.031

ΣM= ΣV·x-ΣH·z= -705.386

Safety for overturning
e= 0.581 < 0.787

Safety for sliding
Fs = 2.60 > 2.0

Safety for bearing capacity
σmax= 17.96 < 100 t/m2

Stability Analysis

y-z plain B= 4.719 m

Case-2 earthquake : y-direction

Vertical	Force (ton)	y (m)	Moment (ton·m)
WA	-227.302	2.275	-517.112
Pz	-14.175	2.000	-28.350
ΣV=	-241.477	ΣV·x=	-545.462

Horizontal	Force (ton)	z (m)	Moment (ton·m)
FwA	34.095	2.500	85.238
Fp	1.933	3.000	5.799
Py	28.430	3.000	85.290
ΣH=	64.458	ΣH·z=	176.327

ΣM= ΣV·x-ΣH·z= -721.789

Safety for overturning
e= 0.630 < 0.787

Safety for sliding
Fs = 2.44 > 2.0

Safety for bearing capacity
σmax= 18.72 < 100 t/m2

Stability Analysis

y-z plain B= 4.719 m

Case-3 earthquake : y-direction

Vertical	Force (ton)	y (m)	Moment (ton·m)
WA	-227.302	2.275	-517.112
Pz	-10.575	2.000	-21.150
ΣV=	-237.877	ΣV·x=	-538.262

Horizontal	Force (ton)	z (m)	Moment (ton·m)
FwA	34.095	2.500	85.238
Fp	1.933	3.000	5.799
Py	23.998	3.000	71.994
ΣH=	60.026	ΣH·z=	163.031

ΣM= ΣV·x-ΣH·z= -701.294

Safety for overturning
e= 0.589 < 0.787

Safety for sliding
Fs = 2.58 > 2.0

Safety for bearing capacity
σmax= 17.91 < 100 t/m2

Stability Analysis

y-z plain B= 4.719 m

Case-4 earthquake : y-direction

Vertical	Force (ton)	y (m)	Moment (ton·m)
WA	-227.302	2.275	-517.112
Pz	-12.129	2.000	-24.258
ΣV=	-239.431	ΣV·x=	-541.370

Horizontal	Force (ton)	z (m)	Moment (ton·m)
FwA	34.095	2.500	85.238
Fp	1.933	3.000	5.799
Py	28.430	3.000	85.290
ΣH=	64.458	ΣH·z=	176.327

ΣM= ΣV·x-ΣH·z= -717.697

Safety for overturning
e= 0.638 < 0.787

Safety for sliding
Fs = 2.41 > 2.0

Safety for bearing capacity
σmax= 18.68 < 100 t/m2

Anchor Block No.2-3 Calculation Sheet

Stability Analysis

x-z plain   B= 5.901 m

Case-1   earthquake : -x-direction

Vertical	Force (ton)	x (m)	Moment (ton•m)
WA	-227.302	2.680	-609.169
Pz	-31.256	3.000	-93.767
ΣV=	-258.558	ΣV•x=	-702.936

Horizontal	Force (ton)	z (m)	Moment (ton•m)
FwA	-34.095	2.500	-85.238
Fp	-4.865	3.000	-14.595
Px	24.534	3.000	73.601
ΣH=	-14.427	ΣH•z=	-26.233

ΣM= ΣV•x-ΣH•z= -676.703

Safety for overturning
e= 0.333 < 0.984

Safety for sliding
Fs = 11.65 > 2.0

Safety for bearing capacity
ÓÓmax= 14.91 < 100 t/m2

Stability Analysis

x-z plain   B= 5.901 m

Case-2   earthquake : -x-direction

Vertical	Force (ton)	x (m)	Moment (ton•m)
WA	-227.302	2.680	-609.169
Pz	-32.980	3.000	-98.941
ΣV=	-260.282	ΣV•x=	-708.110

Horizontal	Force (ton)	z (m)	Moment (ton•m)
FwA	-34.095	2.500	-85.238
Fp	-4.865	3.000	-14.595
Px	16.174	3.000	48.522
ΣH=	-22.786	ΣH•z=	-51.312

ΣM= ΣV•x-ΣH•z= -656.798

Safety for overturning
e= 0.427 < 0.984

Safety for sliding
Fs = 7.42 > 2.0

Safety for bearing capacity
ÓÓmax= 16.08 < 100 t/m2

Stability Analysis

x-z plain   B= 5.901 m

Case-3   earthquake : -x-direction

Vertical	Force (ton)	x (m)	Moment (ton•m)
WA	-227.302	2.680	-609.169
Pz	-29.382	3.000	-88.146
ΣV=	-256.684	ΣV•x=	-697.315

Horizontal	Force (ton)	z (m)	Moment (ton•m)
FwA	-34.095	2.500	-85.238
Fp	-4.865	3.000	-14.595
Px	13.996	3.000	41.989
ΣH=	-24.964	ΣH•z=	-57.845

ΣM= ΣV•x-ΣH•z= -639.471

Safety for overturning
e= 0.459 < 0.984

Safety for sliding
Fs = 6.68 > 2.0

Safety for bearing capacity
ÓÓmax= 16.22 < 100 t/m2

Stability Analysis

x-z plain   B= 5.901 m

Case-4   earthquake : -x-direction

Vertical	Force (ton)	x (m)	Moment (ton•m)
WA	-227.302	2.680	-609.169
Pz	-31.107	3.000	-93.320
ΣV=	-258.409	ΣV•x=	-702.489

Horizontal	Force (ton)	z (m)	Moment (ton•m)
FwA	-34.095	2.500	-85.238
Fp	-4.865	3.000	-14.595
Px	5.637	3.000	16.910
ΣH=	-33.324	ΣH•z=	-82.923

ΣM= ΣV•x-ΣH•z= -619.566

Safety for overturning
e= 0.553 < 0.984

Safety for sliding
Fs = 5.04 > 2.0

Safety for bearing capacity
ÓÓmax= 17.38 < 100 t/m2

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Stability Analysis

x-z plain B= 5.901 m
Case-1 earthquake : x-direction

Vertical	Force (ton)	x (m)	Moment (ton·m)
WA	-227.302	2.680	-609.169
Pz	-31.256	3.000	-93.767
ΣV=	-258.558	ΣV·x=	-702.936

Horizontal	Force (ton)	z (m)	Moment (ton·m)
FwA	34.095	2.500	85.238
Fp	4.865	3.000	14.595
Px	24.534	3.000	73.601
ΣH=	63.494	ΣH·z=	173.434

ΣM= ΣV·x-ΣH·z= -876.370

Safety for overturning
e= 0.439 < 0.984

Safety for sliding
Fs = 2.65 > 2.0

Safety for bearing capacity
σ_{max}= 16.10 < 100 t/m2

Stability Analysis

x-z plain B= 5.901 m
Case-2 earthquake : x-direction

Vertical	Force (ton)	x (m)	Moment (ton·m)
WA	-227.302	2.680	-609.169
Pz	-32.980	3.000	-98.941
ΣV=	-260.282	ΣV·x=	-708.110

Horizontal	Force (ton)	z (m)	Moment (ton·m)
FwA	34.095	2.500	85.238
Fp	4.865	3.000	14.595
Px	16.174	3.000	48.522
ΣH=	55.134	ΣH·z=	148.356

ΣM= ΣV·x-ΣH·z= -856.466

Safety for overturning
e= 0.340 < 0.984

Safety for sliding
Fs = 3.07 > 2.0

Safety for bearing capacity
σ_{max}= 15.08 < 100 t/m2

Stability Analysis

x-z plain B= 5.901 m
Case-3 earthquake : x-direction

Vertical	Force (ton)	x (m)	Moment (ton·m)
WA	-227.302	2.680	-609.169
Pz	-29.382	3.000	-88.146
ΣV=	-256.684	ΣV·x=	-697.315

Horizontal	Force (ton)	z (m)	Moment (ton·m)
FwA	34.095	2.500	85.238
Fp	4.865	3.000	14.595
Px	13.996	3.000	41.989
ΣH=	52.957	ΣH·z=	141.823

ΣM= ΣV·x-ΣH·z= -839.138

Safety for overturning
e= 0.319 < 0.984

Safety for sliding
Fs = 3.15 > 2.0

Safety for bearing capacity
σ_{max}= 14.64 < 100 t/m2

Stability Analysis

x-z plain B= 5.901 m
Case-4 earthquake : x-direction

Vertical	Force (ton)	x (m)	Moment (ton·m)
WA	-227.302	2.680	-609.169
Pz	-31.107	3.000	-93.320
ΣV=	-258.409	ΣV·x=	-702.489

Horizontal	Force (ton)	z (m)	Moment (ton·m)
FwA	34.095	2.500	85.238
Fp	4.865	3.000	14.595
Px	5.637	3.000	16.910
ΣH=	44.597	ΣH·z=	116.744

ΣM= ΣV·x-ΣH·z= -819.233

Safety for overturning
e= 0.220 < 0.984

Safety for sliding
Fs = 3.77 > 2.0

Safety for bearing capacity
σ_{max}= 13.62 < 100 t/m2

Stability Analysis
y-z plain B= 4.719 m
Case-1 earthquake : -y-direction

Vertical	Force (ton)	y (m)	Moment (ton•m)
WA	-227.302	2.275	-517.112
Pz	-31.256	2.000	-62.511
ΣV=	-258.558	ΣV•x=	-579.623

Horizontal	Force (ton)	z (m)	Moment (ton•m)
FwA	-34.095	2.500	-85.238
Fp	-4.865	3.000	-14.595
Py	26.694	3.000	80.081
ΣH=	-12.267	ΣH•z=	-19.753

ΣM= ΣV•x-ΣH•z= -559.870

Safety for overturning
e= 0.194 < 0.787

Safety for sliding
Fs = 13.70 > 2.0

Safety for bearing capacity
óómax= 13.88 < 100 t/m2

Stability Analysis
y-z plain B= 4.719 m
Case-2 earthquake : -y-direction

Vertical	Force (ton)	y (m)	Moment (ton•m)
WA	-227.302	2.275	-517.112
Pz	-32.980	2.000	-65.960
ΣV=	-260.282	ΣV•x=	-583.072

Horizontal	Force (ton)	z (m)	Moment (ton•m)
FwA	-34.095	2.500	-85.238
Fp	-4.865	3.000	-14.595
Py	31.613	3.000	94.839
ΣH=	-7.348	ΣH•z=	-4.995

ΣM= ΣV•x-ΣH•z= -578.077

Safety for overturning
e= 0.139 < 0.787

Safety for sliding
Fs = 23.03 > 2.0

Safety for bearing capacity
óómax= 13.18 < 100 t/m2

Stability Analysis
y-z plain B= 4.719 m
Case-3 earthquake : -y-direction

Vertical	Force (ton)	y (m)	Moment (ton•m)
WA	-227.302	2.275	-517.112
Pz	-29.382	2.000	-58.764
ΣV=	-256.684	ΣV•x=	-575.876

Horizontal	Force (ton)	z (m)	Moment (ton•m)
FwA	-34.095	2.500	-85.238
Fp	-4.865	3.000	-14.595
Py	26.694	3.000	80.081
ΣH=	-12.267	ΣH•z=	-19.753

ΣM= ΣV•x-ΣH•z= -556.123

Safety for overturning
e= 0.193 < 0.787

Safety for sliding
Fs = 13.60 > 2.0

Safety for bearing capacity

Stability Analysis
y-z plain B= 4.719 m
Case-4 earthquake : -y-direction

Vertical	Force (ton)	y (m)	Moment (ton•m)
WA	-227.302	2.275	-517.112
Pz	-31.107	2.000	-62.213
ΣV=	-258.409	ΣV•x=	-579.325

Horizontal	Force (ton)	z (m)	Moment (ton•m)
FwA	-34.095	2.500	-85.238
Fp	-4.865	3.000	-14.595
Py	31.613	3.000	94.839
ΣH=	-7.348	ΣH•z=	-4.995

ΣM= ΣV•x-ΣH•z= -574.330

Safety for overturning
e= 0.137 < 0.787

Safety for sliding
Fs = 22.86 > 2.0

Safety for bearing capacity

2-19

2-20

Stability Analysis

y-z plain B= 4.719 m

Case-1 earthquake : y-direction

Vertical	Force (ton)	y (m)	Moment (ton•m)
WA	-227.302	2.275	-517.112
Pz	-31.256	2.000	-62.511
ΣV=	-258.558	ΣV•x=	-579.623

Horizontal	Force (ton)	z (m)	Moment (ton•m)
FwA	34.095	2.500	85.238
Fp	4.865	3.000	14.595
Py	26.694	3.000	80.081
ΣH=	65.654	ΣH•z=	179.915

ΣM= ΣV•x-ΣH•z= -759.538

Safety for overturning

e= 0.578 < 0.787

Safety for sliding

Fs = 2.56 > 2.0

Safety for bearing capacity

σσmax= 19.32 < 100 t/m2

Stability Analysis

y-z plain B= 4.719 m

Case-2 earthquake : y-direction

Vertical	Force (ton)	y (m)	Moment (ton•m)
WA	-227.302	2.275	-517.112
Pz	-32.980	2.000	-65.960
ΣV=	-260.282	ΣV•x=	-583.072

Horizontal	Force (ton)	z (m)	Moment (ton•m)
FwA	34.095	2.500	85.238
Fp	4.865	3.000	14.595
Py	31.613	3.000	94.839
ΣH=	70.573	ΣH•z=	194.672

ΣM= ΣV•x-ΣH•z= -777.745

Safety for overturning

e= 0.629 < 0.787

Safety for sliding

Fs = 2.40 > 2.0

Safety for bearing capacity

σσmax= 20.17 < 100 t/m2

Stability Analysis

y-z plain B= 4.719 m

Case-3 earthquake : y-direction

Vertical	Force (ton)	y (m)	Moment (ton•m)
WA	-227.302	2.275	-517.112
Pz	-29.382	2.000	-58.764
ΣV=	-256.684	ΣV•x=	-575.876

Horizontal	Force (ton)	z (m)	Moment (ton•m)
FwA	34.095	2.500	85.238
Fp	4.865	3.000	14.595
Py	26.694	3.000	80.081
ΣH=	65.654	ΣH•z=	179.915

ΣM= ΣV•x-ΣH•z= -755.791

Safety for overturning

e= 0.585 < 0.787

Safety for sliding

Fs = 2.54 > 2.0

Safety for bearing capacity

σσmax= 19.28 < 100 t/m2

Stability Analysis

y-z plain B= 4.719 m

Case-4 earthquake : y-direction

Vertical	Force (ton)	y (m)	Moment (ton•m)
WA	-227.302	2.275	-517.112
Pz	-31.107	2.000	-62.213
ΣV=	-258.409	ΣV•x=	-579.325

Horizontal	Force (ton)	z (m)	Moment (ton•m)
FwA	34.095	2.500	85.238
Fp	4.865	3.000	14.595
Py	31.613	3.000	94.839
ΣH=	70.573	ΣH•z=	194.672

ΣM= ΣV•x-ΣH•z= -773.998

Safety for overturning

e= 0.636 < 0.787

Safety for sliding

Fs = 2.38 > 2.0

Safety for bearing capacity

σσmax= 20.12 < 100 t/m2

3. Head Tank

3. HEAD TANK

3.1 Structural Calculation

(1) Structural model

The structural model,type I and II are shown in the Fig.3.

(2) Structural calculation

The structural analysis of frame were made by applying the SAP90 computer program, (Structural Analysis Program), that use the " Finite Element Method ". This program represent the research work conducted at the University of California, Berkeley, by Professor Edward L. Wilson.

(3) Loading condition

The structural analysis were made for the loading conditions of the following four cases and are illustrated in Fig. 3 . 1 . 2

Case 1 : the tank is empty without earthquake effect.

Case 2 : the one side of the tank is fulled with water at HWL and the other is empty without earthquake affect.

Case 3 : the both side of the tank is fulled with water and earthquake effect is not taken account.

Case 4 : the both side of the tank is also fulled with water and earthquake effect is taken into consideration.

The results of internal forces such as bending moment (M), shearing force (Q) and axial forces for each case are attached hereinafter.

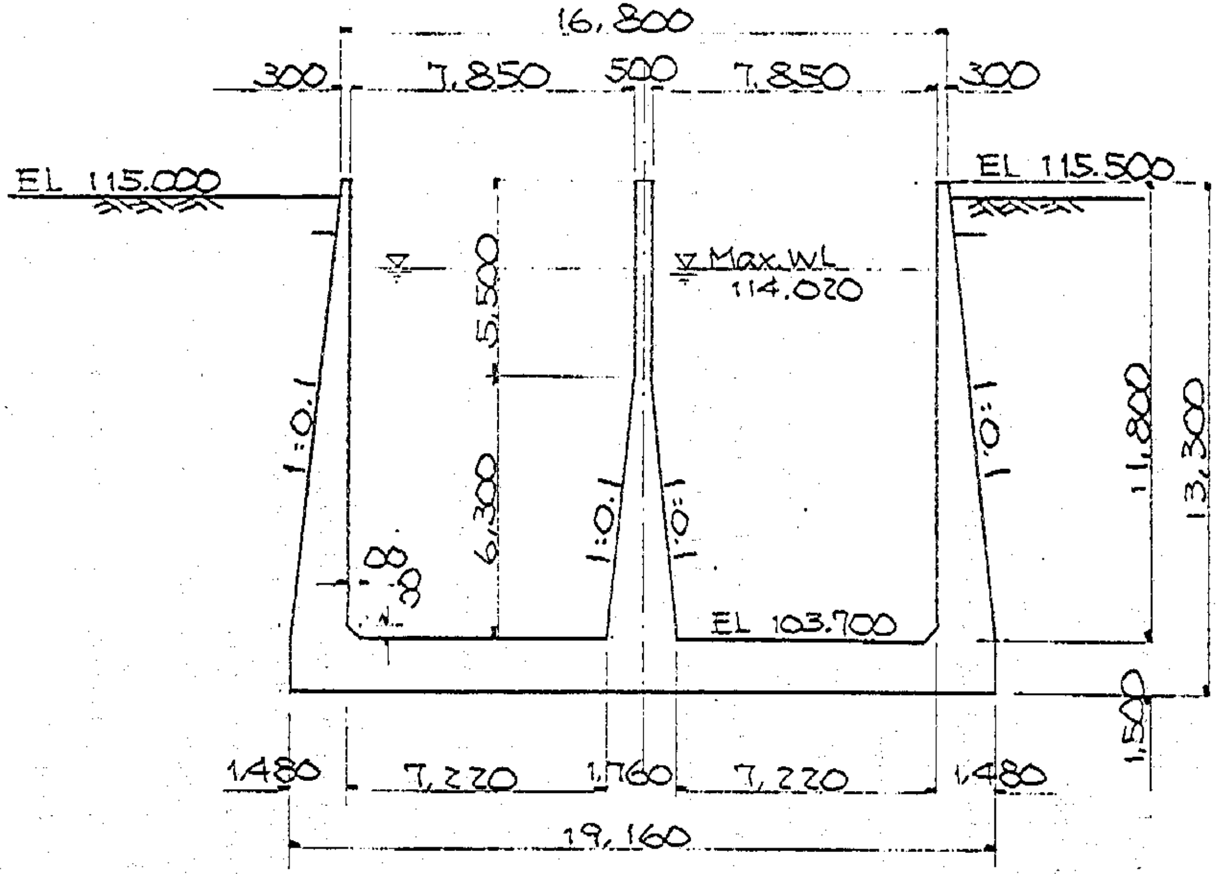


FIG. 3.1.1(a) HEAD TANK
TYPE I

2

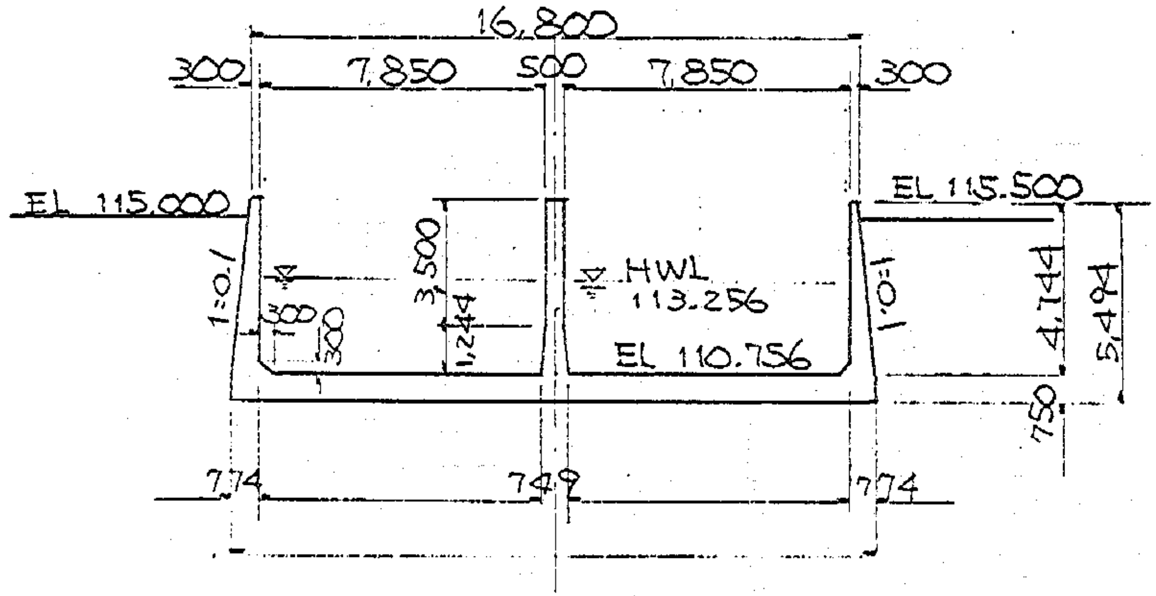


FIG. 3.1.1 (b) HEAD TANK
TYPE II

LOADS HEAD TANK TYPE I

No.	DESCRIPTION	
1	Self Weight	D
2	Earth Pressure due to Live Load	L
3	Earth Pressure	H
4	Foundation Reaction (1)	R1
5	Water Pressure (1)	F1
6	Foundation Reaction (2)	R2
7	Foundation Reaction (3)	R3
8	Water Pressure (2)	F2
9	Earthquake	E

COMBINATIONS

C1	D+L+H+R1
C2	D+L+H+R1+F1+R2
C3	D+L+H+R1+F1+R2+R3+F2
C4	D+L+H+R1+F1+R2+R3+F2+E
C5	1.4D+1.7L+1.7H+1.4R1
C6	1.4D+1.7L+1.7H+1.4R1+1.4F+1.4R2
C7	1.4D+1.7L+1.7H+1.4R1+1.4F+1.4R2+1.4F2
C8	1.05D+1.28L+1.28H+1.05R1+1.05F1+1.05R2+1.05R3+1.05F2+1.4E
C9	0.90D+1.7H+1.4R1+1.4F1+1.4R2+1.4R3+1.4F2+1.43E

Fig 3.1.2(a) HEAD TANK TYPE I

CASE 1 EMPTY WITHOUT EARTHQUAKE

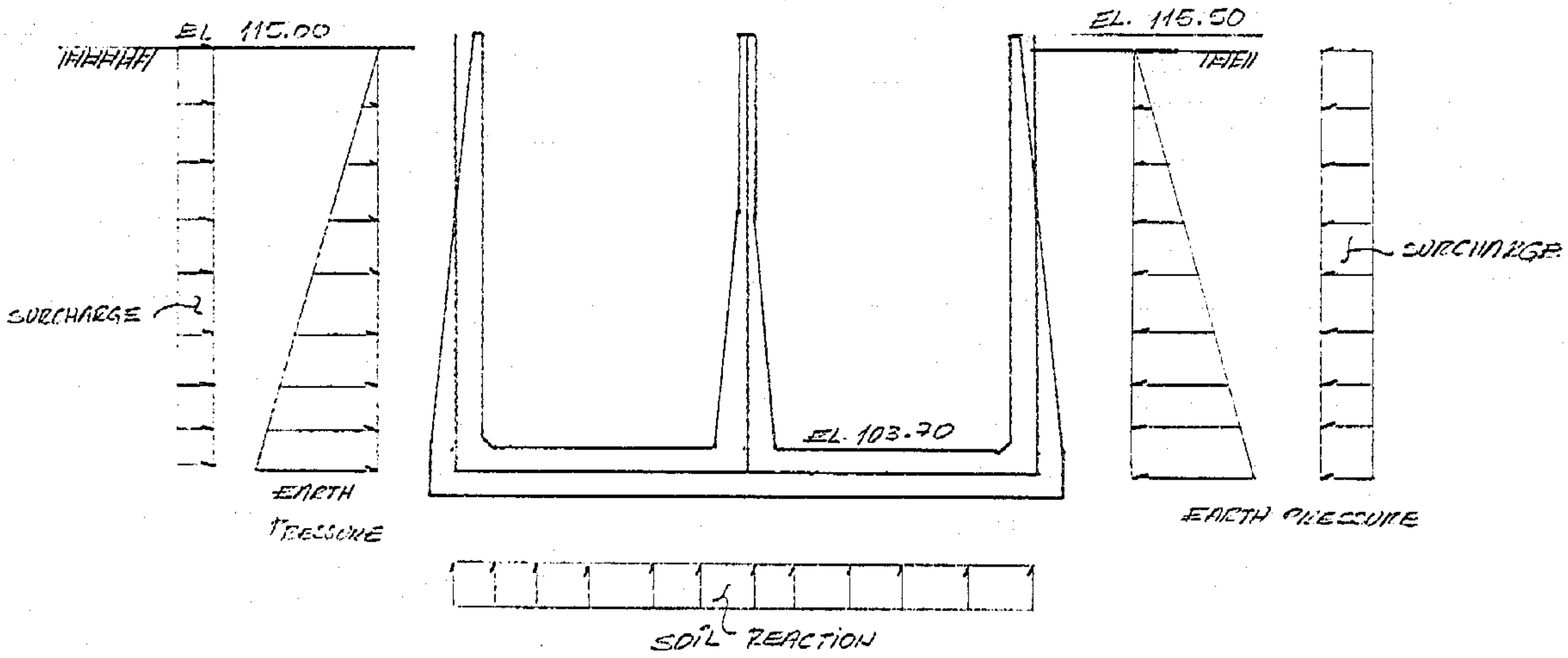


Fig. 3.1.2(b) CASE II : HWL in one side
empty in the other side

HEAD TANK TYPE I

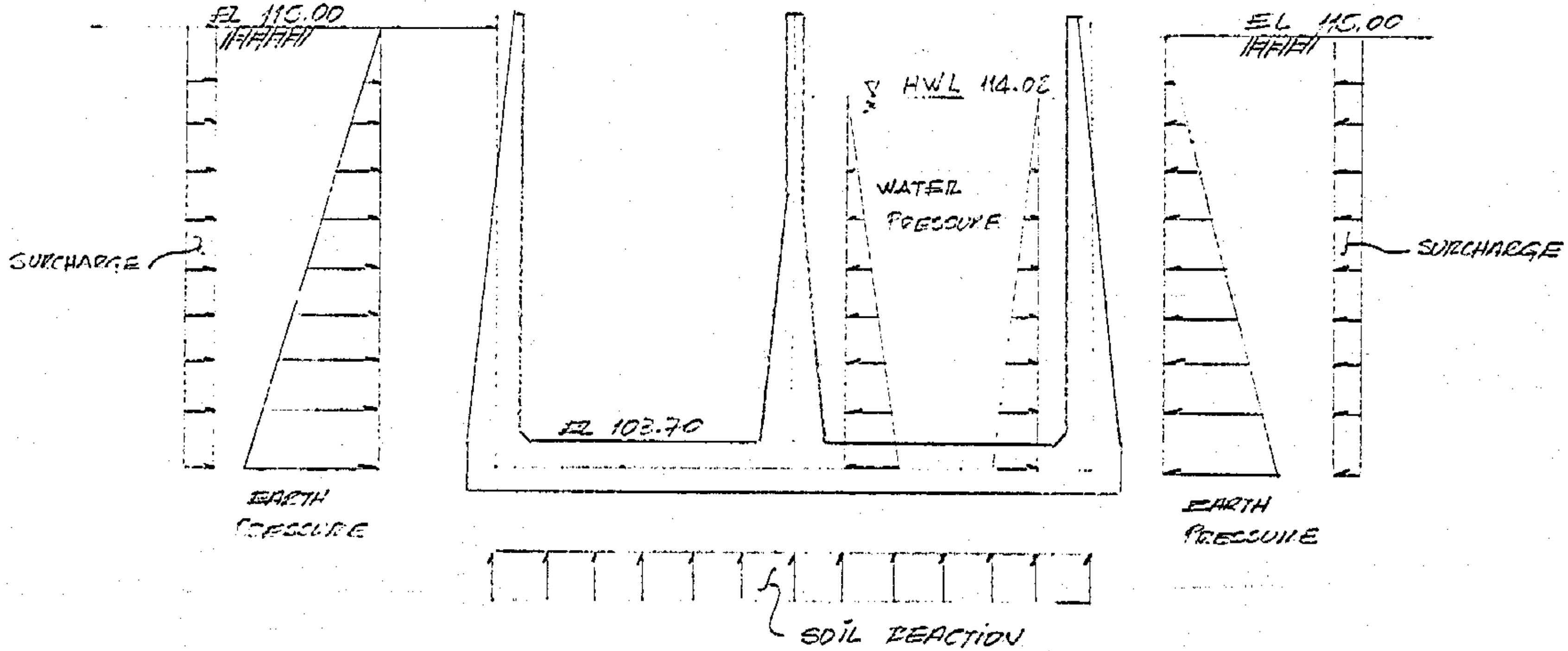
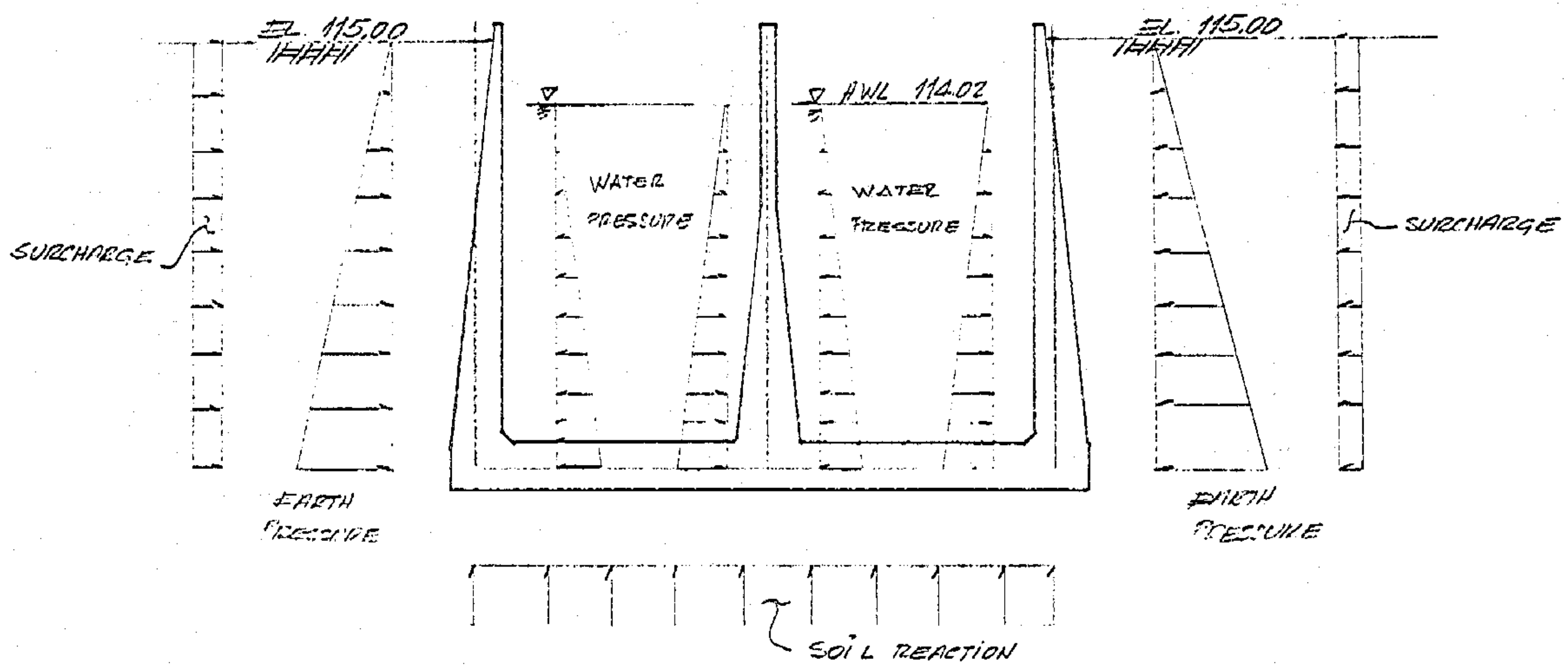


Fig. 3.1.2(c) HEAD TANK TYPE I

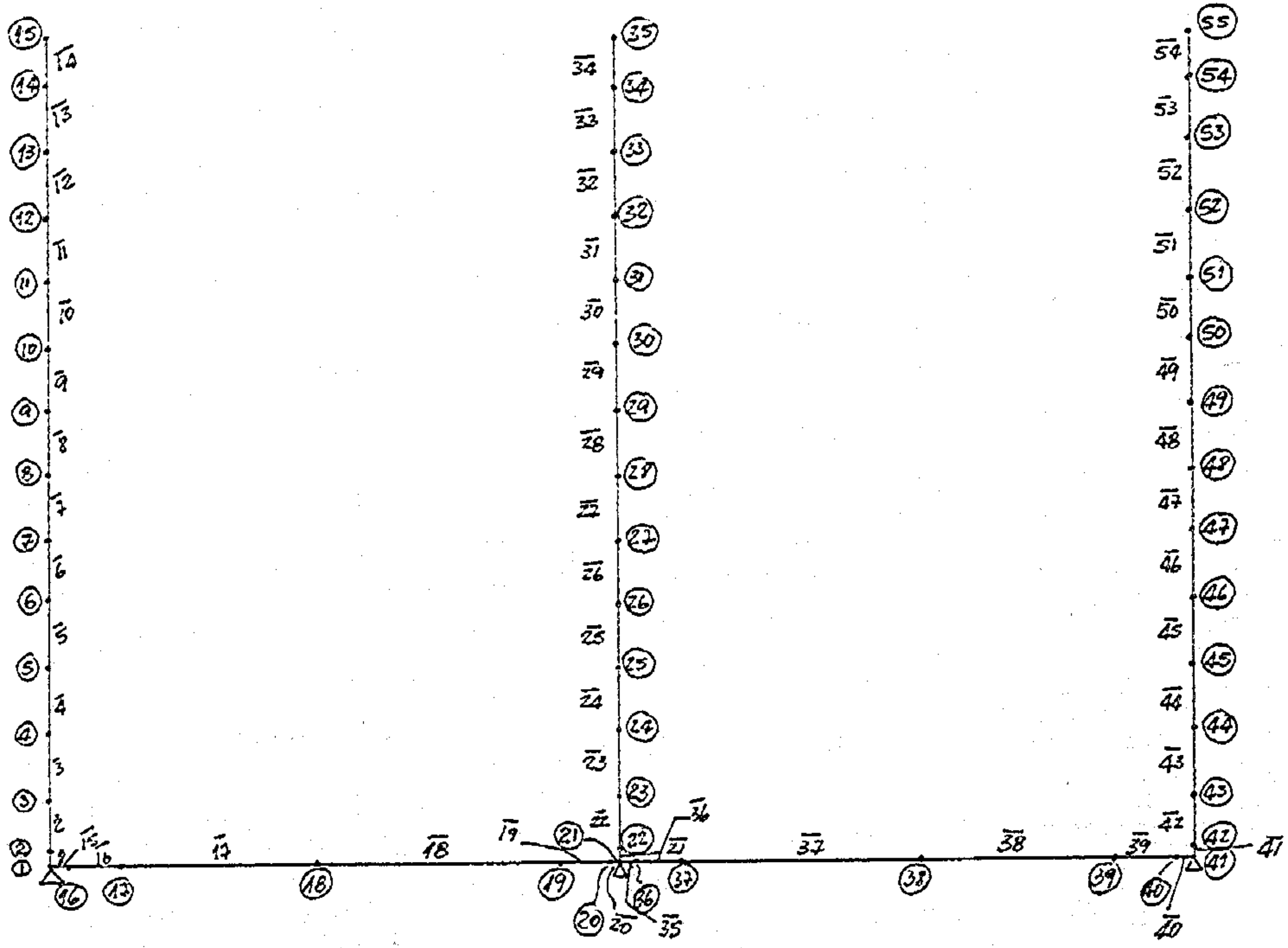
CASE 3.- HWL in both side without earthquake

CASE 4. HWL in both side with earthquake



7

NUMBER OF NODES AND MEMBERS
TYPE I



HEAD TANK TYPE I

JOINT	X	Y	q soil	d (m)	Weight
1	0.00	0.00	11.03	1.500	3.60
2	0.00	0.01	11.02	1.500	3.60
3	0.00	1.00	10.13	1.475	3.54
4	0.00	2.00	9.23	1.375	3.30
5	0.00	3.00	8.33	1.275	3.06
6	0.00	4.00	7.43	1.175	2.82
7	0.00	5.00	6.53	1.075	2.58
8	0.00	6.00	5.63	0.975	2.34
9	0.00	7.00	4.73	0.875	2.10
10	0.00	8.00	3.83	0.775	1.86
11	0.00	9.00	2.93	0.675	1.62
12	0.00	10.00	2.03	0.575	1.38
13	0.00	11.00	1.13	0.475	1.14
14	0.00	12.00	0.23	0.375	0.90
15	0.00	12.75		0.300	0.72
16	0.01	0.00			
17	0.75	0.00			
18	4.43	0.00			
19	7.95	0.00			
20	8.84	0.00			
21	8.85	0.00	11.57	1.80	4.32
22	8.85	0.01	11.56	1.80	4.32
23	8.85	1.00	10.57	1.75	4.20
24	8.85	2.00	9.57	1.55	3.72
25	8.85	3.00	8.57	1.35	3.24
26	8.85	4.00	7.57	1.15	2.76
27	8.85	5.00	6.57	0.95	2.28
28	8.85	6.00	5.57	0.75	1.80
29	8.85	7.00	4.57	0.55	1.32
30	8.85	8.00	3.57	0.50	1.20
31	8.85	9.00	2.57	0.50	1.20
32	8.85	10.00	1.57	0.50	1.20
33	8.85	11.00	0.57	0.50	1.20
34	8.85	12.00		0.50	1.20
35	8.85	12.75		0.50	1.20
36	8.86	0.00			
37	9.75	0.00			
38	13.28	0.00			
39	16.95	0.00			
40	17.69	0.00			
41	17.70	0.00			
42	17.70	0.01			
43	17.70	1.00			
44	17.70	2.00			
45	17.70	3.00			
46	17.70	4.00			
47	17.70	5.00			
48	17.70	6.00			
49	17.70	7.00			
50	17.70	8.00			
51	17.70	9.00			
52	17.70	10.00			
53	17.70	11.00			
54	17.70	12.00			
55	17.70	12.75			

HEAD TANK TYPE I

LOADS

CASE 1.- Earth Pressure + Surcharge

1.1 Self Weight :

Element	V (m3)	W (ton)	
Bottom slab	28.8	69.12	
Side Wall (2)	21.86	52.46	
Central Wall	10.23	24.55	
Total		146.14	ton

1.2 Foundation Reaction :

B = 17.7 m

qs1 = W / B = 8.26 t/m

1.3 Earth Pressure :

H = 12.26 m

r = 1.8 t/m3

Ka = 0.5

p = r.Ka.H = 11.03 t/m²

1.4 Surcharge :

h = 0.61 m

r = 1.8 t/m3

Ka = 0.5

p = r.Ka.H = 0.55 t/m²

CASE 2.- Earth Pressure + Surcharge + Water Pressure (one side)

2.1 Water :

h = 11.57 m

b = 8.85 m

Ww = 102.39 ton

2.2 Foundation Reaction :

B = 17.70 m

W + Ww = 248.53 ton

qs2 = (W + Ww) / B = 14.04 t/m

qs2 - qs1 = 5.79 t/m

CASE 3.- Earth Pressure + Surcharge + Water Pressure (two side)

3.1 Water :

h = 11.57 m

b = 8.85 m

Ww = 204.79 ton

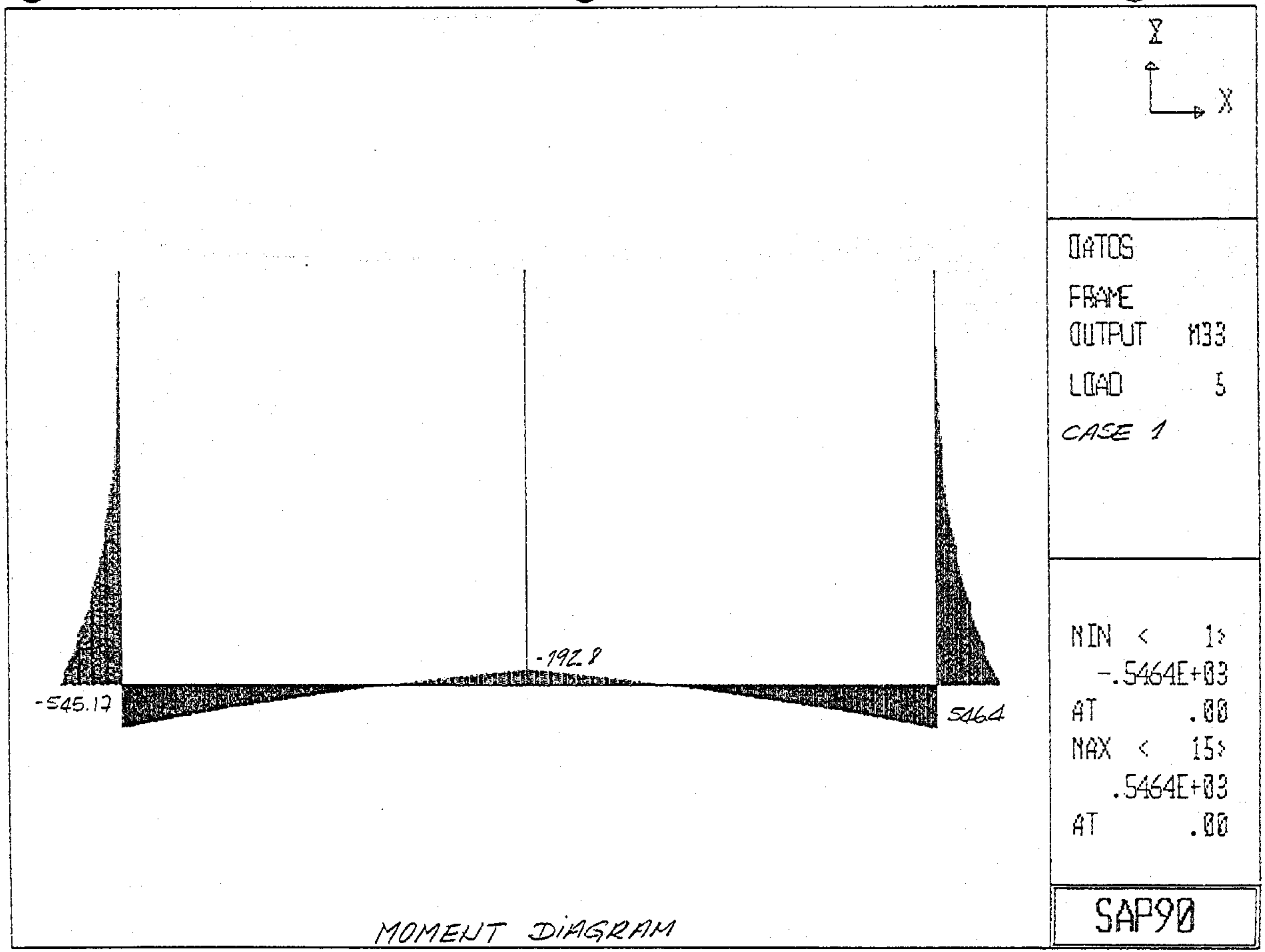
3.2 Foundation Reaction :

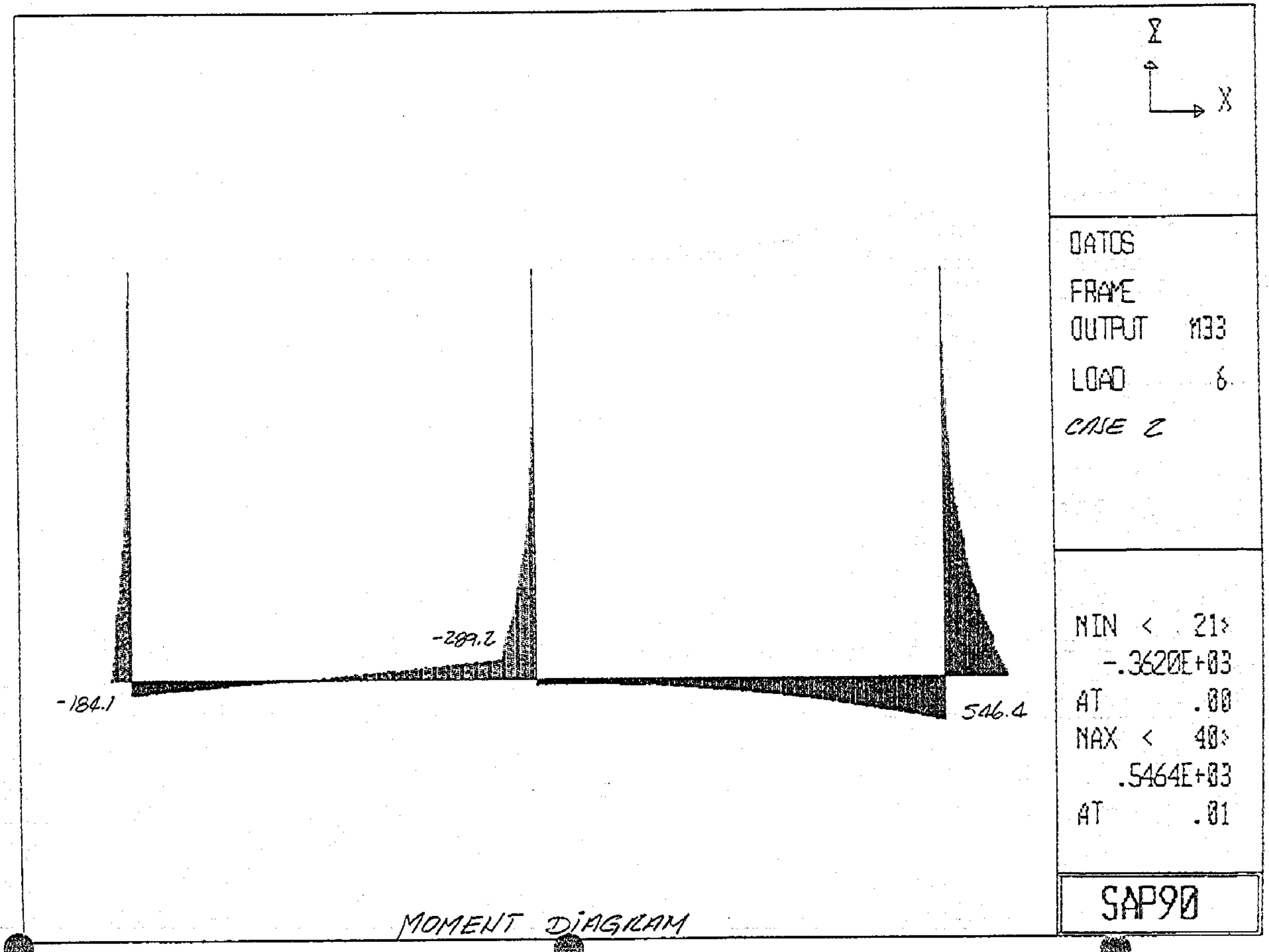
B = 17.70 m

W + Ww = 350.93 ton

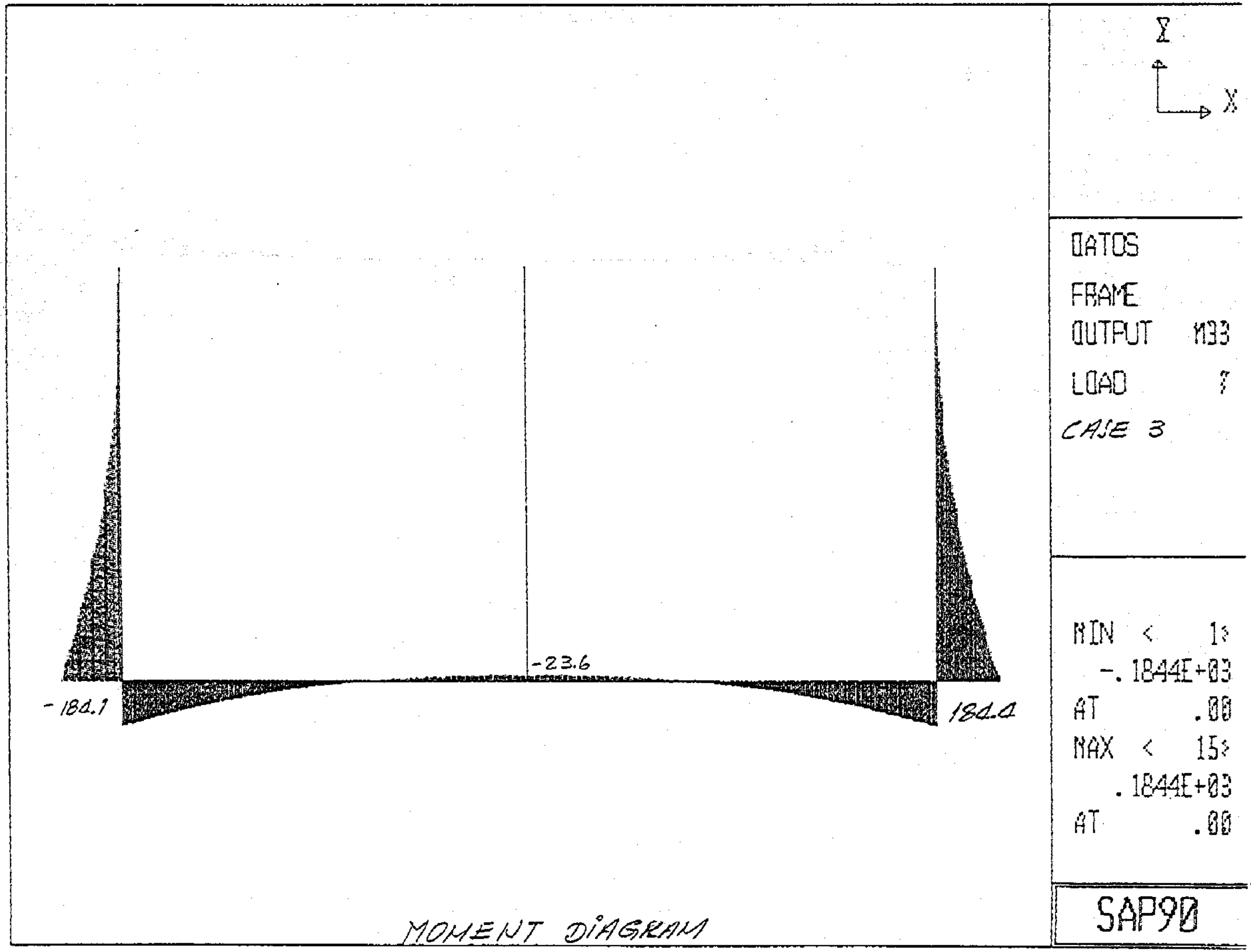
qs3 = (W + Ww) / B = 19.83 t/m

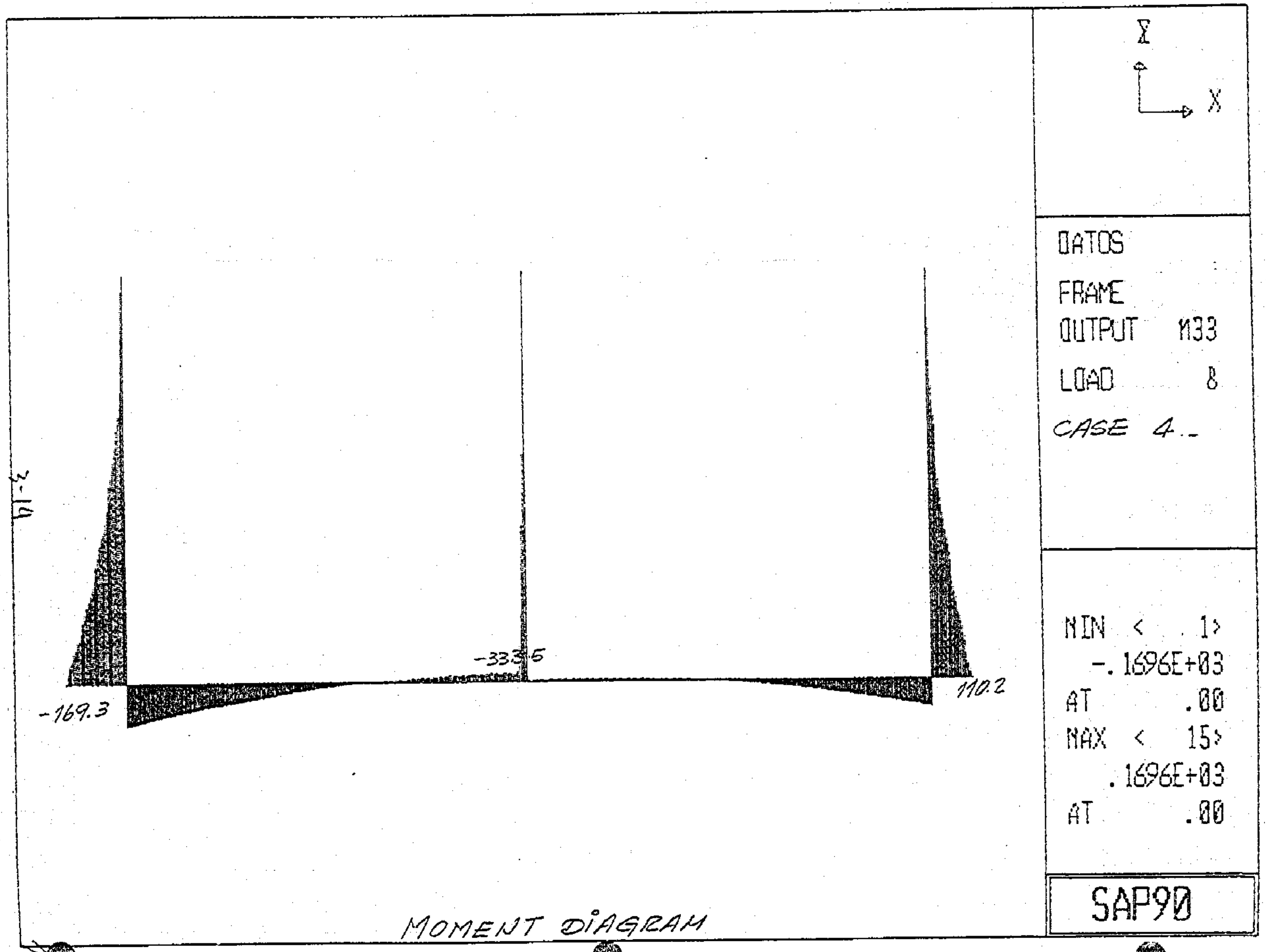
qs3 - qs2 = 5.79 t/m

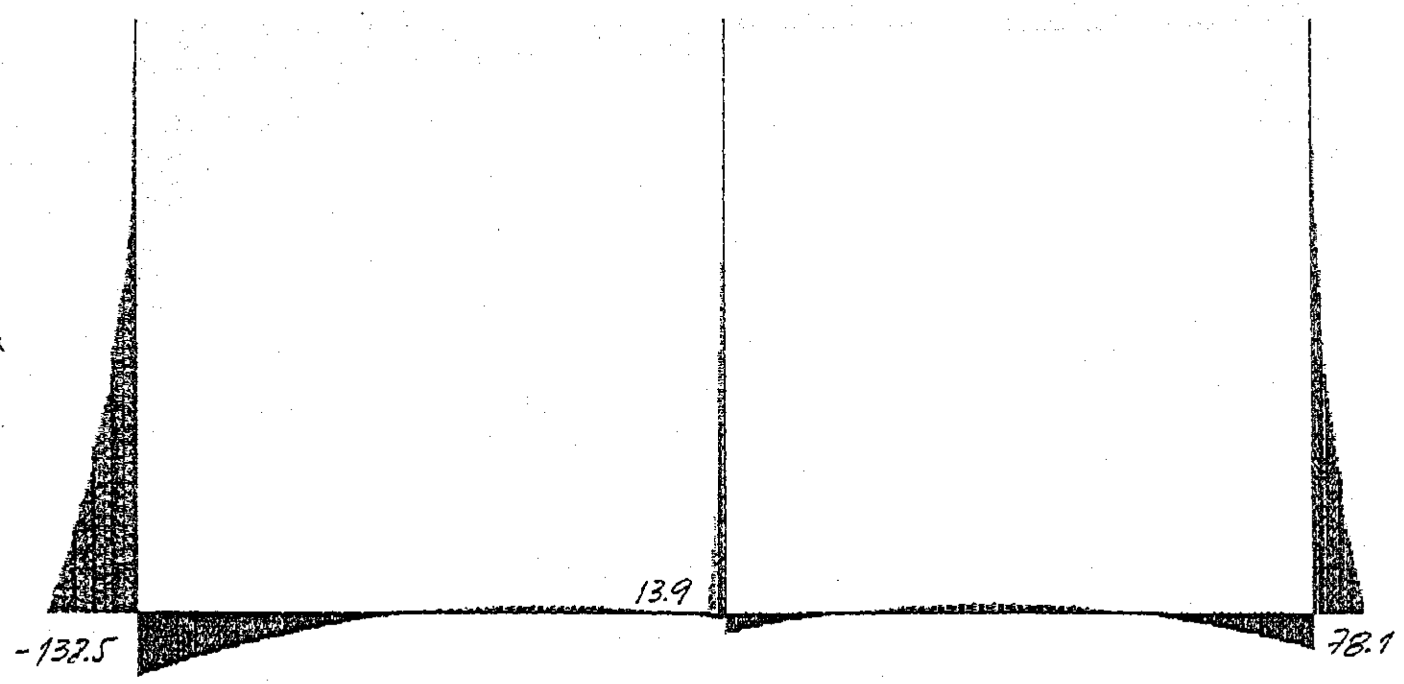

Y
X
DATOS
FRAME
OUTPUT M33
LOAD 5
CASE 1
MIN < 1>
-.5464E+03
AT .00
MAX < 15>
.5464E+03
AT .00
SAP90
-545.17
-192.8
546.4
MOMENT DIAGRAM



DATOS
FRAME
OUTPUT M33
LOAD 6
CASE 2
MIN < 21>
-.3620E+03
AT .00
MAX < 40>
.5464E+03
AT .01
SAP90
-184.1
-289.2
546.4
MOMENT DIAGRAM


Y
X
DATOS
FRAME
OUTPUT M33
LOAD 7
CASE 3
MIN < 1>
-.1844E+03
AT .00
MAX < 15>
.1844E+03
AT .00
SAP90
-23.6
-184.1
184.4
MOMENT DIAGRAM


Y
X
DATOS
FRAME
OUTPUT M33
LOAD 8
CASE 4.-
MIN < 1>
-.1696E+03
AT .00
MAX < 15>
.1696E+03
AT .00
SAP90
-169.3
-333.5
110.2
3-14
MOMENT DIAGRAM



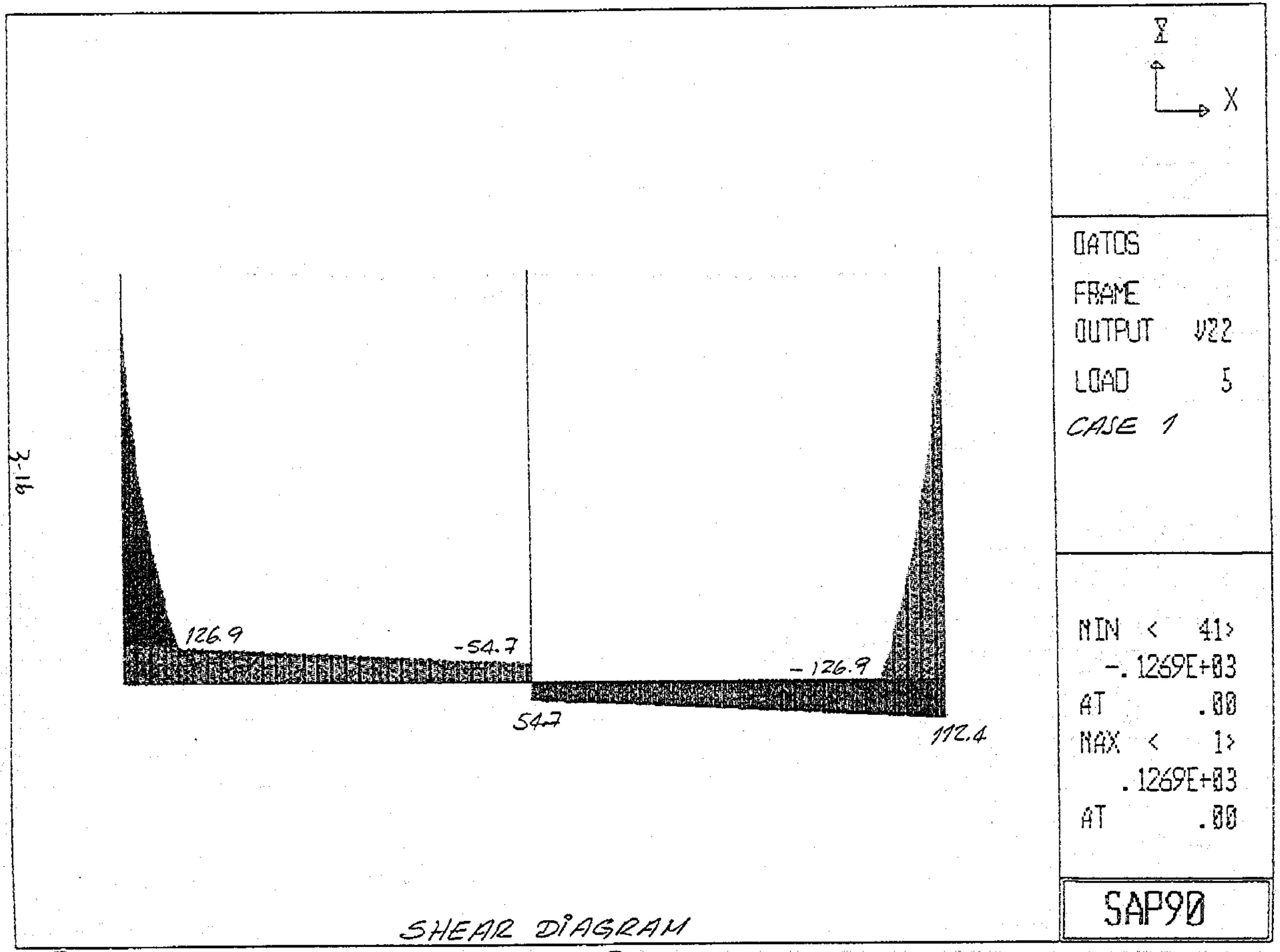
MOMENT DIAGRAM

DATOS

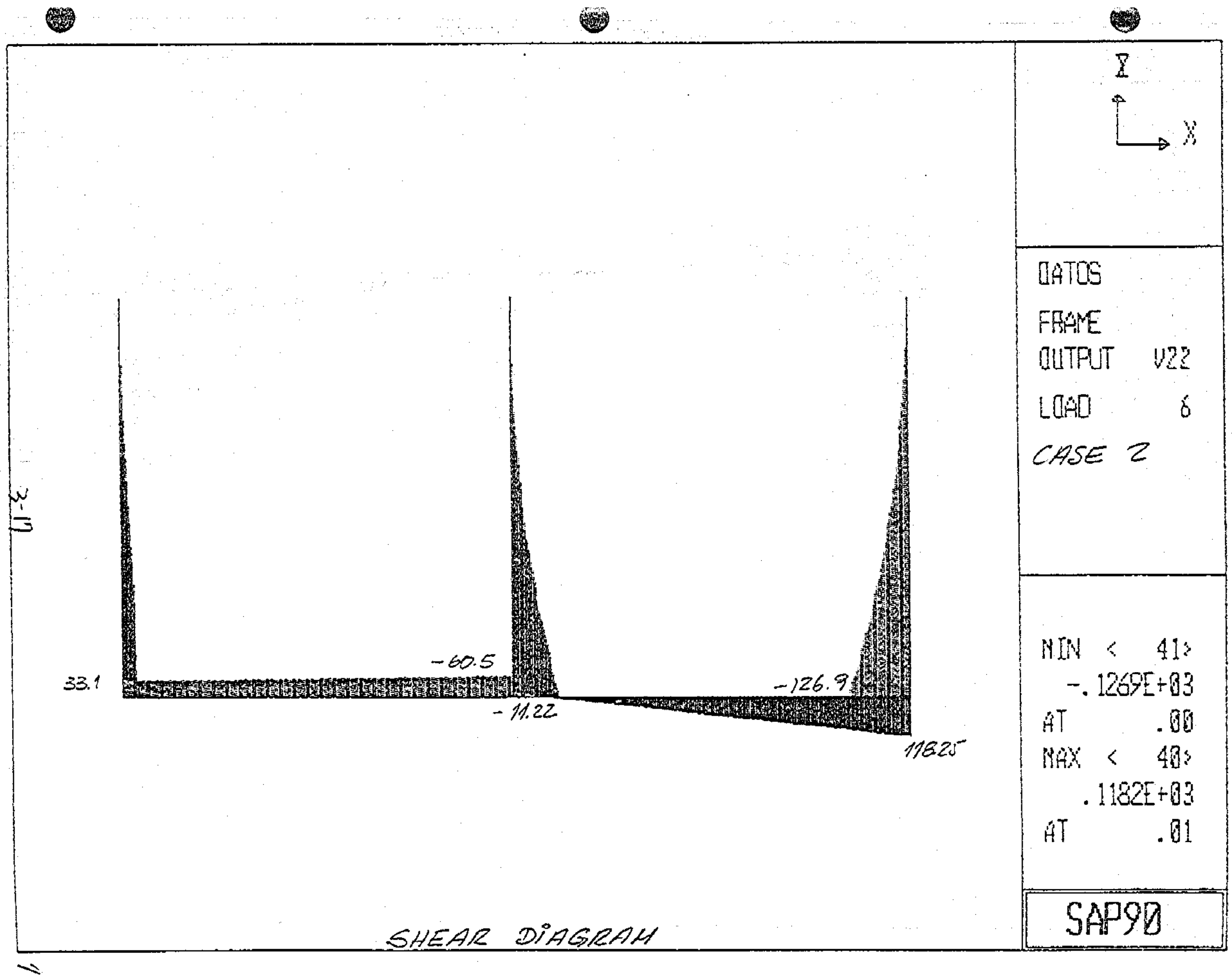
FRAME
OUTPUT M33
LOAD 9
CASE 4.-

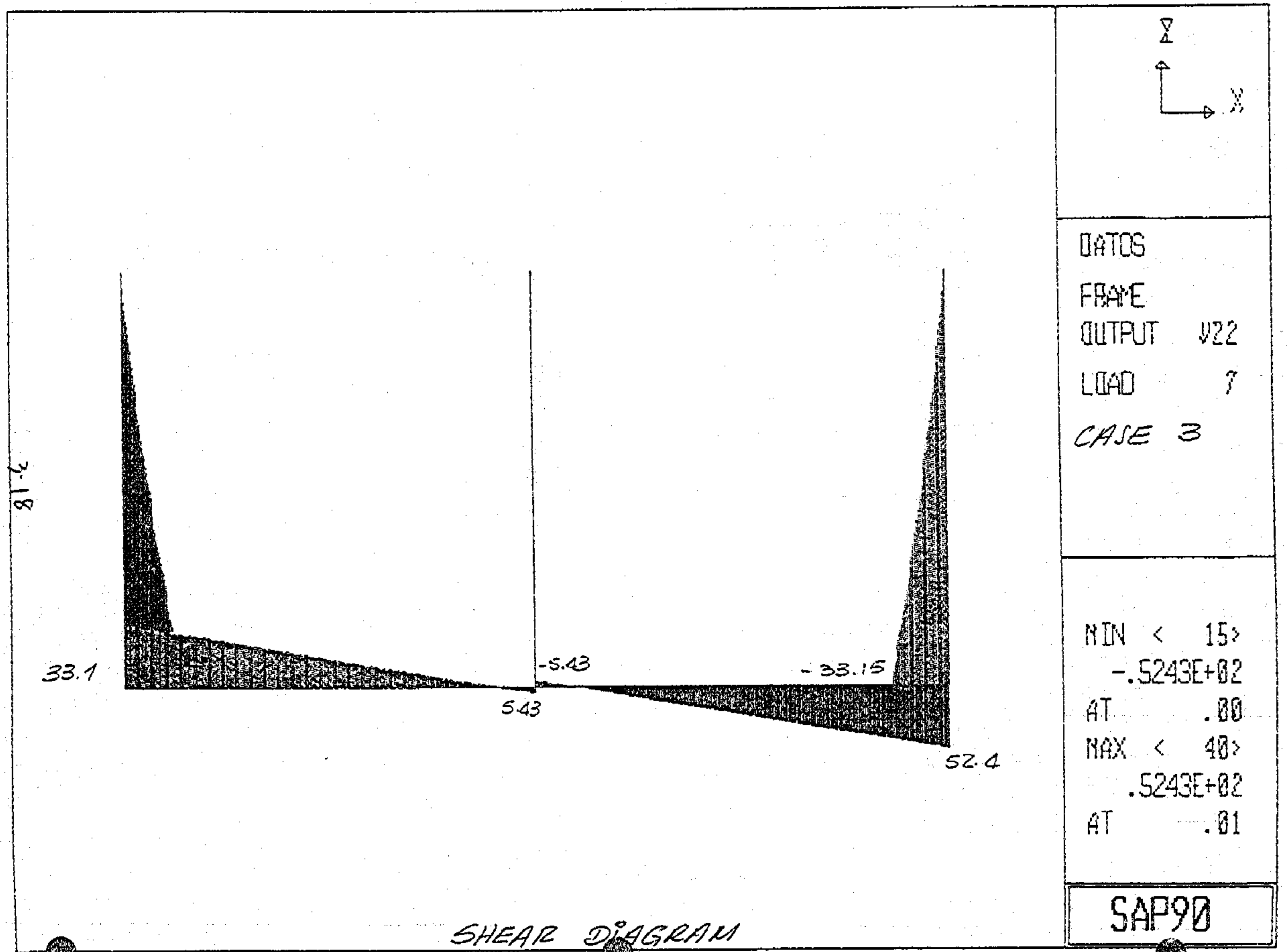
MIN < 1>
-.1388E+03
AT .00
MAX < 15>
.1388E+03
AT .00

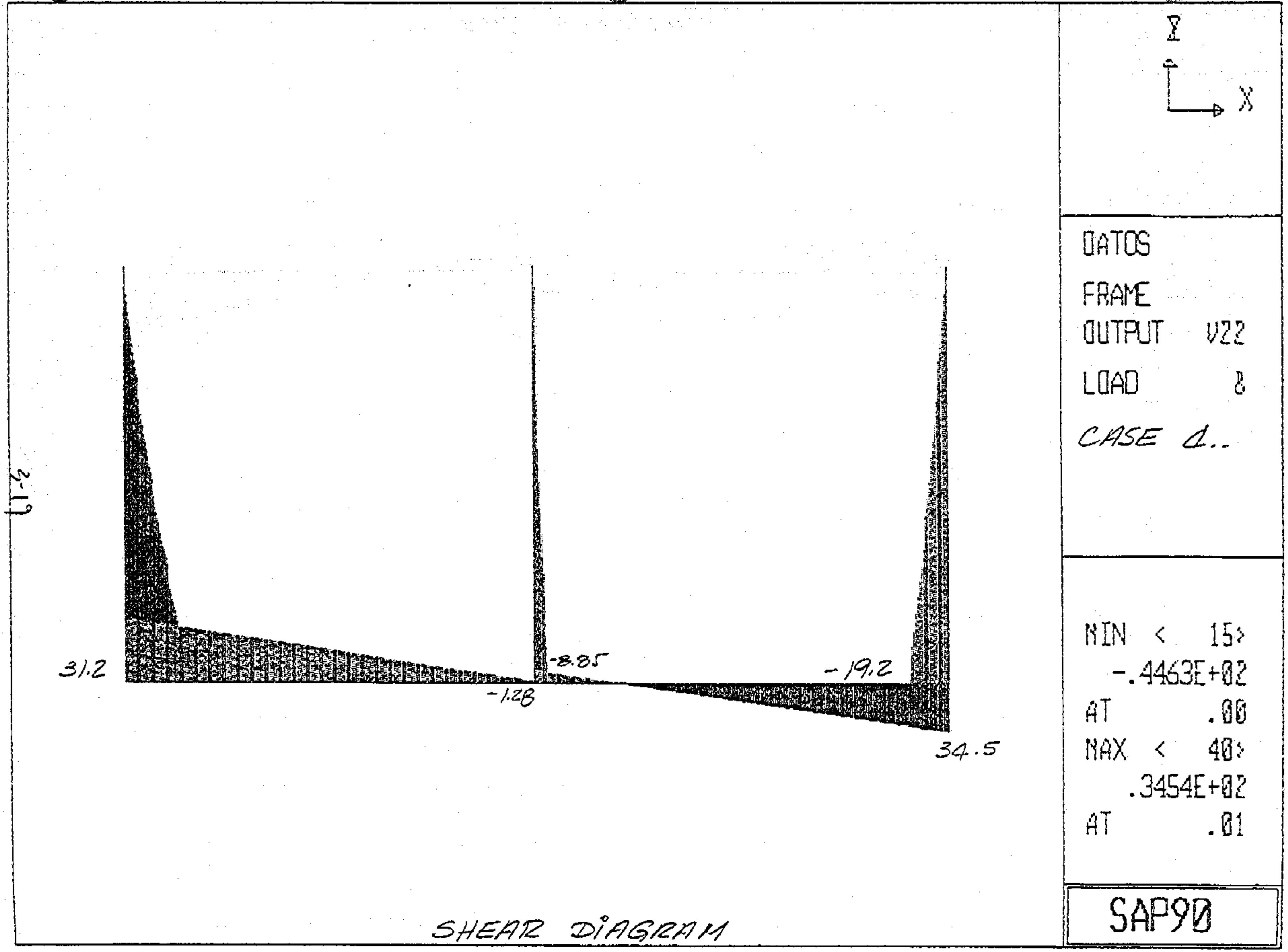
SAP90

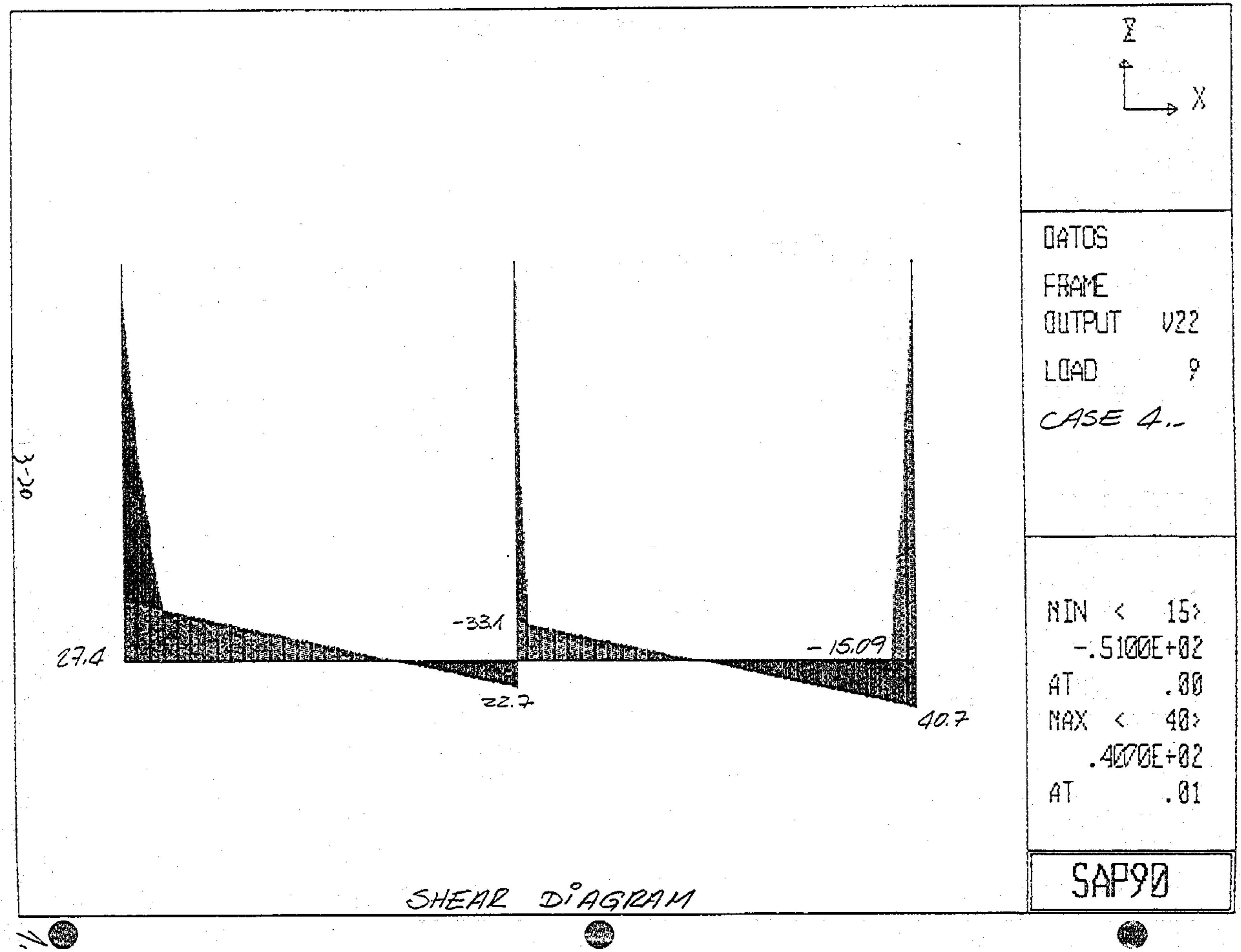


Y
X
126.9
-54.7
54.7
-126.9
112.4
DATOS
FRAME
OUTPUT V22
LOAD 5
CASE 1
MIN < 41>
-.1269E+03
AT .00
MAX < 1>
.1269E+03
AT .00
SAP90
SHEAR DIAGRAM


Y
X
DATOS
FRAME
OUTPUT V22
LOAD 6
CASE 2
MIN < 41>
-.1269E+03
AT .00
MAX < 40>
.1182E+03
AT .01
SAP90
33.1
-60.5
-11.22
-126.9
118.25
SHEAR DIAGRAM
3-17
1


Y
X
DATOS
FRAME
OUTPUT V22
LOAD 7
CASE 3
MIN < 15>
-.5243E+02
AT .00
MAX < 40>
.5243E+02
AT .01
SAP90
33.1
-5.43
5.43
-33.15
52.4
SHEAR DIAGRAM


Y
X
DATOS
FRAME
OUTPUT V22
LOAD 8
CASE d..
31.2
-1.28
-8.85
-19.2
34.5
MIN < 15>
-.4463E+02
AT .00
MAX < 40>
.3454E+02
AT .01
SAP90
SHEAR DIAGRAM


Z
X
DATOS
FRAME
OUTPUT V22
LOAD 9
CASE 4.-
MIN < 15>
-.5100E+02
AT .00
MAX < 40>
.4070E+02
AT .01
SAP90
27.4
-33.1
22.7
-15.09
40.7
SHEAR DIAGRAM
3-20
1.

HEAD TANK TYPE I

GIVEN :

f'c = 210 Kg/cm²

fy = 4200 "

SHEAR STRENGTH DESIGN

ID ELEM	Vu (ton)	bw (cm)	dn (cm)	h (cm)	d(adopt) (cm)	øVc (ton)
2	107.99	100.0	165.41	180.0	175.00	114.25
3	90.59	100.0	138.77	140.0	135.00	88.13
4	74.73	100.0	114.47	127.5	122.50	79.97
5	60.40	100.0	92.52	117.5	112.50	73.44
6	47.60	100.0	72.91	107.5	102.50	66.92
7	36.33	100.0	55.65	97.5	92.50	60.39
8	26.59	100.0	40.73	87.5	82.50	53.86
9	18.38	100.0	28.15	77.5	72.50	47.33
10	11.70	100.0	17.92	67.5	62.50	40.80
11	6.55	100.0	10.03	57.5	52.50	34.27
12	2.93	100.0	4.48	47.5	42.50	27.75
13	0.86	100.0	1.31	37.5	32.50	21.22
14	0.01	100.0	0.01	30.0	25.00	16.32
16	107.50	100.0	164.67	170.0	165.00	107.72
17	83.53	100.0	127.94	150.0	145.00	94.66
23	78.26	100.0	119.88	175	170.00	110.98
28	14.67	100.0	22.47	50	45.00	29.38
39	107.50	100.0	164.67	170.0	165.00	107.72

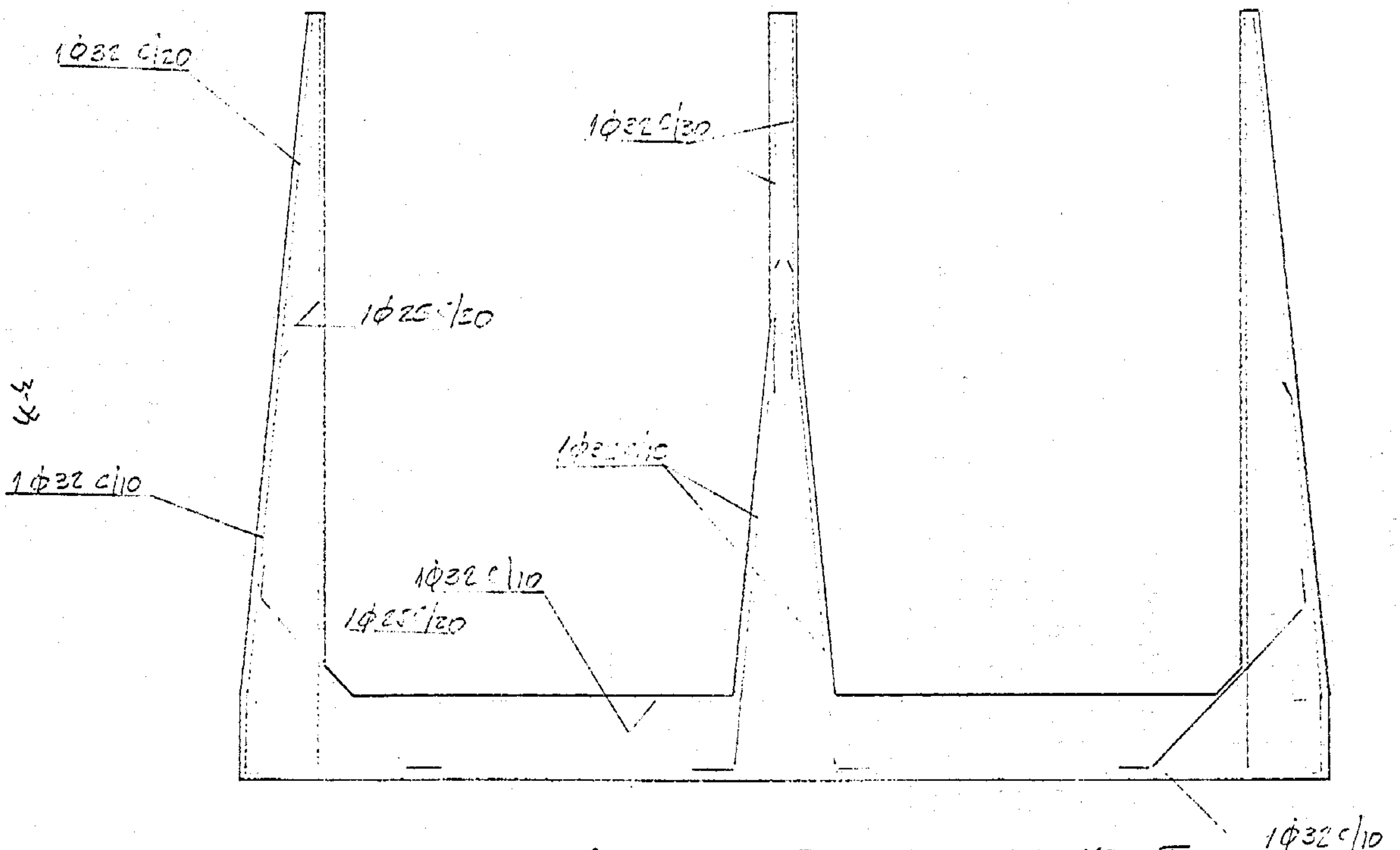
HEAD TANK TYPE I

FLEXURE STRENGTH DESIGN

GIVEN :

f'c =	210	Kg/cm²
fy =	4200	"
r =	7.5	cm
ρmax = 0.75ρb =	1.61	(%)
ρs =	0.90	(%)

ID ELEM	Mu (t.m)	b (cm)	h (cm)	d (cm)	ρ (%)	As (cm²)	As(min) (cm²)	As(temp) (cm²)	As(adopt) (cm²)	As(adopt) (varillas)
1	546.43	100	150.0	142.5	0.78	111.79	47.50	18.75		
2	545.16	100	150.0	142.5	0.78	111.50	47.50	18.75		
3	429.11	100	147.5	140.0	0.63	87.55	46.67	18.44		
4	329.95	100	137.5	130.0	0.55	71.83	43.33	17.19	80.43	1 ø 32 @ 10 cm.
5	247.41	100	127.5	120.0	0.48	57.63	40.00	15.94	80.43	1 ø 32 @ 10 cm.
6	179.97	100	117.5	110.0	0.41	45.50	36.67	14.69	40.21	1 ø 32 @ 10 cm.
7	126.09	100	107.5	100.0	0.35	34.79	33.33	13.44	40.21	1 ø 32 @ 20 cm.
8	84.26	100	97.5	90.0	0.28	25.63	30.00	12.19	40.21	1 ø 32 @ 20 cm.
9	52.92	100	87.5	80.0	0.22	17.98	26.67	10.94	40.21	1 ø 32 @ 20 cm.
10	30.57	100	77.5	70.0	0.17	11.79	23.33	9.69	24.54	1 ø 32 @ 20 cm.
11	15.66	100	67.5	60.0	0.12	7.00	20.00	8.44	24.54	1 ø 32 @ 20 cm.
12	6.65	100	57.5	50.0	0.07	3.55	16.67	7.19	24.54	1 ø 32 @ 20 cm.
13	2.05	100	47.5	40.0	0.03	1.30	13.33	5.94	24.54	1 ø 32 @ 20 cm.
14	0.30	100	37.5	30.0	0.01	0.26	10.00	4.69	24.54	1 ø 32 @ 20 cm.
15	546.43	100	150.0	142.5	0.78	111.79	47.50	18.75		
16	545.30	100	150.0	142.5	0.78	111.54	47.50	18.75		
17	463.964	100	150.0	142.5	0.66	93.35	47.50	18.75	80.43	1 ø 32 @ 10 cm.
18	140.94	100	150.0	142.5	0.19	26.76	47.50	18.75	53.62	1 ø 32 @ 10 cm.
19	289.56	100	150.0	142.5	0.39	55.18	47.50	18.75	57.45	1 ø 32 @ 10 cm.
20	289.16	100	150.0	142.5	0.40	56.31	47.50	18.75	57.45	1 ø 32 @ 10 cm.
21	362.00	100	180.0	172.5	0.34	57.80	57.50	22.50		
22	351.07	100	175.0	167.5	0.36	59.52	55.83	21.88		
23	276.11	100	155.0	147.5	0.35	51.66	49.17	19.38	53.62	1 ø 32 @ 15 cm.
24	205.02	100	135.0	127.5	0.35	44.36	42.50	16.88	53.62	1 ø 32 @ 15 cm.
25	147.32	100	115.0	107.5	0.35	37.82	35.83	14.38	53.62	1 ø 32 @ 15 cm.
26	101.63	100	95.0	87.5	0.37	32.12	29.17	11.88	40.21	1 ø 32 @ 15 cm.
27	69.52	100	75.0	67.5	0.41	27.33	22.50	9.38	40.21	1 ø 32 @ 15 cm.
28	40.63	100	55.0	47.5	0.51	24.06	15.83	6.88	40.21	1 ø 32 @ 15 cm.
29	22.51	100	50.0	42.5	0.34	14.61	14.17	6.25	15.71	1 ø 32 @ 30 cm.
30	10.81	100	50.0	42.5	0.16	6.66	14.17	6.25	15.71	1 ø 32 @ 30 cm.
31	4.10	100	50.0	42.5	0.06	2.57	14.17	6.25	15.71	1 ø 32 @ 30 cm.
32	1.00	100	50.0	42.5	0.01	0.62	14.17	6.25	15.71	1 ø 32 @ 30 cm.
33	0.39	100	50.0	42.5	0.01	0.25	14.17	6.25	15.71	1 ø 32 @ 30 cm.
34	0.07	100	50.0	42.5	0.00	0.04	14.17	6.25	15.71	1 ø 32 @ 30 cm.
35	192.78	100	150.0	142.5	0.26	36.92	47.50	18.75	53.62	1 ø 32 @ 10 cm.
36	192.23	100	150.0	142.5	0.26	36.81	47.50	18.75	53.62	1 ø 32 @ 10 cm.
37	165.43	100	150.0	142.5	0.22	31.73	47.50	18.75	53.62	1 ø 32 @ 10 cm.
38	463.96	100	150.0	142.5	0.65	93.35	47.50	18.75	80.43	1 ø 32 @ 10 cm.
39	545.30	100	150.0	142.5	0.78	111.54	47.50	18.75		
40	546.43	100	150.0	142.5	0.78	111.79	47.50	18.75		
41	546.43	100	150.0	142.5	0.78	111.79	47.50	18.75		
42	545.16	100	150.0	142.5	0.78	111.50	47.50	18.75		
43	429.11	100	147.5	140.0	0.63	87.55	46.67	18.44		
44	329.95	100	137.5	130.0	0.55	71.83	43.33	17.19	80.43	1 ø 32 @ 10 cm.
45	247.41	100	127.5	120.0	0.48	57.63	40.00	15.94	80.43	1 ø 32 @ 10 cm.
46	179.97	100	117.5	110.0	0.41	45.50	36.67	14.69	40.21	1 ø 32 @ 10 cm.
47	126.09	100	107.5	100.0	0.35	34.79	33.33	13.44	40.21	1 ø 32 @ 20 cm.
48	84.26	100	97.5	90.0	0.28	25.63	30.00	12.19	40.21	1 ø 32 @ 20 cm.
49	52.92	100	87.5	80.0	0.22	17.98	26.67	10.94	40.21	1 ø 32 @ 20 cm.
50	30.57	100	77.5	70.0	0.17	11.79	23.33	9.69	24.54	1 ø 32 @ 20 cm.
51	15.66	100	67.5	60.0	0.12	7.00	20.00	8.44	24.54	1 ø 32 @ 20 cm.
52	6.66	100	57.5	50.0	0.07	3.55	16.67	7.19	24.54	1 ø 32 @ 20 cm.
53	2.05	100	47.5	40.0	0.03	1.30	13.33	5.94	24.54	1 ø 32 @ 20 cm.
54	0.30	100	37.5	30.0	0.01	0.26	10.00	4.69	24.54	1 ø 32 @ 20 cm.



REINFORCEMENT HEAD TANK TYPE I

LOADS HEAD TANK TYPE II

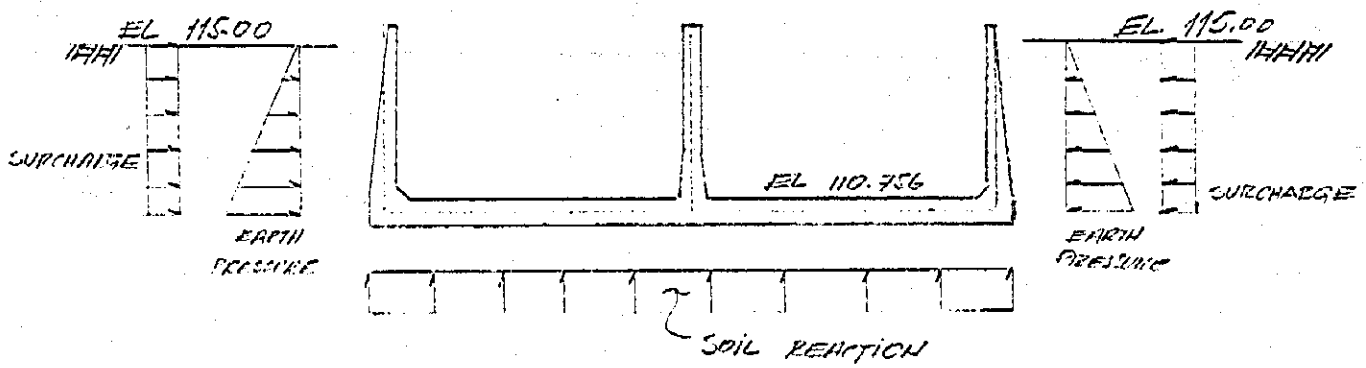
No.	DESCRIPTION	
1	Self Weight	D
2	Earth Pressure due to Live Load	L
3	Earth Pressure	H
4	Foundation Reaction (1)	R1
5	Water Pressure	F
5'	Water Weight	F
6	Foundation Reaction (2)	R2
7	Earthquake	E

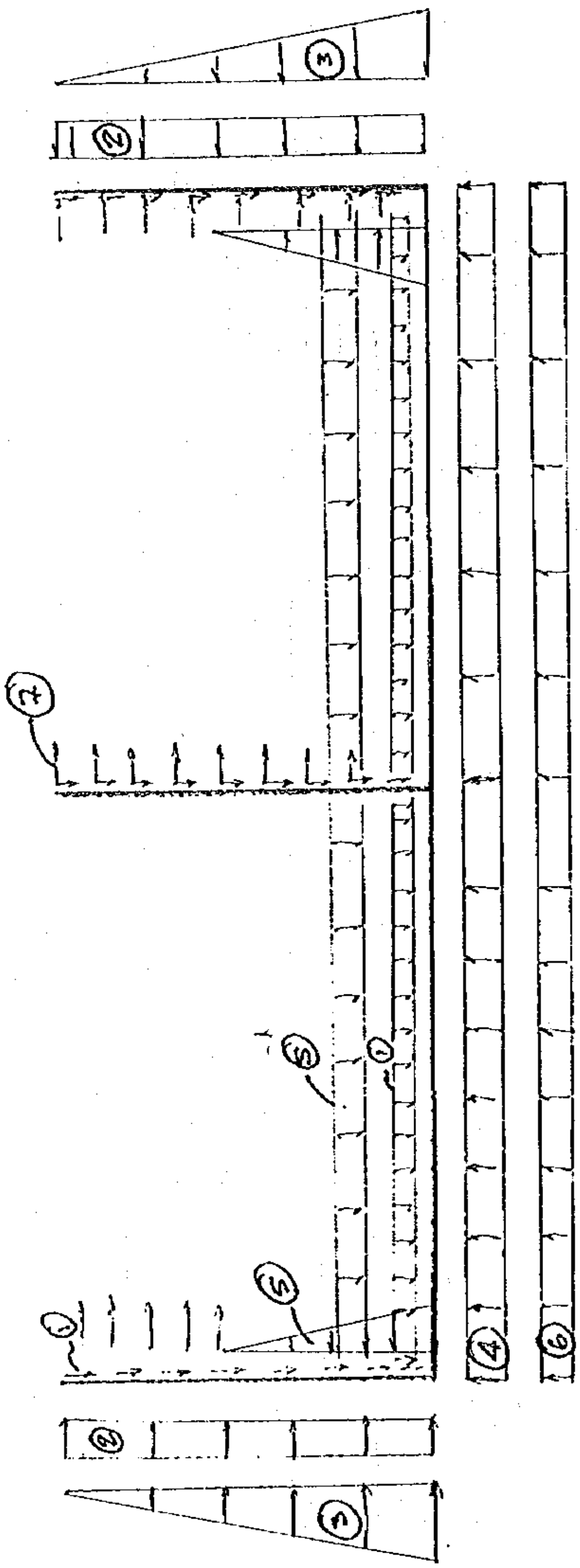
COMBINATIONS

C1	D+L+H+R1
C2	D+L+H+R1+F+R2
C3	D+L+H+R1+F+R2+E
C4	1.4D+1.7L+1.7H+1.4R1
C5	1.4D+1.7L+1.7H+1.4R1+1.4F+1.4R2
C6	0.75(1.4D+1.7L+1.7H+1.4R1+1.4F+1.4R2+1.87E)
C7	0.9D+1.7H+1.4R1+1.43E

HEAD TANK TYPE II

CASE 1. EMPTY WITHOUT EARTHQUAKE

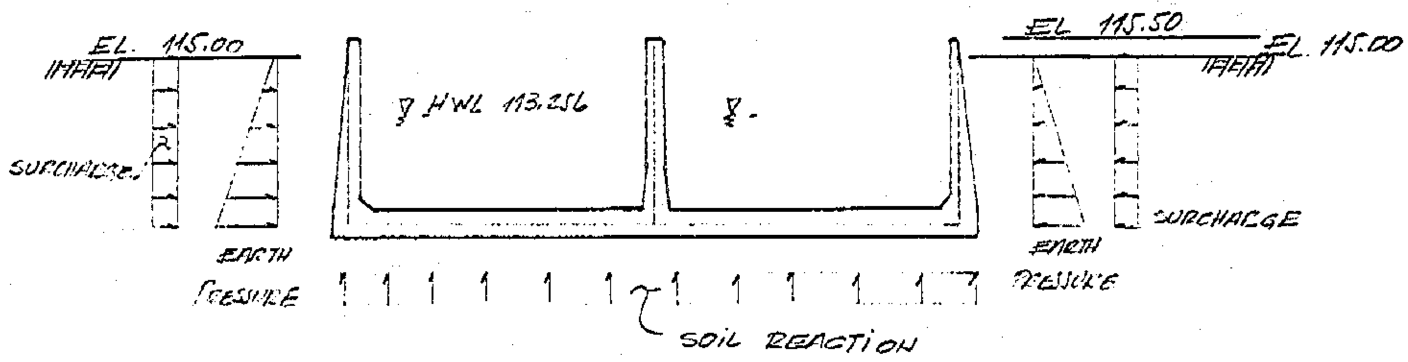




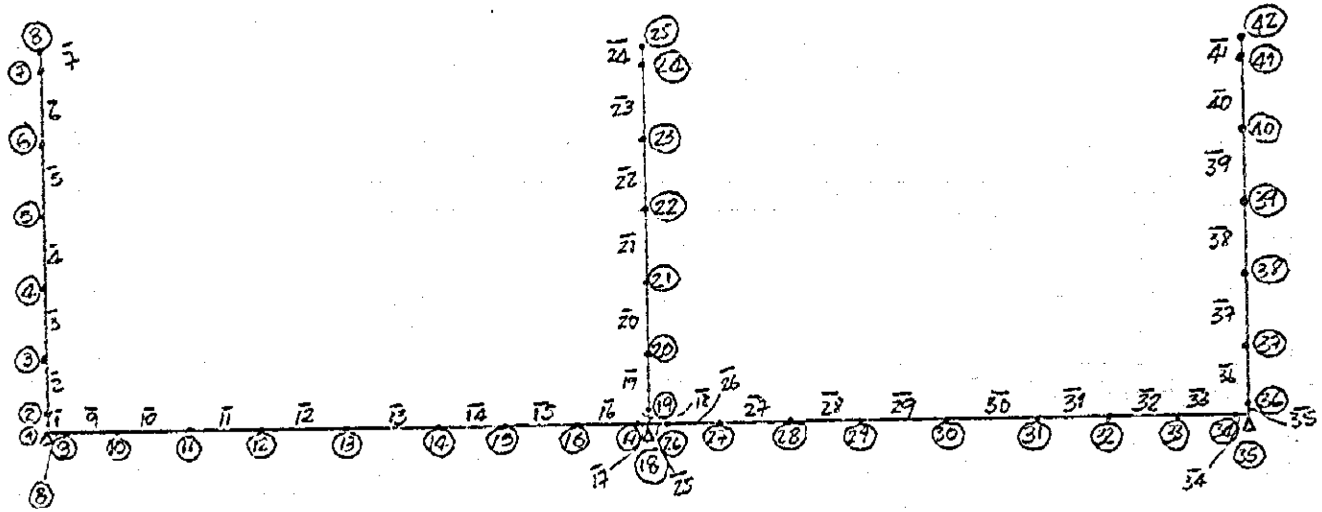
HEAD TANK TYPE II

CASE 2.. HWL IN BOTH SIDE WITHOUT EARTHQUAKE

CASE 3.. HWL IN BOTH SIDE WITH EARTHQUAKE



NUMBER OF NODES AND MEMBERS



HEAD TANK TYPE II

JOINT	X	Y	q soil	d (m)	Weight
1	0.00	0.00	4.32	0.790	1.90
2	0.00	0.01	4.31	0.790	1.90
3	0.00	1.00	3.42	0.730	1.75
4	0.00	2.00	2.52	0.630	1.51
5	0.00	3.00	1.62	0.530	1.27
6	0.00	4.00	0.72	0.430	1.03
7	0.00	5.00		0.330	0.79
8	0.00	5.30		0.300	0.72
9	0.01	0.00			
10	1.00	0.00			
11	2.00	0.00			
12	3.00	0.00			
13	4.25	0.00			
14	5.50	0.00			
15	6.50	0.00			
16	7.50	0.00			
17	8.49	0.00			
18	8.50	0.00		0.88	2.11
19	8.50	0.01		0.88	2.11
20	8.50	1.00		0.76	1.82
21	8.50	2.00		0.56	1.34
22	8.50	3.00		0.5	1.20
23	8.50	4.00		0.5	1.20
24	8.50	5.00		0.5	1.20
25	8.50	5.30		0.5	1.20
26	8.51	0.00			
27	9.50	0.00			
28	10.50	0.00			
29	11.50	0.00			
30	12.74	0.00			
31	13.99	0.00			
32	14.99	0.00			
33	15.99	0.00			
34	16.98	0.00			
35	16.99	0.00			
36	16.99	0.01			
37	16.99	1.00			
38	16.99	2.00			
39	16.99	3.00			
40	16.99	4.00			
41	16.99	5.00			
42	16.99	5.30			

HEAD TANK TYPE II

LOADS

CASE 1.- Earth Pressure + Surcharge

1.1 Self Weight :

Element	V (m3)	W (ton)	
Bottom slab	14.22	34.13	
Side Wall	5.43	13.03	
Central Wall	2.81	6.74	
Total		53.90	ton

1.2 Foundation Reaction :

B = 16.99 m

qs1 = W / B = 3.17 t/m

1.3 Earth Pressure :

H = 4.8 m

r = 1.8 t/m3

Ka = 0.5

p = r.Ka.H 4.32 t/m²

1.4 Surcharge :

h = 0.61 m

r = 1.8 t/m3

Ka = 0.5

p = r.Ka.H 0.55 t/m²

CASE 2.- Earth Pressure + Surcharge + Water Pressure (two sides)

2.1 Water :

h = 2.9 m

b = 8.50 m

Ww = 49.27 ton

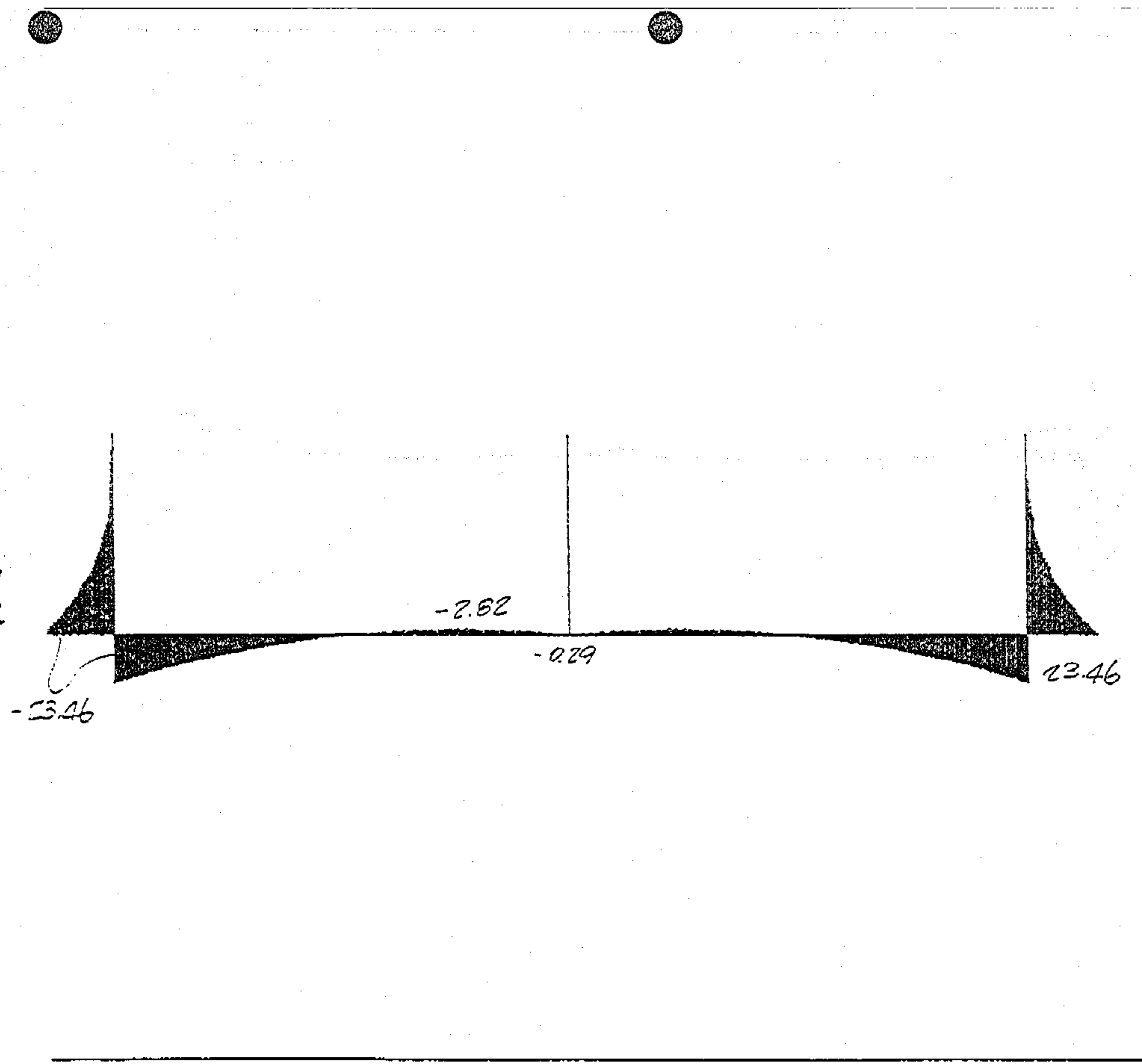
2.2 Foundation Reaction :

B = 16.99 m

W + Ww = 103.18 ton

qs2 = (W + Ww) / B = 6.07 t/m

qs2 - qs1 = 2.90 t/m



DATOS

FRAME

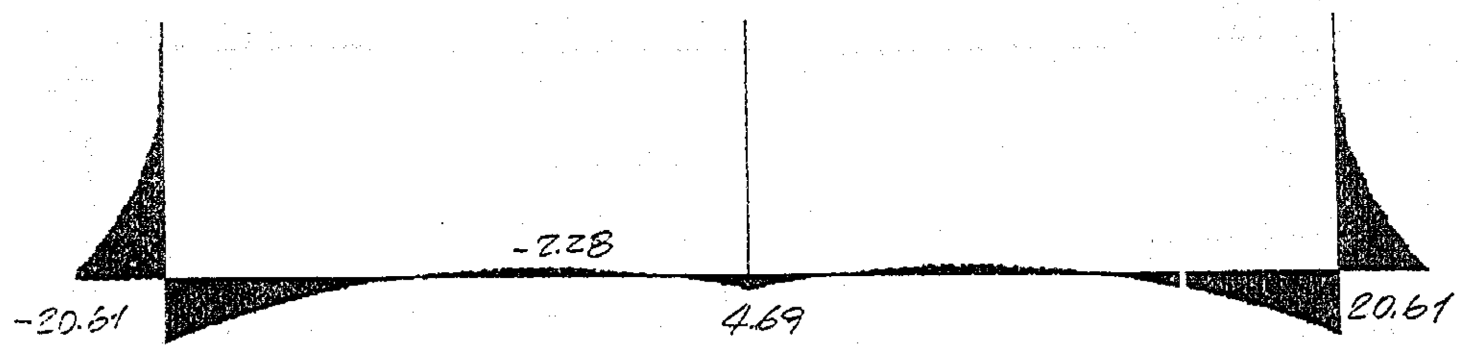
OUTPUT M33

LOAD 1

MIN < 1>
-.2346E+02
AT .00
MAX < 8>
.2346E+02
AT .00

SAP90

3-31

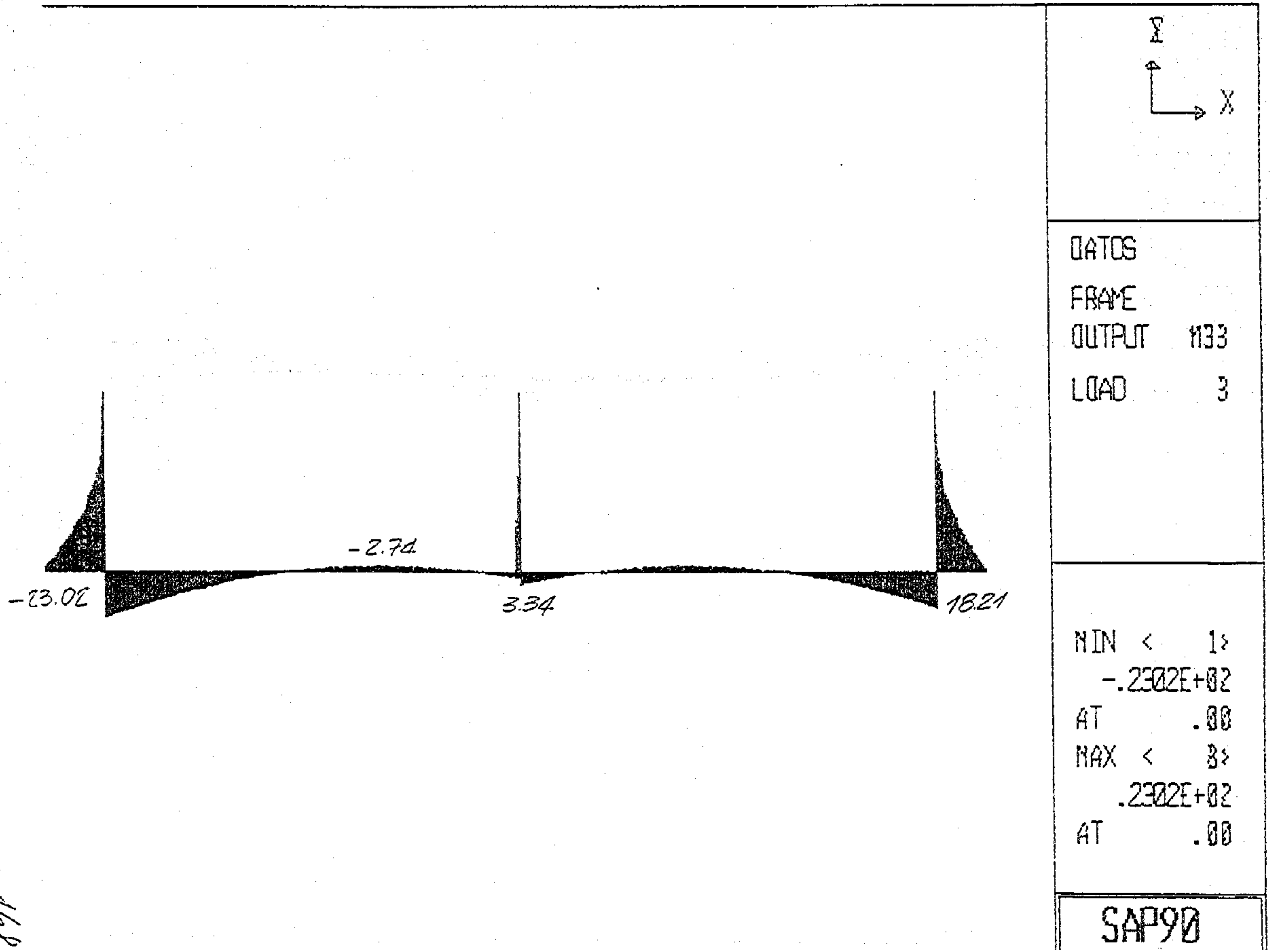


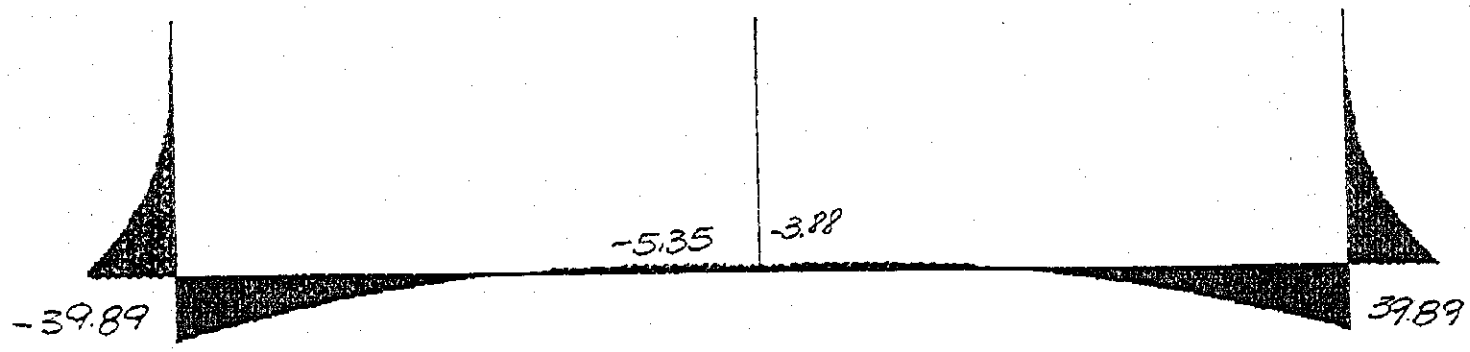
DATOS

FRAME
OUTPUT M33
LOAD 2

MIN < 1>
-.2061E+02
AT .00
MAX < 8>
.2061E+02
AT .00

SAP90


Y
X
-23.02
-2.74
3.34
18.21
DATOS
FRAME
OUTPUT M33
LOAD 3
MIN < 1>
-.2302E+02
AT .00
MAX < 3>
.2302E+02
AT .00
SAP90

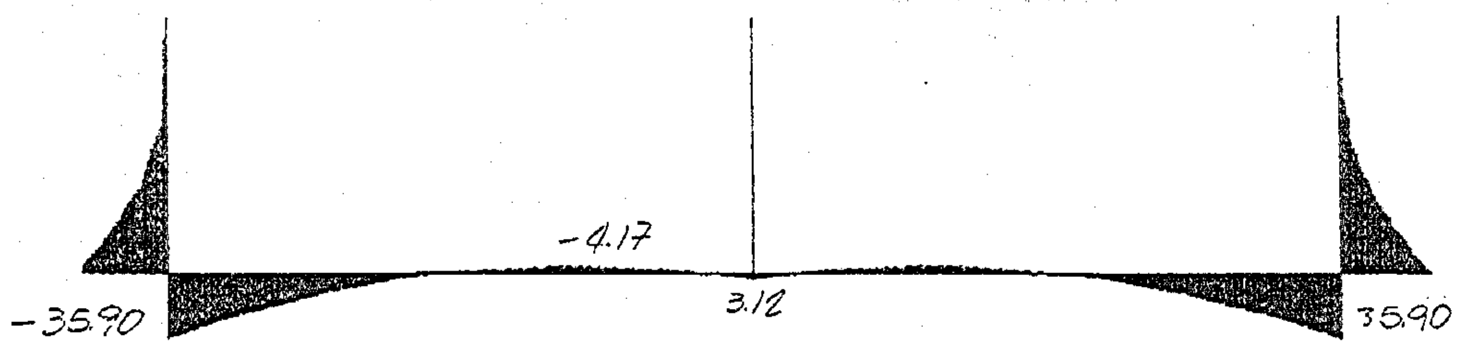


DATOS
FRAME
OUTPUT M33
LOAD 1

MIN < 1>
-.3989E+02
AT .00
MAX < 3>
.3989E+02
AT .00

SAP90

3-34



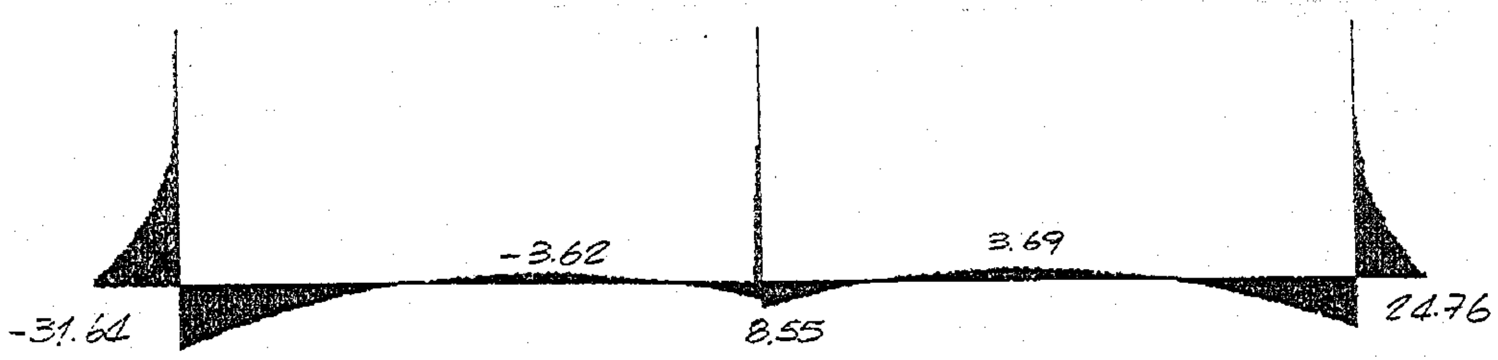
Y
X

DATOS

FRAME
OUTPUT M33
LOAD 5

MIN < 1>
-.3590E+02
AT .00
MAX < 8>
.3590E+02
AT .00

SAP90



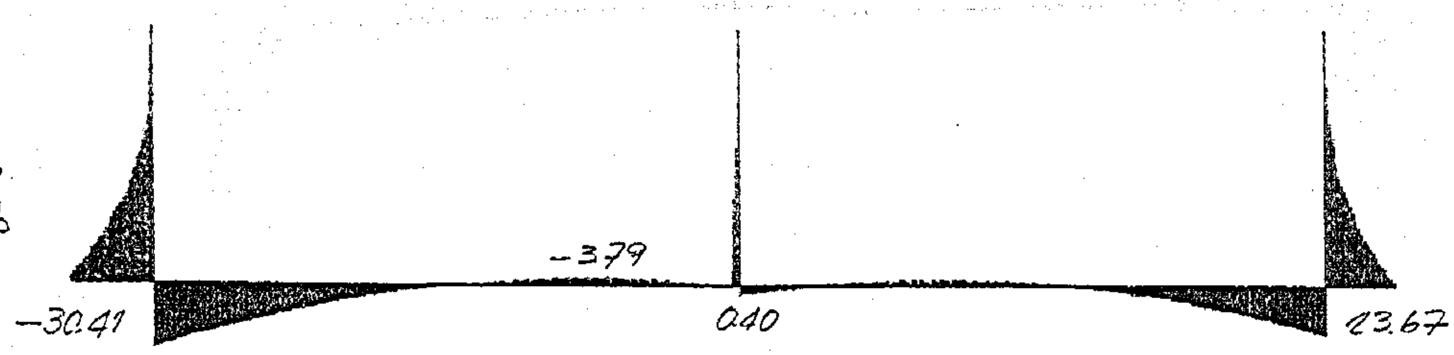
DATOS
FRAME
OUTPUT M33
LOAD 7

MIN < 1>
-.3164E+02
AT .00
MAX < 8>
.3164E+02
AT .00

SAP90

3-36

172

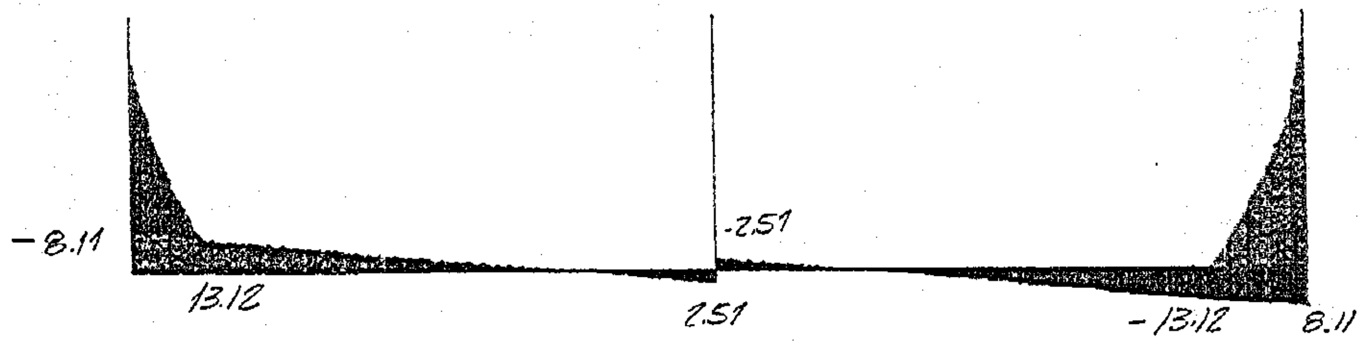


DATOS
FRAME
OUTPUT M33
LOAD 6

MIN < 1>
-.3041E+02
AT .00
MAX < 3>
.3041E+02
AT .00

SAP90

3-37



X
Y

DATOS

FRAME

OUTPUT V22

LOAD 1

MIN < 35>

-.1312E+02

AT .00

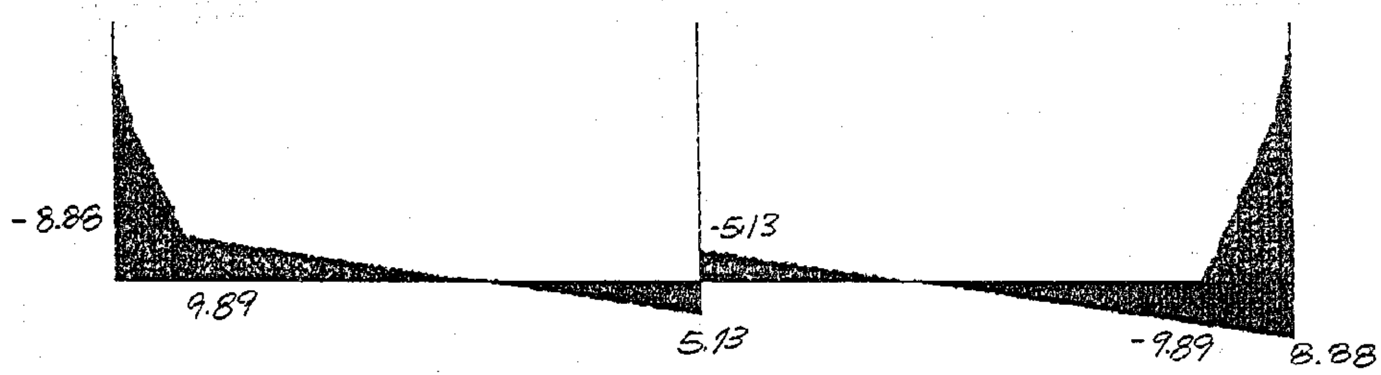
MAX < 1>

.1312E+02

AT .00

SAP90

3.38



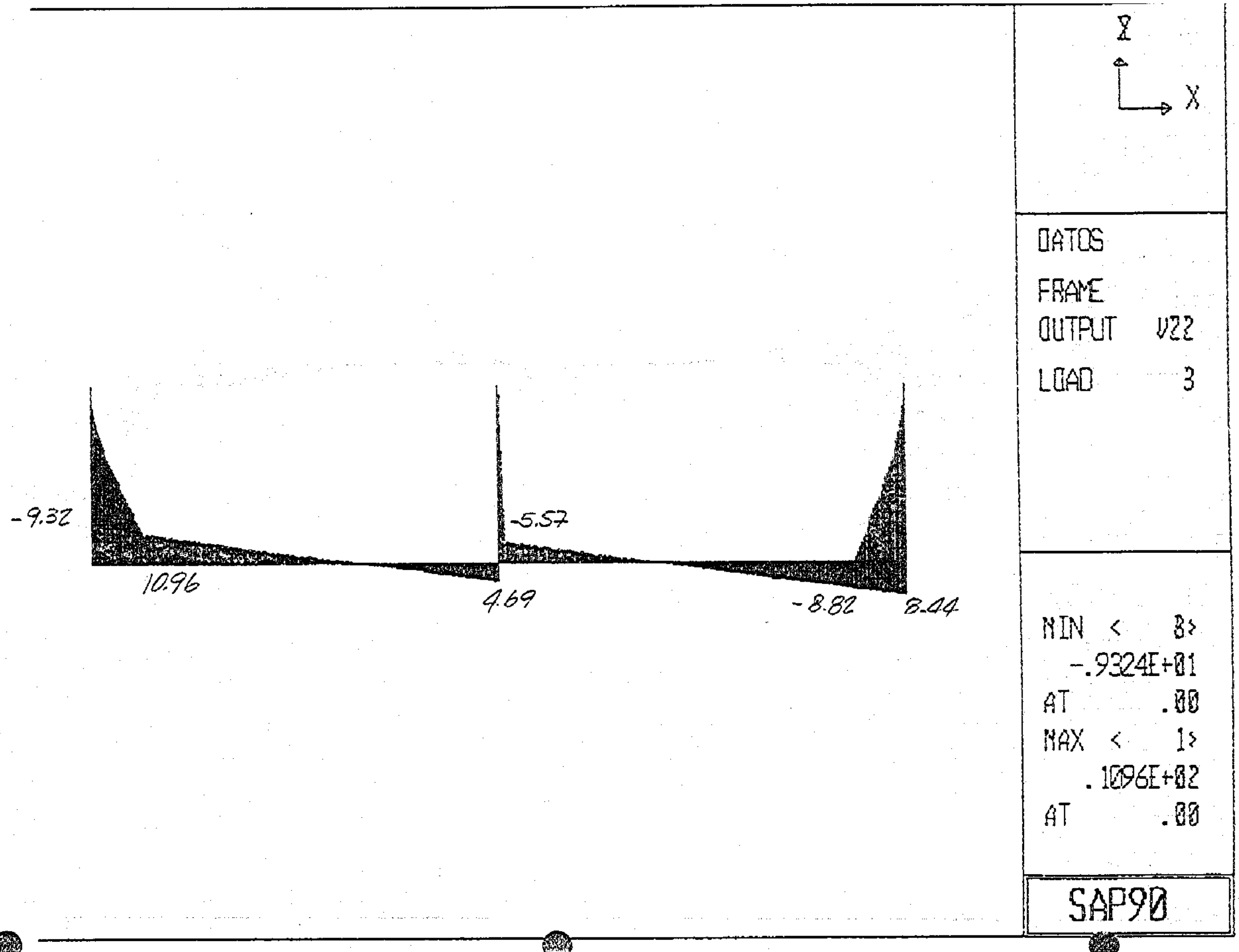
Z

X

DATOS
FRAME
OUTPUT V22
LOAD 2

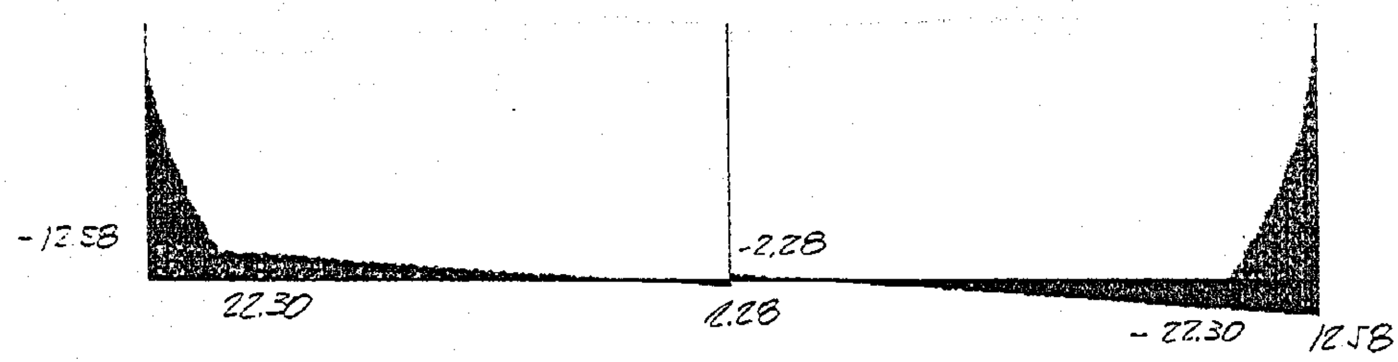
MIN < 35>
-.9891E+01
AT .00
MAX < 1>
.9891E+01
AT .00

SAP90


Y
X
DATOS
FRAME
OUTPUT V22
LOAD 3
-9.32
10.96
-5.57
4.69
-8.82
8.44
MIN < 8>
-.9324E+01
AT .00
MAX < 1>
.1096E+02
AT .00
SAP90

3-40

175



DATOS

FRAME

OUTPUT V22

LOAD 4

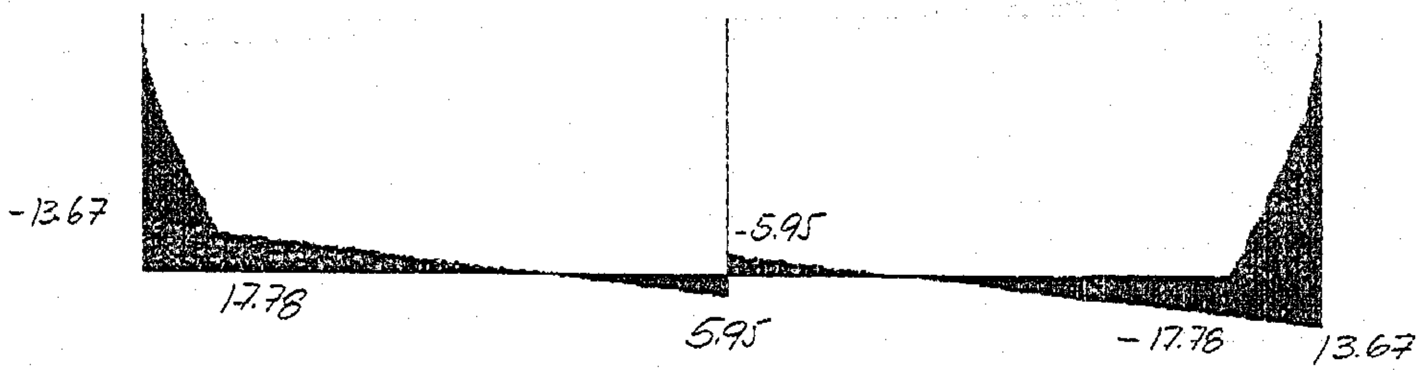
MIN < 35>
-.2230E+02
AT .00
MAX < 1>
.2230E+02
AT .00

SAP90

3-41

176

3-12



Z

X

DATOS

FRAME

OUTPUT V22

LOAD 5

MIN < 35>

-.1778E+02

AT .00

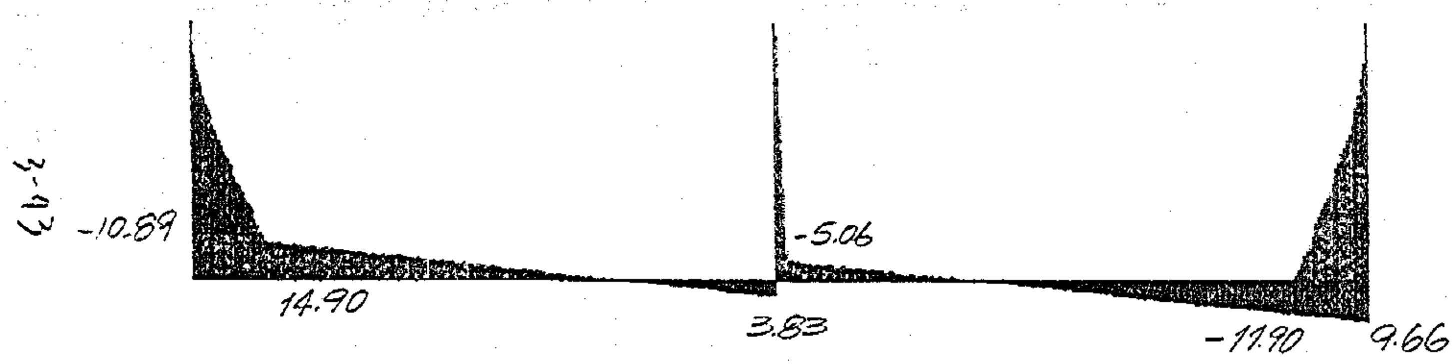
MAX < 1>

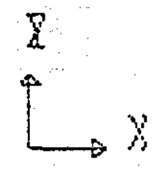
.1778E+02

AT .00

SAP90

177





DATOS

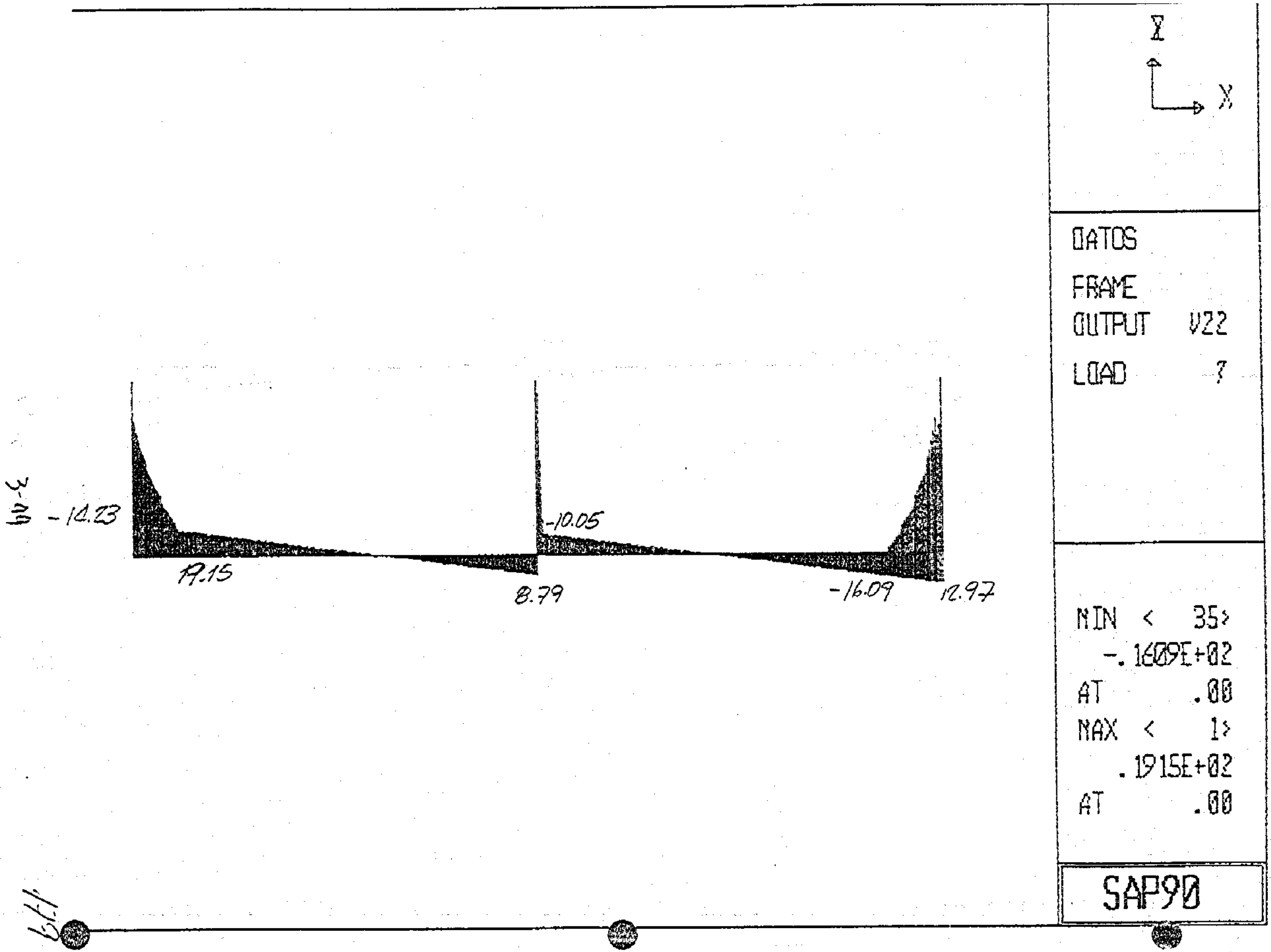
FRAME

OUTPUT V22

LOAD 6

MIN < 35>
-.1190E+02
AT .00
MAX < 1>
.1490E+02
AT .00

SAP90



Y
X
DATOS
FRAME
OUTPUT V22
LOAD 7
MIN < 35>
-.1609E+02
AT .00
MAX < 1>
.1915E+02
AT .00
SAP90
3-19
-14.23
19.15
-10.05
8.79
-16.09
12.97

179

HEAD TANK TYPE II

GIVEN :

f'c = 210 Kg/cm²

fy = 4200 "

SHEAR STRENGTH DESIGN

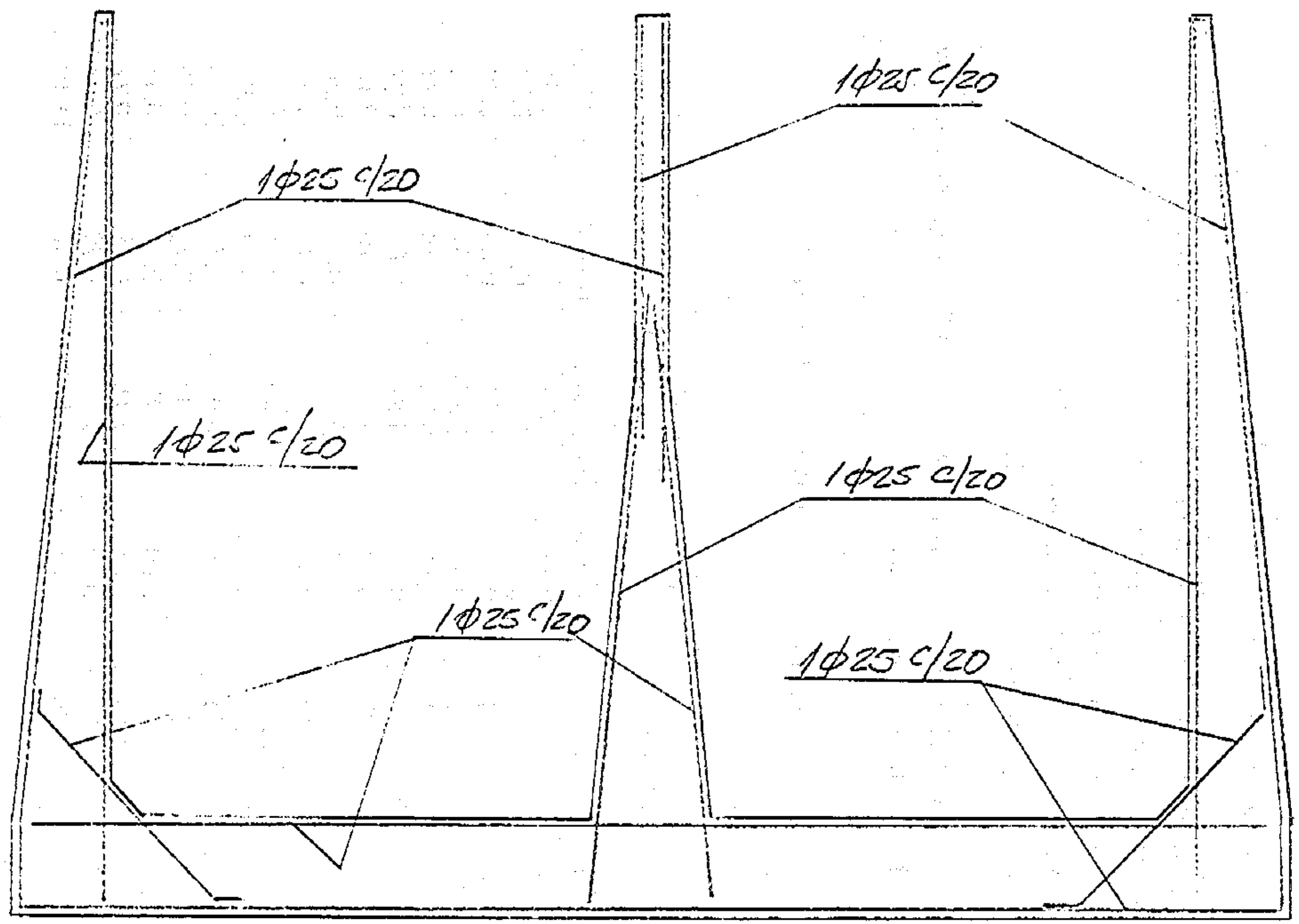
ID ELEM	Vu (ton)	bw (cm)	dn (cm)	h (cm)	d(adopt) (cm)	øVc (ton)
1	22.30	100	34.16	79.0	71.50	46.68
2	22.22	100	34.03	73.0	65.50	42.76
3	14.79	100	22.65	63.0	55.50	36.23
4	8.80	100	13.48	53.0	45.50	29.70
5	4.35	100	6.66	43.0	35.50	23.18
6	1.43	100	2.18	33.0	25.50	16.65
7	0.03	100	0.05	30.0	22.50	14.69
9	14.20	100	21.75	80.0	72.50	47.33
19	1.62	100	2.48	88.0	80.50	52.55
21	0.86	100	1.32	50.0	42.50	27.75

FLEXURE STRENGTH DESIGN

GIVEN :

f'c = 210 Kg/cm²
fy = 4200 "
r = 7.5 cm
ρmax = 0.75ρb = 1.61 (%)
ρs = 0.90 (%)

ID ELEM	Mu (t-m)	b (cm)	h (cm)	d (cm)	ρ (%)	As (cm²)	As(min) (cm²)	As(temp) (cm²)	As(adopt) (cm²)	As(adopt) (varillas)
1	39.89	100	79.0	71.5	0.21	15.14	23.83	9.88	24.54	1 ø 25 @ 20 cm.
2	39.67	100	73.0	65.5	0.25	16.51	21.83	9.13	24.54	1 ø 25 @ 20 cm.
3	21.47	100	63.0	55.5	0.19	10.47	18.50	7.88	24.54	1 ø 25 @ 20 cm.
4	9.81	100	53.0	45.5	0.13	5.79	15.17	6.63	15.71	1 ø 20 @ 20 cm.
5	3.36	100	43.0	35.5	0.07	2.52	11.83	5.38	15.71	1 ø 20 @ 20 cm.
9	39.76	100	80.0	72.5	0.21	14.87	24.17	10.00	24.54	1 ø 25 @ 20 cm.
10	28.18	100	80.0	72.5	0.14	10.46	24.17	10.00	24.54	1 ø 25 @ 20 cm.
11	18.22	100	80.0	72.5	0.09	6.72	24.17	10.00	24.54	1 ø 25 @ 20 cm.
12	10.02	100	80.0	72.5	0.05	3.68	24.17	10.00	24.54	1 ø 25 @ 20 cm.
13	5.72	100	80.0	72.5	0.03	2.09	24.17	10.00	24.54	1 ø 25 @ 20 cm.
14	5.64	100	80.0	72.5	0.03	2.06	24.17	10.00	24.54	1 ø 25 @ 20 cm.
15	5.35	100	80.0	72.5	0.03	1.96	24.17	10.00	24.54	1 ø 25 @ 20 cm.
16	8.45	100	80.0	72.5	0.04	3.10	24.17	10.00	24.54	1 ø 25 @ 20 cm.
18	3.76	100	88.0	80.5	0.02	1.24	26.83	11.00	30.79	1 ø 28 @ 20 cm.
19	3.83	100	88.0	80.5	0.02	1.26	26.83	11.00	30.79	1 ø 28 @ 20 cm.
20	2.43	100	76.0	68.5	0.01	0.94	22.83	9.50	24.54	1 ø 25 @ 20 cm.
25	12.39	100	80.0	72.5	0.06	4.55	24.17	10.00	24.54	1 ø 25 @ 20 cm.
28	5.58	100	80.0	72.5	0.03	2.04	24.17	10.00	24.54	1 ø 25 @ 20 cm.
29	6.26	100	80.0	72.5	0.03	2.29	24.17	10.00	24.54	1 ø 25 @ 20 cm.
30	10.02	100	80.0	72.5	0.05	3.68	24.17	10.00	24.54	1 ø 25 @ 20 cm.
31	13.18	100	80.0	72.5	0.07	4.85	24.17	10.00	24.54	1 ø 25 @ 20 cm.
32	28.18	100	80.0	72.5	0.14	10.46	24.17	10.00	24.54	1 ø 25 @ 20 cm.
33	39.76	100	80.0	72.5	0.21	14.87	24.17	10.00	24.54	1 ø 25 @ 20 cm.
34	39.89	100	80.0	72.5	0.21	14.92	24.17	10.00	24.54	1 ø 25 @ 20 cm.
36	39.67	100	73.0	65.5	0.25	16.51	21.83	9.13	24.54	1 ø 25 @ 20 cm.
37	21.47	100	63.0	55.5	0.19	10.47	18.50	7.88	24.54	1 ø 25 @ 20 cm.
38	9.81	100	53.0	45.5	0.13	5.79	15.17	6.63	24.54	1 ø 25 @ 20 cm.
39	3.36	100	43.0	35.5	0.07	2.52	11.83	5.38	24.54	1 ø 25 @ 20 cm.



REINFORCEMENT HEAD TANK TYPE II

Determination of Water Depth on Overflow Weir of Severino Head Tank

Length of the overflow weir was determined based on the required width of the settling basin of the head tank. The overflow depth on the weir is calculated in the condition of complete overflow by the following formula.

$$L = Q / C \cdot Ho^{1.5} + 2 \cdot Ka \cdot Ho$$

where, L : length of overflow weir (= 7.85 m for each weir)
Q : overflow discharge (m^3/sec)
Ho : overflow depth (m)
Ka : coefficient of contraction (= 0.02)
C : coefficient of overflow (= 2.0)

The following table shows various overflow depth on the weir versus overflow discharges calculated by the above formula.

Table Overflow Depth on Weir in Head Tank

$L = Q/CHo^{1.5} + 2KaHo$

C= 2.00 Ka= 0.02
L= 7.85 m for each weir

Ho (m)	Q (m^3/sec)	Ho (m)	Q (m^3/sec)	Ho (m)	Q (m^3/sec)
0.30	2.58	0.47	5.05	0.64	8.01
0.31	2.71	0.48	5.21	0.65	8.20
0.32	2.84	0.49	5.37	0.66	8.39
0.33	2.97	0.50	5.54	0.67	8.58
0.34	3.11	0.51	5.70	0.68	8.77
0.35	3.25	0.52	5.87	0.69	8.97
0.36	3.38	0.53	6.04	0.70	9.16
0.37	3.53	0.54	6.21	0.71	9.36
0.38	3.67	0.55	6.39	0.72	9.56
0.39	3.82	0.56	6.56	0.73	9.76
0.40	3.96	0.57	6.74	0.74	9.96
0.41	4.11	0.58	6.91	0.75	10.16
0.42	4.26	0.59	7.09	0.76	10.36
0.43	4.42	0.60	7.27	0.77	10.57
0.44	4.57	0.61	7.46	0.78	10.77
0.45	4.73	0.62	7.64	0.79	10.98
0.46	4.89	0.63	7.83	0.80	11.19

4. Open Channel

4. Open Channel

Contents

4.1 Hydraulic Calculation

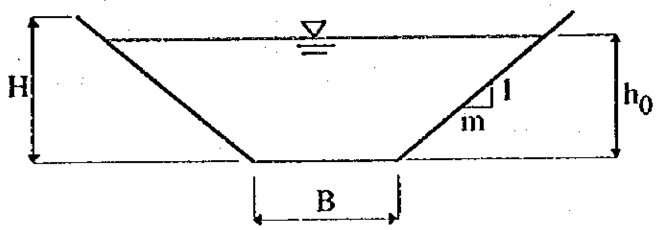
1. Section of open channel

Decision of open channel section adopt the Manning formula as follows.

$$\frac{Q \cdot n}{\sqrt{S_0}} = A \cdot R^{2/3}$$

where,

- Q : discharge = 16 m^3/s
- n : coefficient of roughness = 0.015
- S_0 : longitudinal slope = 1/3,000
- A : sectional area (as shown below)
- R : hydraulic radius



- B: bottom width = 1.6 m
- m : Side slope ratio = 1.2
- h_0 : Water depth

$$A = (B + mh_0)\, h_0$$

$$R = \frac{(B + mh_0)h_0}{B + 2\sqrt{1+m^2}\,h_0}$$

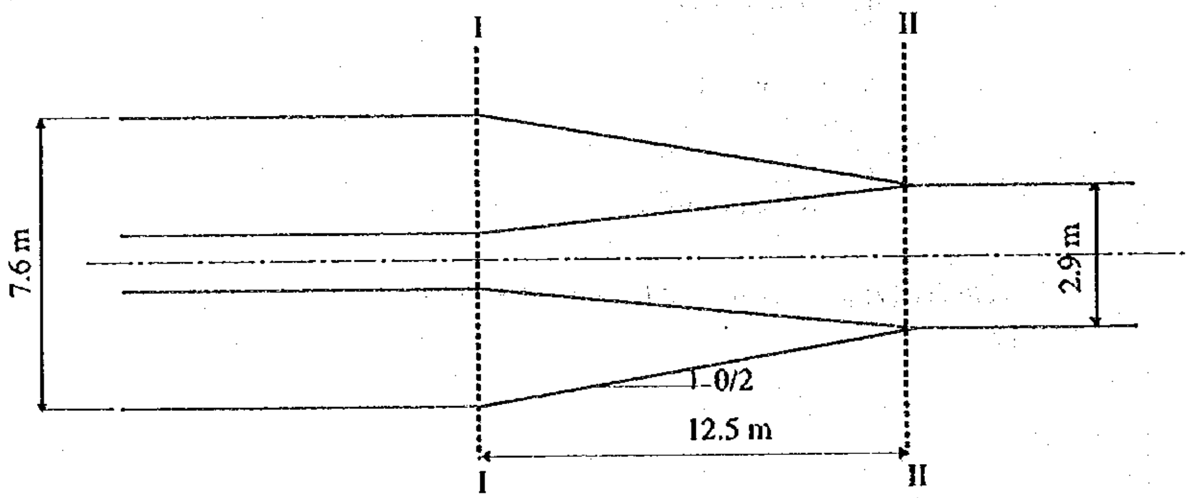
Velocity (v) and discharge(Q) for each water depth are shown in the following table.

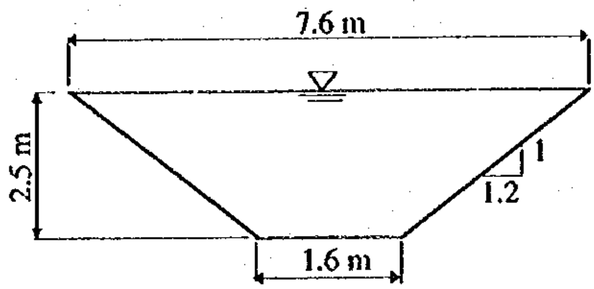
h_o (m)	h_o/H (%)	v (m/s)	Q (m^3/s)
0.28	10	0.442	0.240
0.56	20	0.638	0.812
0.84	30	0.786	1.721
1.12	40	0.910	3.001
1.40	50	1.021	4.690
1.68	60	1.124	6.826
1.96	70	1.220	9.446
2.24	80	1.310	12.587
2.50	89.20	1.391	16.000
2.52	90	1.397	16.283
2.80	100	1.481	20.568

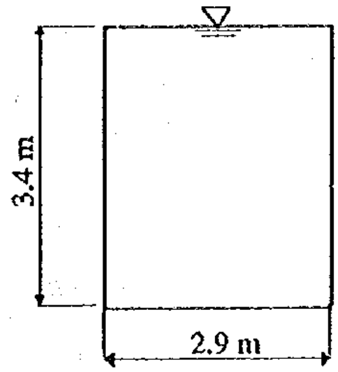
2. Head loss of open Transition

1) Inlet open transition

Head loss calculation of inlet open transition applies for the case of gradual contraction.







$A_1 = 11.5\ m^2$
$V_1 = 1.391\ m/s$

$A_2 = 9.86\ m^2$
$V_2 = 1.623\ m/s$

$$\tan\frac{\theta}{2} = \frac{\frac{7.6-2.9}{2}}{12.5} = 0.188$$

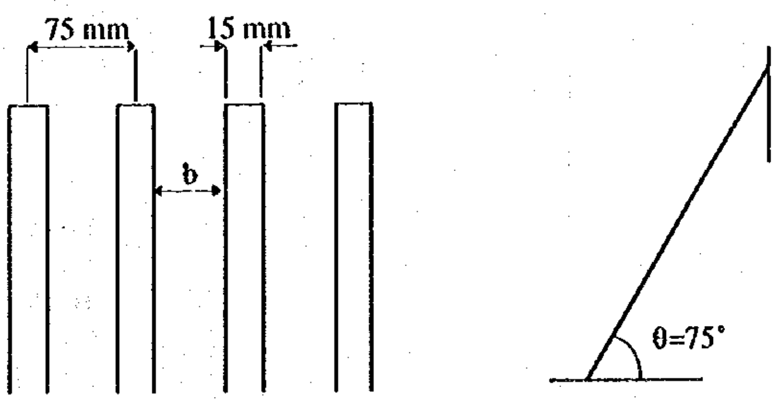
$$\therefore\ \theta = 21.294^\circ$$

$$\frac{A_2}{A_1} = \frac{11.5}{9.86} = 0.857$$

$$\therefore\ f_c = 0.0025$$

$$\Delta h_c = f_c \cdot \frac{V_2^2}{2g} + \left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right)$$
$$= 0.0025 \times \frac{1.623^2}{2\times 9.8} + \left(\frac{1.623^2}{2\times 9.8} - \frac{1.391^2}{2\times 9.8}\right)$$
$$= 0.036\ (m)$$

- Trash rack



$b = 75 - 15 = 60$ (mm)

$$\frac{t}{b} = \frac{15}{60} = 0.25$$

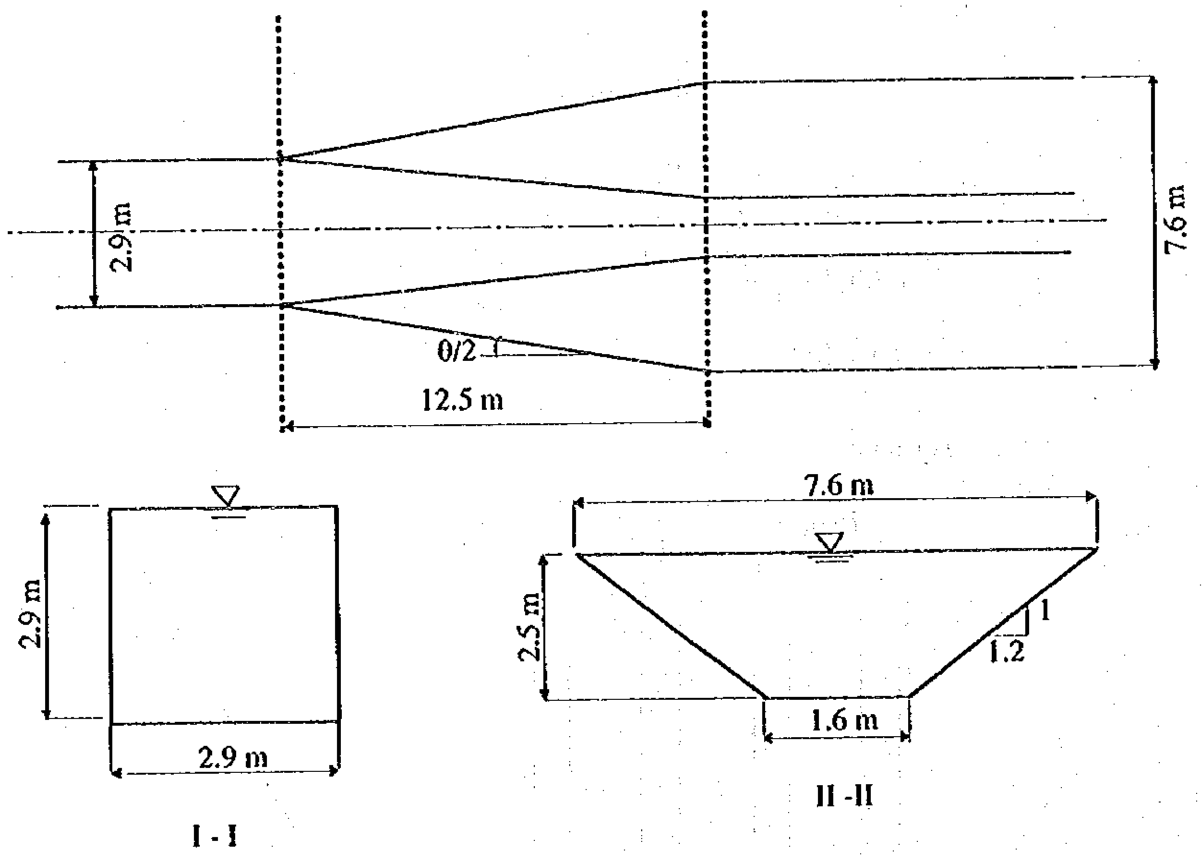
$b = 2.42$, $\theta = 75°$, $V = 1.623$ m/s

$$fr = b\sin\theta \cdot \left(\frac{t}{b}\right)^{4/3}$$
$$= 2.42 \times \sin 75° \times 0.25^{4/3}$$
$$= 0.368$$

$$\therefore \Delta h_r = f_r \frac{V^2}{2g} = 0.368 \times \frac{1.623^2}{2 \times 9.8}$$
$$= 0.049 \text{ (m)}$$

2) Outlet open transition

Head loss calculation of outlet open transition applies for the case of gradual enlargement.



$A_1 = 11.5\ m^2$ $A_2 = 9.86\ m^2$

$V_1 = 1.391$ m/s $V_2 = 1.623$ m/s

$$h_{ge} = f_{gc}\frac{(V_1 - V_2)^2}{2g} = f_{gc}\left\{1-\left(\frac{A_1}{A_2}\right)\right\}^2\frac{V_1^2}{2g}$$

$$= f_{gc}\, f_{se}\frac{V_1^2}{2g}$$

$\theta = 21.294°$ $\therefore f_{ge}=0.4$

$$\sqrt{\frac{A_1}{A_2}} = \sqrt{\frac{8.41}{11.5}} = 0.855$$

$\therefore f_{se} = 0.081$

$$\therefore\ h_{ge} = 0.0805 \times 0.4 \times \frac{1.902^2}{2\times 9.8} = 0.006\ (m)$$

3. Head loss of siphon

1) Siphon No. 1

- Basic condition

Selection	:	2.9 m x 2.9 m (Rectangular type)
Hydraulic radius	:	$R = 0.967$
Length	:	$L = 40.105$ (m)
Velocity	:	$v = 1.902$ (m/s)
Radius of curvature	:	$\rho = 15.0$ (m)

- Coefficient of entrance loss : f_e

$A_1 = 11.5\ (m^2)$

$a = 8.41\ (m^2)$

$$\sqrt{\frac{a}{A_1}} = \sqrt{\frac{8.41}{11.5}} = 0.855 \quad \text{(sudden contract)}$$

$\therefore f_e = 0.12$ (same as all siphon)

- Coefficient of bending loss : Σf_b

$\rho = 15m$

$\frac{\rho}{D} = \frac{15}{2.9} = 5.172$ ∴ $f_{b1} = 0.08$ (same as all siphon)

$\theta_1 = 25.565°$ ∴ $f_{b21} = 0.45$

$\theta_2 = 25.565°$ ∴ $f_{b22} = 0.45$

$$\Sigma f_b = 0.08 \times 0.45 \times 2$$
$$= 0.072$$

- Coefficient of friction lost : $f'\frac{L}{R}$

$$f'\frac{L}{R} = \frac{2gn^2}{R^{1/3}} \cdot \frac{L}{R} = \frac{2 \times 9.8 \times 0.015^2}{0.967^{1/3}} \times \frac{49.105}{0.967}$$
$$= 0.226$$

- Coefficient of outlet loss : f_o

$A_1 = 11.5\ (m^2)$

$a = 8.41\ (m^2)$

$$\sqrt{\frac{a}{A_2}} = 0.855$$

∴ $f_o = 0.081$ (same as all siphon)

- Total head loss : Δh_s

$$\Sigma f = f_e + \Sigma f_b + f'\frac{L}{R} + f_o = 0.12 + 0.072 + 0.226 + 0.081$$
$$= 0.499$$

$$\Delta h_s = \Sigma f \cdot \frac{v^2}{2g} = 0.499 \times \frac{1.902^2}{2 \times 9.8}$$
$$= 0.092\ (m)$$

2) Siphon No. 2

- Basic condition ($_r$: rectangular type, $_c$: circular type)

Section : 2.9 x 2.9 m (rectangular type)
3.2 m diameter (circular type)

Hydraulic radius	:	$R = 0.967$ (rectangular type)
Velocity	:	$V_r = 1.902$ m/s $V_c = 1.989$ m/s
Length	:	$L_r = 141.028$ m $L_c = 74.663$ m
Radius of curvature	:	$\rho = 15.0$ m

- Coefficient of entrance loss : f_e

  $f_e = 0.12$

- Coefficient of bending loss : Σf_{br}, Σf_{bc}

  $f_{b1} = 0.08$

  $\theta_1 = 14.532°$ $\therefore f_{b21} = 0.28$

  $\theta_2 = 29.899°$ $\therefore f_{b22} = 0.50$

  $\theta_3 = 29.745°$ $\therefore f_{b23} = 0.50$

  $\theta_4 = 14.507°$ $\therefore f_{b24} = 0.28$

  $\Sigma f_{br} = 0.08 \times 0.28 = 0.022$ (rectangular part)

  $\Sigma f_{bc} = 0.08 \times (0.50 + 0.50 + 0.28) = 0.102$ (circular part)

- Coefficient of friction lose : $f' \dfrac{L_r}{R}$, $f \dfrac{L_c}{D}$

$$f' \frac{L_r}{R} = \frac{2gn^2}{R^{1/3}} \cdot \frac{L_r}{R} = \frac{2 \times 0.98 \times 0.015^2}{0.967^{1/3}} \cdot \frac{141.028}{0.967}$$
$$= 0.650$$

$$f \frac{L_c}{D} = \frac{124.5 n^2 L_c}{D^{4/3}} = \frac{124.5 \times 0.015^2 \times 74.663}{3.2^{4/3}}$$
$$= 0.444$$

- Coefficient of outlet loss : f_o

$f_o = 0.081$

- Total head loss : Δh_s

$$\Sigma f_r = f_e + \Sigma f_b + f \frac{L_r}{R} + f_o = 0.12 + 0.022 + 0.650 + 0.081 = 0.873$$

$$\Sigma f_c = \Sigma f_{bc} + f \frac{L_c}{D} = 0.12 + 0.444 = 0.546$$

$$\Delta h_{sr} = \Sigma f_r \times \frac{V_r^2}{2g} = 0.863 \times \frac{1.902^2}{2 \times 9.8} = 0.161$$

$$\Delta h_{sr} = \Sigma f_c \times \frac{V_c^2}{2g} = 0.546 \times \frac{1.989^2}{2 \times 9.8} = 0.110$$

$$\Delta h_s = \Delta h_{sr} + \Delta h_{sc} = 0.161 + 0.110$$
$$= 0.271 \text{ (m)}$$

3) Siphon No. 3

- Basic condition ($_r$: rectangular type, $_c$: circular type)

Section	:	2.9 x 2.9 m (rectangular type) 3.2 m diameter (circular type)
Hydraulic radius	:	R = 0.967 (rectangular type)
Velocity	:	$V_r = 1.902$ m/s $V_c = 1.989$ m/s
Length	:	$L_r = 129.874$ m $L_c = 187.960$ m
Radius of curvature	:	$\rho = 15.0$ m

- Coefficient of entrance loss : f_e

$f_e = 0.12$

- Coefficient of bending loss : Σf_{br}, Σf_{bc}

$f_{b1} = 0.08$

$\theta_1 = 13.859°$ $\therefore f_{b21} = 0.27$

$\theta_2 = 24.969°$ $\therefore f_{b22} = 0.45$

$\theta_3 = 29.953°$ $\therefore f_{b23} = 0.50$

$\theta_4 = 16.659°$ $\therefore f_{b24} = 0.30$

$\Sigma f_{br} = 0$ (rectangular part)

$\Sigma f_{bc} = 0.08 \times (0.27 + 0.45 + 0.50 + 0.30) = 0.122$ (circular part)

- Coefficient of friction lose : $f' \frac{L_r}{R}, f \frac{L_c}{D}$

$$f' \frac{L_r}{R} = \frac{2gn^2}{R^{1/3}} \cdot \frac{L_r}{R} = \frac{2 \times 0.98 \times 0.015^2}{0.967^{1/3}} \cdot \frac{129.874}{0.967}$$
$$= 0.599$$

$$f \frac{L_c}{D} = \frac{124.5 n^2 L_c}{D^{4/3}} = \frac{124.5 \times 0.015^2 \times 187.960}{3.2^{4/3}}$$
$$= 1.117$$

- Coefficient of outlet loss : f_o

$f_o = 0.081$

- Total head loss : Δh_s

$$\Sigma f_r = f_e + \Sigma f_b + f' \frac{L_r}{R} + f_o = 0.12 + 0.599 + 0.081 = 0.800$$
$$\Sigma f_c = \Sigma f_{bc} + f \frac{L_c}{D} = 0.122 + 1.117 = 1.239$$

$$\Delta h_{sr} = \Sigma f_r \times \frac{V_r^2}{2g} = 0.800 \times \frac{1.902^2}{2 \times 9.8} = 0.148$$
$$\Delta h_{sr} = \Sigma f_c \times \frac{V_c^2}{2g} = 1.239 \times \frac{1.989^2}{2 \times 9.8} = 0.250$$

$$\Delta h_s = \Delta h_{sr} + \Delta h_{sc} = 0.148 + 0.250$$
$$= 0.348 \text{ (m)}$$

4) Siphon No. 4

- Basic condition

Selection	:	2.9 m x 2.9 m (Rectangular type)
Hydraulic radius	:	$R = 0.967$
Length	:	$L = 50.6$ (m)
Velocity	:	$v = 1.902$ (m/s)
Radius of curvature	:	$\rho = 15.0$ (m)

- Coefficient of entrance loss : f_e

$A_1 = 11.5\ (m^2)$

$a = 8.41\ (m^2)$

$$\sqrt{\frac{a}{A_1}} = \sqrt{\frac{8.41}{11.5}} = 0.855 \quad \text{(sudden contract)}$$

$\therefore f_e = 0.12$ (same as all siphon)

- Coefficient of bending loss : Σf_b

$\rho = 15m$

$$\frac{\rho}{D} \cdot \frac{15}{2.9} = 5.172 \qquad \therefore fb_1 = 0.08 \text{ (same as all siphon)}$$

$\theta_1 = 12.881^\circ \qquad \therefore f_{b21} = 0.26$

$\theta_2 = 15.201^\circ \qquad \therefore f_{b22} = 0.29$

$$\Sigma f_b = 0.08 \times (0.26 + 0.29) = 0.044$$

- Coefficient of friction lost : $f' \frac{L}{R}$

$$f' \frac{L}{R} = \frac{2gn^2}{R^{1/3}} \cdot \frac{L}{R} = \frac{2 \times 9.8 \times 0.015^2}{0.967^{1/3}} \times \frac{50.60}{0.967}$$
$$= 0.233$$

- Coefficient of outlet loss : f_o

$f_o = 0.081$

- Total head loss : Δh_s

$$\Sigma f = f_e + \Sigma f_b + f' \frac{L}{R} + f_o = 0.12 + 0.044 + 0.233 + 0.081$$
$$= 0.478$$

$$\Delta h_s = \Sigma f \cdot \frac{v^2}{2g} = 0.478 \times \frac{1.902^2}{2 \times 9.8}$$
$$= 0.088\ (m)$$

5) Siphon No. 5

- Basic condition

Selection	:	2.9 m x 2.9 m (Rectangular type)
Hydraulic radius	:	$R = 0.967$
Length	:	$L = 153.089$ (m)
Velocity	:	$v = 1.902$ (m/s)
Radius of curvature	:	$\rho = 15.0$ (m)

- Coefficient of entrance loss : f_e

$f_e = 0.12$

- Coefficient of bending loss : Σf_b

$\rho = 15m$

$\frac{\rho}{D} \cdot \frac{15}{2.9} = 5.172 \quad \therefore fb_1 = 0.08$ (same as all siphon)

$\theta_1 = 18.736° \quad \therefore f_{b21} = 0.34$

$\theta_2 = 12.868° \quad \therefore f_{b22} = 0.25$

$$\Sigma f_b = 0.08 \times (0.34 + 0.25) = 0.047$$

- Coefficient of friction lost : $f' \frac{L}{R}$

$$f' \frac{L}{R} = \frac{2gn^2}{R^{1/3}} \cdot \frac{L}{R} = \frac{2 \times 9.8 \times 0.015^2}{0.967^{1/3}} \times \frac{153.809}{0.967}$$
$$= 0.706$$

- Coefficient of outlet loss : f_o

$f_o = 0.081$

- Total head loss : Δh_s

$$\Sigma f = f_e + \Sigma f_b + f' \frac{L}{R} + f_o = 0.12 + 0.047 + 0.706 + 0.081$$
$$= 0.954$$

$$\Delta h_s = \Sigma f \cdot \frac{v^2}{2g} = 0.954 \times \frac{1.902^2}{2 \times 9.8}$$
$$= 0.176 \text{ (m)}$$

4. Discharge of crossing streams/rivers

Crossing stream discharge along the Severino open channel are calculated depend on design criteria. There are thirty-three parts which include twenty-eight streams for drainage culverts and five for siphons. Results of calculation are shown in the following table.

No.	Area	Max Contour (m)	Min. Contour (m)	Difference height (m)	Length of Stream (m)	Time of concentration (min)	Qmax Tr=25 (m3/s)	Drainage Culvert & Siphon No.
1	0.049	184	105	79	250	2.13	1.33	Siphon No.1
2	0.435	355	90	265	875	5.69	11.79	Siphon No.2
3	0.020	145	103	42	230	2.47	0.55	CD-1
4	0.013	145	103	42	250	2.72	0.36	CD-2
5	0.126	200	102	98	600	5.40	3.42	CD-3
6	0.046	175	105	70	350	3.30	1.25	CD-4
7	0.014	145	112	33	170	1.91	0.38	CD-5
8	0.064	300	112	188	750	5.43	1.73	CD-6
9	0.017	160	107	53	300	3.07	0.47	CD-7
10	0.053	250	102	148	550	4.16	1.42	CD-8
11	0.011	150	103	47	210	2.13	0.30	CD-9
12	0.161	300	110	190	610	4.26	4.36	CD-10
13	0.023	155	113	42	250	2.72	0.63	CD-11
14	0.060	170	112	58	460	4.24	1.63	CD-12
15	0.018	140	113	27	200	2.49	0.49	CD-13
16	0.017	175	115	60	230	2.15	0.47	CD-14
17	6.934	416	70	346	3125	HYMO Model	78.40	Siphon No.3
18	0.009	155	105	50	220	2.19	0.25	CD-15
19	0.028	140	112	28	200	2.46	0.75	CD-16
20	0.037	156	110	46	200	2.03	1.00	CD-17
21	0.010	133	108	25	140	1.70	0.28	CD-18
22	0.150	165	110	55	500	5.46	4.06	CD-19
23	0.072	200	112	88	600	5.63	1.94	CD-20
24	0.024	150	110	40	250	2.77	0.65	CD-21
25	0.090	250	110	140	500	3.81	2.45	CD-22
26	0.010	144	108	36	150	1.60	0.26	CD-23
27	0.220	290	105	185	600	4.23	5.96	Siphon No.4
28	0.015	150	107	43	150	1.49	0.41	CD-24
29	0.236	390	104	286	825	5.16	6.41	CD-25
30	0.094	285	112	173	450	3.11	2.54	CD-26
31	1.071	447	93	354	1750	11.33	29.05	Siphon No.5
32	0.018	155	105	50	220	2.19	0.49	CD-27
33	0.056	220	105	115	300	2.28	1.51	CD-28

5. Side channel spillway at Siphon No. 1

1). Overflow Section

Condition

Discharge in channel upstream of weir	$Q_1 = 20.8\ m^3/sec$
Discharge in channel downstream of weir	$Q_2 = 16.0\ m^3/sec$
Discharge to spill out from weir	$Q_d = Q_1 - Q_2 = 4.8\ m^3/sec$
Weir height	$H_d = 2.5\ m$
Overflow coefficient (for broad-crest weir)	$K = 1.7\ m$
Water depth in channel at Q = 20.8 m^3/sec	$h_1 = 2.814\ m$
Water depth in channel at Q = 16.0 m^3/sec	$h_2 = 2.5\ m$
Overflow depth at downstream end of weir	$H_2 = h_1 - h_2 = 0.314\ m$

Calculation

(INPUT)

H_1 = 0.275 m

H_2 = 0.314 m

Q_1 = 20.800 m^3/sec

Q_2 = 16.000 m^3/sec

$V_2 = Q_2/A_2$ = 1.142 m/sec

A_2 = 14.005 m^2

$V_1 = Q_1/A_1$ = 1.520 m/sec

A_1 = 13.681 m^2

$$Q_d = K \cdot L\{(H_1 + H_2)/2\}^{2/3}$$

L = 17.667 m

= 18.000 m

(Check)

- Friction loss in side weir portion

$$H_f = (I_1 + I_2)/2 \times L$$

$$I_1 = (nV_1)^2 / R_1^{4/3} = 0.000354815$$

$$I_2 = (nV_2)^2 / R_2^{4/3} = 0.000197271$$

n = 0.015

R_1 = 1.332

R_2 = 1.347

h_f = 0.005

- Loss of overflow

$H_b = f_b(V_1^2/2g)$

$f_b = 0.5807Q_r - (0.0788Q_r + 0.003)^{0.5} + 0.0171$

$Q_r = Q_d/Q_1 = 0.2308$

$F_b = 0.0151$

$H_b = 0.0018$

$E_1 = H_1 + H_d + V_1^2/2g = 2.893$

$E_1' = E_2 + H_d + z + H_f + H_b$

$= H_2 + V_2^2/2g + L/3000 + H_f + H_b = 2.893$

$E_1 = E_1'$ (O.K.)

6. Side Channel

Condition

Discharge in channel upstream of weir	Q =	4.8 m^3/sec
Water depth at downstream end	d	
Channel width at downstream end	B	$d/B = 0.5$
Channel gradient of rapid slope portion	i_1 =	1/13.0
Channel gradient of gentle slope portion	i_2	
Slope gradient of overflow side	1 : m	m = 0

$$d^3B^2\left(1+\frac{m}{2}\frac{d}{B}\right)^3 / \left(1+m\cdot\frac{d}{B}\right) = \frac{Q^2}{g}\frac{1}{F_r^2}$$

$d = 0.480 \times (Q/F_r)^{0.4}$

$F_r = 0.44 \sim 0.5$

$d = 1.25 \sim 0.19 = 1.2$ (m)

$B = 2.4$ (m)

$F_r = 0.486$

Length of channel with gentle slope portion

$l = > 4d$

$= 4.8$

$= 5.0$ m

$$i_2 = gn^2Fr^2\left(1+\frac{2d}{B}\right)^{4/3}\frac{1}{d^{1/3}}$$

$= 1/809.70$

$= 1/810$

4.2 Structural Calculation

GENERAL

The structural elements were designed in order to obtain the design strength in all sections at least same as the required strength calculated to withstand the weighted loads and the strength stipulated in the Ecuadorian Construction code (CEC) and American Concrete Institute (ACI).

Furthermore within the analysis and design the structures guarantee an adecuated performance for the service loads.

For the analysis and design of different elements the following structures have been foreseen, box structures for the box culverts, walls and foundation for transition, box of inner and outer rectangular cross section and box of outer rectangular cross section and inner circular cross section.

These patterns have been selected taking into account the stresses to which these structures have to work.

The utilized loads are the service loads and own load; these loads will be analyzed afterward.

LOADS

Before to the load analysis an elements predesign was carried out in order to estimate the real service loads due to own load plus surcharge. The given values in the analysis table for each structure correspond to several conditions and surcharge height, the calculation procedure is detailed on the data preparation for each structure.

For the loads calculation the following input data were used:

Gs	=	1.8 t/m^3 as soil unit weight
Gc	=	2.4 t/m^3 as concrete unit weight
Ka	=	active earth pressure coefficient
Ec	=	2.1×10^5 kg/cm^2 concrete elasticity modulus
Es	=	2.1×10^6 kg/cm^2 steel elasticity modulus

DETAIL OF APPLIED LOADS TO EACH STRUCTURE BOX CULVERTS, SIPHONS, TRANSITIONS

W_1	=	live load of a HS-20-44 truck
W_2	=	vertical load due to earth surcharge plus weight of the structure cover
W_3	=	uplift surcharge in the foundation
W_4	=	foundation reaction
Wt	=	soil pressure
Ww	=	water pressure
Pw	=	inner water pressure
Pw_1	=	inner water pressure due to the water surcharge
Pc	=	Concrete wall weight

With the above mentioned data and load condition previously stated the structural analysis was made by using the software SAP90.

STRUCTURAL ANALYSIS

For the structural analysis the software SAP90 was fully utilized, this software uses the FEM analysis method. The subroutine frame was utilized for the structural analysis.

This software allow us for examine the structures as well as to model them. All loads were included for the analysis purpose. After the software graphical check which allow as to know the free body scheme, sections, total and local axis, loads in each bar and in the different states too, the analysis was carried out and getting as output the graphical and analytical displacement, moments, shear and axial load.

For the following each load combination and for each structural element, solutions were obtained:

ANALYSIS CONDITION

- General

Wu	=	1.4 DL + 1.7 LL + Ep + 1.7 Pw
D_l	=	dead load
L_l	=	live load

Ep = earth pressure

Pw = water pressure

- Box culvert

$Wu = 1.4\ W_2 + 1.7\ W_2 = 1.7\ W_3 + 1.4\ W_4 + 1.7\ W_4 + 1.7\ Wt$

$Wu = 1.4\ W_2 + 1.4\ W_4 + 1.7\ Pw$

- Transition

$Wu = 1.4\ W_4 + 1.7\ Wt$

$Wu = 1.4\ W_4 + 1.7\ Wt + 1.7\ Pw$

$Wu = 1.4\ W_4 + 1.7\ Pw$

- Siphon

$Wu = 1.4\ W_2 + 1.7\ W_2 + 1.7\ W_3 + 1.4\ W_4 + 1.7\ W_4 + 1.7\ Wt$

$Wu = 1.4\ W_2 + 1.7\ W_2 + 1.7\ W_3 + 1.4\ W_4 + 1.7\ W_4 + 1.7\ Wt + Ww$

$Wu = 1.4\ W_2 + 1.4\ W_4 + 1.7\ Pw + 1.7\ Pw_1$

DESIGN

With the final reactions coming from the structural analysis the design was carried out.

With the final reactions (ultimate moments, shear and stresses) the elements were designed using for this purpose the plastic design. The following general criterias were considered:

- Concrete

$Pu \leq Pd$

$Mu \leq Md$

$Vu \leq Vd$

- Materials Characteristics

f'c = 210 kg/cm^2 compressive strength of concrete

fy = 4,200 kg/cm^2 steel yield point

b = 100 cm

d = effective height of the concrete cross section

r = 7.0 cm covering of the concrete cross section

The designs were carried out for all elements due to moment, shearing, moment and axial load.

The form used for design purpose is detailed in each analysis attached herewith.

Diseño Estructural de Canal Abierto
Structural Design of Open Channel

1. Datos/Data

f'c	:	210 kg/cm^2
fy	:	4,200 kg/cm^2
Ø	:	30° (Angulo de friccion interna del suelo/ Internal friction angle)
α	:	39°48'20" (Angulo de inclinacion de talua/ Inclination angle)
m	:	1:1.2
γ	:	2,000 kg/m^3 (Peso unitario del suelo/ Unit weight of sand)
Nf	:	1.00 m (Nivel del agua subterránea/ Groundwater level)

2. Diseño/ Design

1) Factor de cohesion/Cohesion factor

$$K = \frac{\cos^2(\theta+\alpha)}{\cos^3(1+\dfrac{\sin\theta}{\cos\alpha})^2} = \frac{0.119}{0.997} = 0.119$$

2) Empjes/Thrust

$$E = \frac{1}{2}\gamma \times h^2 \times K$$ (Formula para triangulo/Formula for triangle)

$$E = \gamma \times h^2 \times K$$ (Formula para cuadrado/Formula for square)

$E_1 = \frac{1}{2}\gamma \times h^2 \times K = \frac{1}{2} \times 2000 \times 1.8 \times 1.2 \times 0.119 = 463$ kg (Suelo seco/Dry soil, Triangulo/Triangle)

$E_2 = 1.2 \times 1.8 \times 1 \times 1.000 \times 0.119 = 257$ kg (Suelo saturado/Saturated soil, Rectangulo/Rectangular)

$E_3 = 0.5 \times 1.2 \times 1 \times 1.000 \times 0.119 = 60$ kg (Suelo saturado/Saturated soil, Triangulo/Triangle)

$E_w = 0.5 \times 1.2 \times 1 \times 1.000y = 600$ kg

$E_t = E_1 + E_2 + E_3 + E_w = 1380$ kg

3) Momentos/Moment

$M = E \times L$

$M_1 = E_1 \times 1.60 = 741 kg-m$

$M_2 = E_2 \times 0.50 = 129 kg-m$

$M_3 = E_3 \times 0.39 = 20 kg-m$

$M_4 = E_4 \times 0.33 = 198 kg-m$

$Mr=M_1+M_2+M_3+M_4=1088$ kg-m

4) Posicion del empuje de momento maxima/Maximum momentum location

$$h = \frac{M}{T} = \frac{1088kg - m}{1380kg} = 0.79m$$

5) Momento ultimo/Ultimate moment

$$M_u = 1.7 \times M = 1.7 \times 1088kg - m = 1850kg - m$$

6) Calculo de penalte/Thickness calculation

$$d = \sqrt{\frac{Mu}{I \times b}} = \sqrt{\frac{1850\ kg - m}{30.55 \times 1.00}} = 7.9 < 12cm,\ OK$$

7) Calculo de Ku/ Calculation of Ku

$$K_u = \frac{M_u}{b \times d} = \frac{1850kg - m}{1.00 \times 12} = 12.85kg / cm^2$$

8) Calculo de la cuantos de acero/ Calculation of minimum reinforcement

$$\rho = 0.85 \frac{f'c}{fy}\left(1 - \sqrt{1 - \frac{2.36K_u}{\phi f'c}}\right) = 0.85 \times \frac{210}{4200}\left(1 - \sqrt{1 - \frac{2.36 \times 12.85}{0.9 \times 210}}\right) = 0.0036$$

9) Calculo de seccion de acero/ Calculation of reinforcement

$$A_s = \rho \times b \times t = 0.0036 \times 100 \times 12 = 4.32cm^2$$

de acero/ # of reinforcement

$4.32cm^2/0.785cm^2=5.5\phi10mm=6.0\ \phi10mm$

para sentido longitudinal/ in longitudinal direction

$\rho_{min}=14/f_y=0.0033$

$A_s = 0.0033 \times 100 \times 12 = 3.96cm^2$

1 ϕ10mm c/20cm

10) Carga sobre losa de fondo/ Load acting on slab

(Peso propio revestimiento/Dead load)

$$P_{pr} = \frac{1}{6}\left[(8.74 \times 0.15 \times 2400)\cos 39°48'20'' + (0.15 \times 1.71 \times 2400)\right] = 505\frac{kg}{m^2}$$

(Subpresion/ Uplift)

$S_p = 1000 \times 1.00 = 1000\frac{kg}{m^2}$

(Reaccion del terreno/ Soil reaction)

Rt=Pw+Pt+Ppn-Sp=0+0+505-1000=495 kg/m^2

(Peso propio losa inferior/ Dead load of bottom slab)

$$P_{pli} = 0.15 \times 2400 = 360\frac{kg}{m^2}$$

(Carga sobre losa inferior/ Load acting on upper slab)

$W=R_t+S_p-P_{pli}=495+1000-360=1135$ kg/cm^2

$$M = \frac{W \times L^2}{8} = \frac{1135 \times 1.11^2}{8} = 415 kgm$$

$$M_u = 1.7 \times 415 = 704 kgm$$

$$d = \sqrt{\frac{704}{330.55 \times 1.00}} = 4.8 < 12$$

$$K_u = \frac{704}{1.00 \times 12^2} = 4.89$$

$$\rho = 0.85\frac{f'c}{fy}\left(1 - \sqrt{1 - \frac{2.36 \times 4.89}{0.9 \times 210}}\right) = 0.0013$$

$\rho=14/f_y=0.0033$

$A_s = 0.0033 \times 1.00 \times 12 = 3.96 cm^2$

1ϕ10mm, c/20cm,

General ANALISIS AND DESIGN OF BOX CULVERT

- Concrete protection for reinforcement CEC 7.7
  r = 7.0 cm
- shrinkage and temperature 4200 Kg/cm², ratio of reinforcement area : 0.0020 CEC 7.12.2.1

- REQUIRED STRESS CEC 9.2.1

$$W_U = 1.4D + 1.7L + 1.7E_p + 1.7P_W$$

W_U = ULTIMATE LOAD

D = dead load
L = Live load
E_p = Earth pressure
P_W = WATER PRESSURE.

- DESIGN STRESS CEC 9.3
  REDUCTION STRESS FACTOR ϕ
  $\phi = 0.9$ Flexure
  $\phi = 0.85$ Shear and torsión

minimon height $\frac{L}{24}$

maximon deflexion $\frac{L}{180}$

L = Length of the beam or slab clear span in one direction

AXIAL AND FLEXURAL LOAD CEC 10.0

minimum reinforcement in member subjected to flexure

$$\rho_{min} = \frac{14}{f_y}$$

SHEAR STRESS CEC 11

$V_U \leq \phi V_n$

V_U Shear force in the section
V_n nominal shear strength

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$V_n = \phi(V_c + V_s)$

V_c = nominal shear strength supplied by the concrete.

V_s = " " strength supplied by the reinforcement

for member subjected to shear and flexure.

$V_c = \phi\, 0.53 \sqrt{f'_c}\, b_w\, d$

b_w = width of the web

d = effective depth.

if $V_u > V_c$ require Av with steel

$\phi V_s = \dfrac{A_v f_y d}{s}$

A_v = shear reinforcement area

f_y = yield strength.

$A_v = \dfrac{V_s}{\phi f_y \cdot \sin\alpha}$ (for diagonal bars)

s = separation of shear reinforcement in parallel direction

$A_v = \dfrac{V_s \times s}{\phi f_y \cdot d}$ (for stirrups)

— REINFORCEMENT IN MEMBER SUBJECTED TO FLEXURE

$A_s = \dfrac{M_u}{\phi f_y \left(d - \frac{a}{2}\right)}$

A_s = flexural reinforcement area.

ρ = ratio for calculation of the reinforcement.

$\rho = \dfrac{A_s}{b d}$

$a = \dfrac{\rho \cdot f_y \cdot d}{0.85 f'_c}$

$a = \dfrac{A_s \cdot M}{0.85 \cdot f'_c \cdot b}$

CEC 10.5

$\rho_{min} = \dfrac{14}{f_y}$

CEC 7.12.1.a

$\rho_{\text{shrinkage/temperatura}} = 0.002$

$A_s = 0.002 \times b \times d$

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1.- Design Condition

1.1 General Condition

1) Clearence of culvert 1.2 x 1.2 mt.

2) Earth Covering $h = 5.0$ mt.
$h = 10.0$ mt.
$h = 13.0$ mt.

1.2 NATURAL CONDITION

1) ELEVATION of GROUND water

$G_{WL} = 0.0$ mt.

1.3 Geological condition (unit weight)

1) Earth unit weight = 1.80 T/m³

2) coefficient of earth pressure = 0.50 = Ka

1.4 Material

1) $F'c = 180$ Kg/cm² STRENGTH OF CONCRETE.
$Fy = 4200$ Kg/cm² YIELD STRENGTH OF REINFORCING BAR.

MODULUS OF ELASTICITY

$E_{CONCRETE} = 2.1 \times 10^5$ Kg/cm²

$E_{STEEL} = 2.1 \times 10^6$ Kg/cm²

$\gamma_{CONCRETE} = 2.400$ Kg/m³ UNIT WEIGHT

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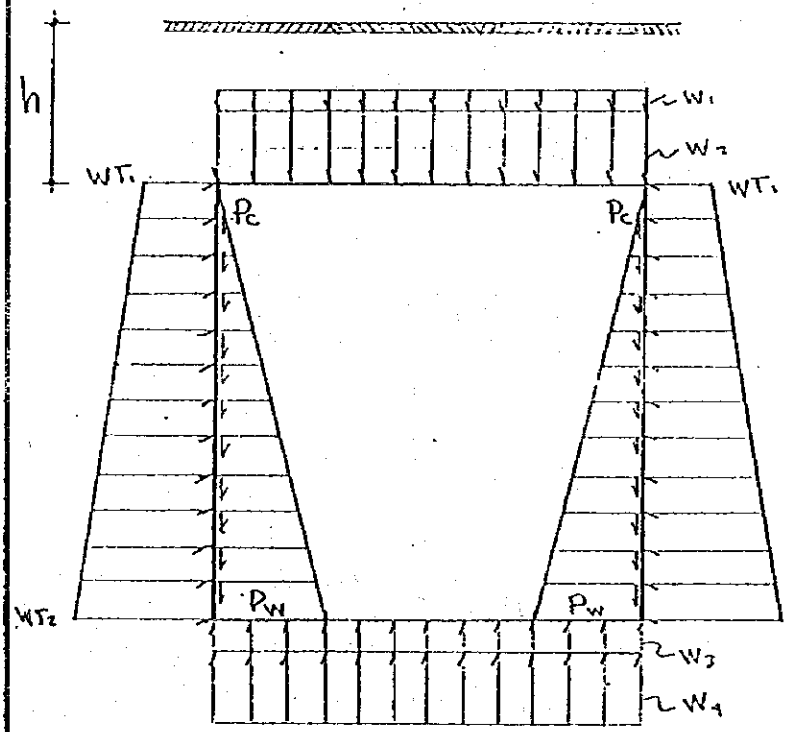
LOADS FOR BOX CULVERT 1.2 x 1.2 x 0.3

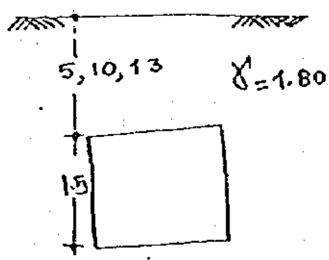
- THE LOADS APPLIED IN THE DESIGN OF BOX CULVERT ARE AS FOLLOWS:

W_1: LIVE LOAD OF HS-20-44 TRAILER
W_2: VERTICAL LOAD OF EARTH WEIGHT AND SELF WEIGHT OF TOP SLAB
W_3: SURCHARGE OF THE UPLIFT ACT ON BOTTOM SLAB.
W_4: REACTION LOAD ACT ON BOTTOM SLAB
WT: LATERAL EARTH PRESSURE
Pw: LATERAL WATER PRESURE
Pc: LOAD OF CONCRETE

DATA

$\gamma_{soil} = 1.8\ T/M^3$ UNIT WEIGHT
$\phi = 20°$ ANGLE OF INTERNAL FRICTION
$\gamma_{concrete} = 2.4\ T/M^3$ UNIT WEIGHT
$Ka = 0.50$ COEFFICIENT OF EARTH PRESSURE
$f'c = 180\ Kg/cm^2$ STRENGTH OF CONCRETE
$fy = 1200\ Kg/cm^2$ YIELD STRENGHT OF REFORCING BAR
$E_{concrete} = 2.1 \times 10^5$ MODULUS OF ELASTICITY (Kg/cm^2)
$E_{steel} = 2.1 \times 10^6$





Load / Section	W_1	W_2	W_3	W_4	WT_1	WT_2	Pc	Pw	h
1.2x1.2 0.3		9.27 0.72		9.27 2.16	4.64	5.99	1.08	1.5	5.0
1.2x1.2 0.3		18.27 0.72		18.27 2.16	9.14	10.49	1.08	1.5	10.0
1.2x1.2 0.3		23.67 0.72		23.67 2.16	11.84	13.19	1.08	1.5	13.0

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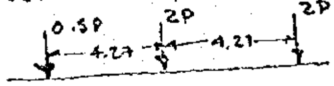
ANALYSIS FOR W1

W1 = LIVE LOAD OF HS-20-44 TRAILER

SURCHARGE HS-20-44



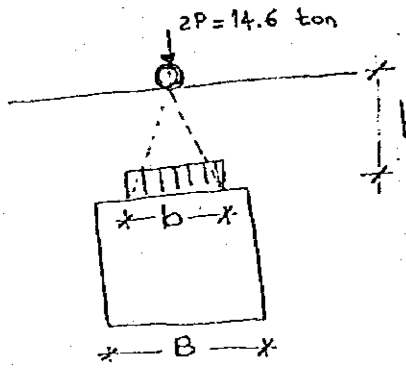
W = 40000 lb

2P = 0.8 W

P = 7.3 ton

$W_1 = \frac{14.60}{4.27 \times 3.05} = 1.12\ T/m^2$

$h'_s = \frac{1.12}{1.8} = 0.622\ mt \approx 0.65\ mt$ (SOIL EQUIVALENT HEIGTH)



a = 2h + 2.25 (1 lane)

a = 2h + 5.0 (2 lane)

b = 2h + 0.2

b = 2 x 1.5 = 3.0 BOXCULVERT 1.2 x 1.2

b > B

$W_1 = \frac{2PN(1+i)}{a \cdot b}$

W_1 = vertical load (ton/m²)

i = impact coeficient

P = rear wheel load (ton)

a = Distribution width (m)

b = Distribution Length (m)

N = number of lanes

h = earth coverage depth.

a = 2 x 5 + 2.25 = 12.25

b = 2 x 5 + 0.2 = 10.20

$W = \frac{2 \times 7.3 (1.3)}{12.25 \times 10.2} = 0.15\ Ton/m^2$

$h'_s = \frac{0.15}{1.8} = 0.15\ Ton/m^2$

$h'_{s(min)} = 0.65\ mt$ (CODE AASHTO)

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1.5 DESIGN LOAD

W_1 = LIVE LOAD OF HS-20 44 TRAILER

W_1 can be neglected because W_2 (vertical load of earth weight) > W_1

W_2 = VERTICAL LOAD OF EARTH WEIGHT AND SELF WEIGHT OF TOP SLAB

$= \gamma_{EARTH}$ (unit weight) × h + $\gamma_{CONCRETE}$ (unit weight) × thickness

W_3 = SURCHARGE OF THE UPLIFT ACT ON BOTTOM SLAB

$= \gamma'_{WATER}$ (unit) × H H: (Groundwater level - Bottom level)

W_4 = REACTION LOAD ACT ON BOTTOM SLAB

$$= W_1 + W_2 + \frac{2P_c}{B}$$

P_c = load of concrete of the walls

W_T = Lateral earth pressure

$= \gamma_{EARTH}$ (unit weight) × h × Ka

Ka = 0.5

$W_2 = 1.80 \times 5.150 + 2.4 \times 030 = 9.99$ T/m

$W_3 = 0.0$ T/m

$$W_4 = 9.27 + 0.72 + 2 \times \frac{0.30 \times 2.4 \times 1.5}{1.5}$$

$$= 9.27 + 0.72 + \frac{2 \times 1.08}{1.5} = 11.43 \text{ T/m}$$

$W_{T1} = 1.80 \times 5.15 \times 0.50 = 4.64$ T/m

$W_{T2} = 1.80 \times 6.65 \times 0.50 = 5.99$ T/m

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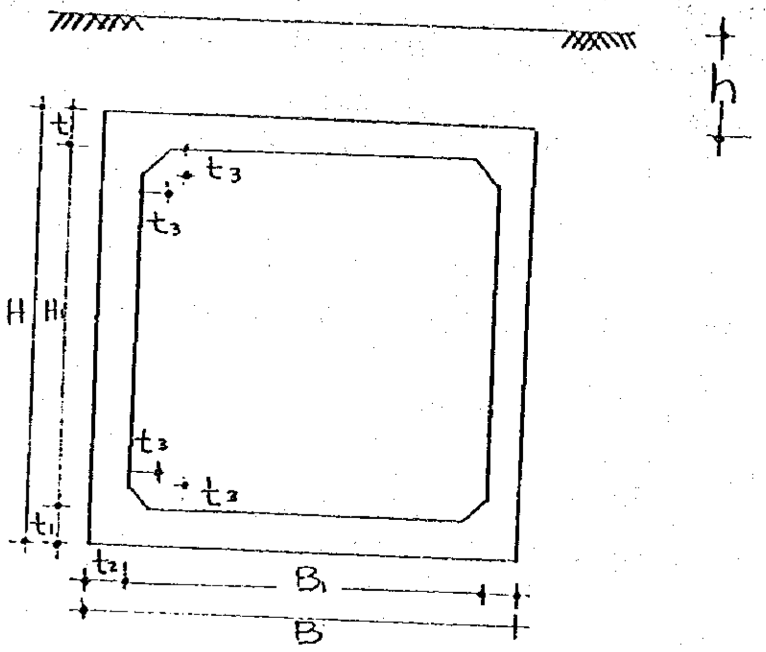
ANALYSIS AND DESIGN OF THE BOX CULVERT 1.2x1.2

- SHAPE AND DIMENSION



DIM	h		
	5.0	10.0	13.0
H	1.8	1.8	1.8
H1	1.2	1.2	1.2
B	1.8	1.8	1.8
B1	1.2	1.2	1.2
t	0.30	0.30	0.30
t1	0.30	0.30	0.30
t2	0.3	0.30	0.30
t3	0.15	0.15	0.15

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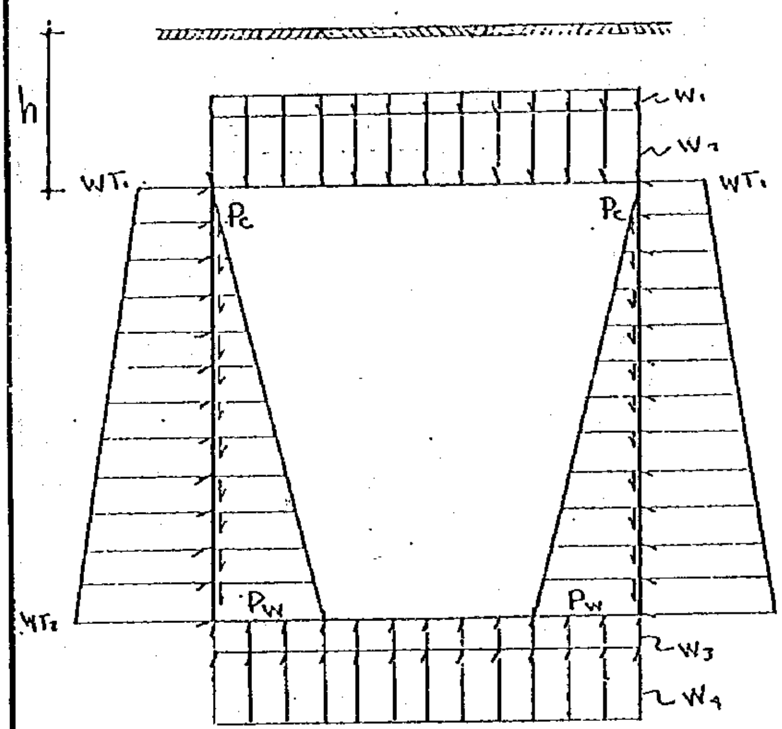
BOX CULVERT 1.5 x 1.5 x 0.3

- THE LOADS APPLIED IN THE DESIGN OF ARE AS FOLLOWS :
  - W_1: LIVE LOAD OF HS-20-44 TRAILER
  - W_2: VERTICAL LOAD OF EARTH WEIGHT AND SELF WEIGHT OF TOP SLAB
  - W_3: SURCHARGE OF THE UPLIFT ACT ON BOTTOM SLAB.
  - W_4: REACTION LOAD ACT ON BOTTOM SLAB
  - WT: LATERAL EARTH PRESSURE
  - Pw: LATERAL WATER PRESSURE
  - Pc: LOAD OF CONCRETE



DATA

$\gamma_{soil} = 1.8$ T/m³ UNIT WEIGHT

$\phi = 20°$ ANGLE OF INTERNAL FRICTION

$\gamma_{concrete} = 2.4$ T/m³ UNIT WEIGHT

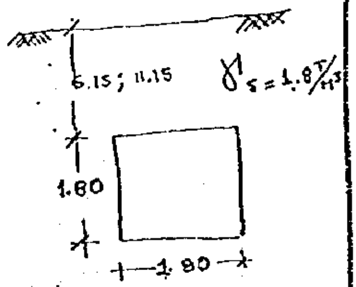
Ka = 0.5 COEFFICIENT OF EARTH PRESSURE

f'c = 180 Kg/cm² STRENGTH OF CONCRETE

fy = 1200 Kg/cm² YIELD STRENGHT OF REFORCING BAR

E concrete = 2.1x10⁵ MODULUS OF ELASTICITY (Kg/cm²)

E steel = 2.1x10⁶



CASE / SECTION	W_1	W_2	W_3	W_4	WT_1	WT_2	P_c	P_w	h
1.5,1.5 0.3		11.07 0.72		11.07 2.16	5.54	7.16	1.30	1.80	6.0
1.5x1.5 0.3		20.07 0.72		20.07 2.16	10.04	11.66	1.30	1.80	11.0

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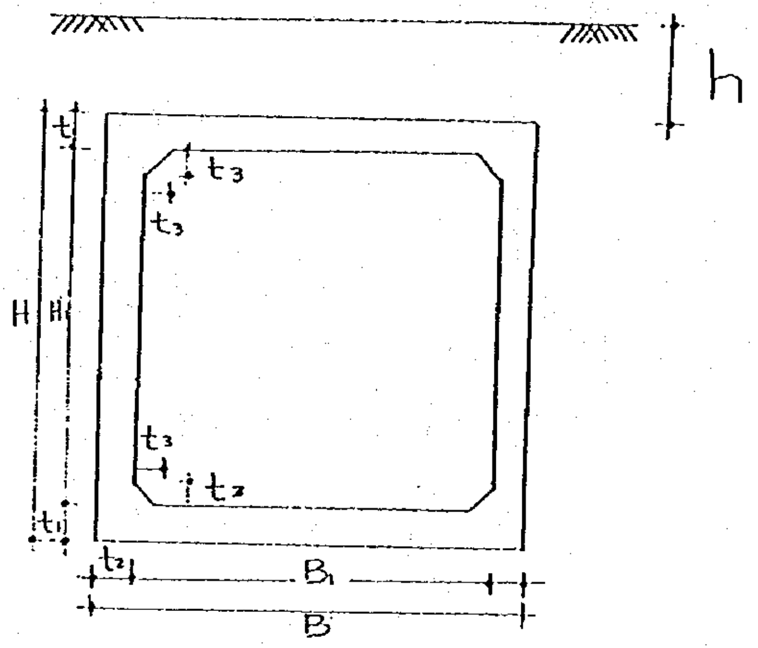
BOX CULVERT 1.5 x 1.5



DIM	h	
	6.0	11.0
H	2.10	2.10
H_1	1.50	1.50
B	2.10	2.10
B_1	1.50	1.50
t	0.30	0.30
t_1	0.30	0.30
t_2	0.30	0.30
t_3	0.15	0.15

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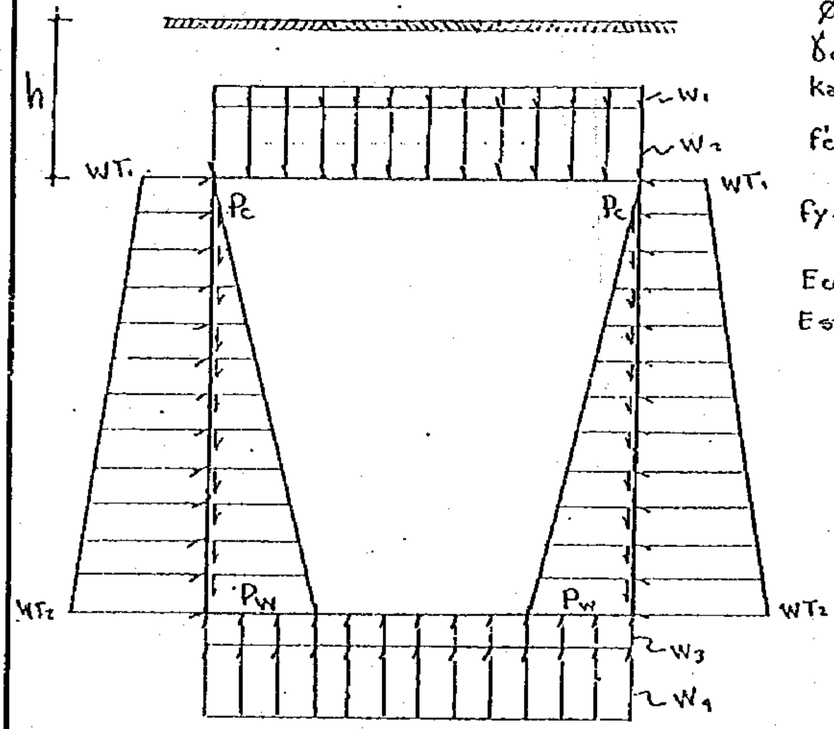
BOX CULVERT 2.0 × 2.0 × 0.3

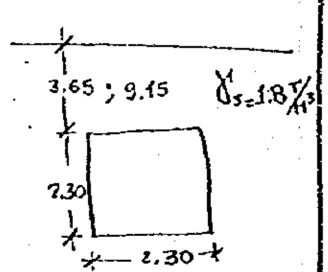
- THE LOADS APPLIED IN THE DESIGN OF ARE AS FOLLOWS :

W1 : LIVE LOAD OF HS-20-44 TRAILER
W2 : VERTICAL LOAD OF EARTH WEIGHT AND SELF WEIGHT OF TOP SLAB
W3 : SURCHARGE OF THE UPLIFT ACT ON BOTTOM SLAB.
W4 : REACTION LOAD ACT ON BOTTOM SLAB
WT : LATERAL EARTH PRESSURE
Pw : LATERAL WATER PRESSURE
Pc : LOAD OF CONCRETE



DATA

$\gamma_{soil} = 1.8\ T/M^3$ UNIT WEIGHT
$\phi = 20°$ ANGLE OF INTERNAL FRICTION
$\gamma_{concrete} = 2.4\ T/M^3$ UNIT WEIGHT
$K_a = 0.5$ COEFFICIENT OF EARTH PRESSURE
$f'_c = 180\ Kg/cm^2$ STRENGTH OF CONCRETE
$f_y = 1200\ Kg/cm^2$ YIELD STRENGHT OF REFORCING BAR
$E_{concrete} = 2.1 \times 10^5$ MODULUS OF ELASTICITY (Kg/cm^2)
$E_{steel} = 2.1 \times 10^6$



CARGA / SECCION	W1	W2	W3	W4	WT1	WT2	Pc	Pw	h
2.0×2.0×0.3		6.57 0.72		6.57 2.16	3.29	5.36	1.66	2.30	3.5
2.0×2.0×0.3		16.47 0.72		16.47 2.16	8.24	10.31	1.66	2.3	9.0

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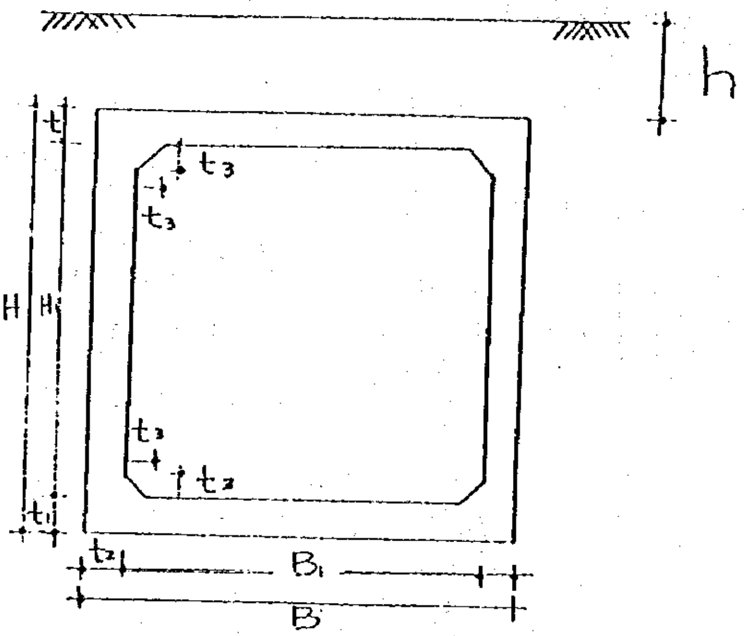
BOX CULVERT 2.0 x 2.0



DIM	h	
	3.50	9.0
H	2.60	2.60
H_1	2.0	2.0
B	2.60	2.60
B_1	2.0	2.0
t	0.3	0.3
t_1	0.3	0.3
t_2	0.3	0.3
t_3	0.15	0.15

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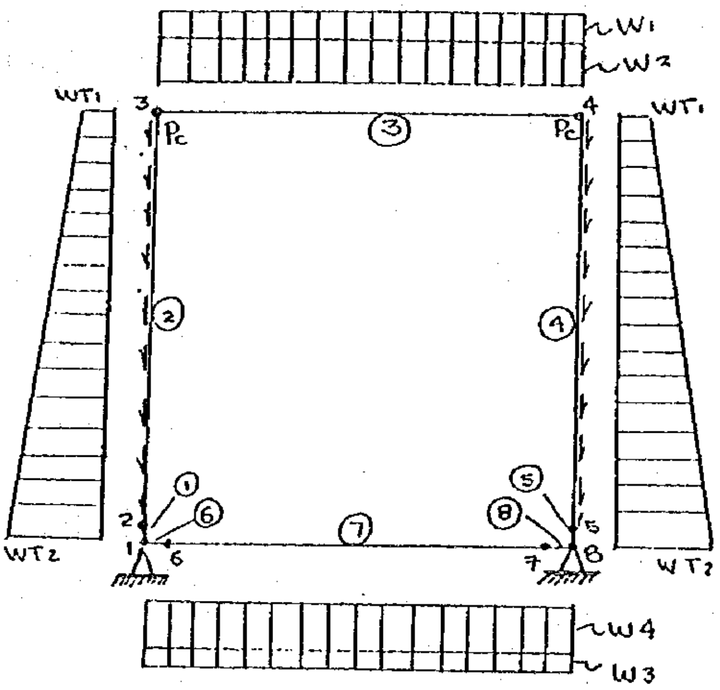
4- MODEL OF STRUCTURAL ANALISIS.



PARA EL ANALISIS SE USO EL MODELO QUE SE INDICA EN EL GRAFICO, LOS DATOS ESTAN PREPARADOS PARA EL PROGRAMA SAP90.

LA CONSIDERACION DE CARGA ES LA SIGUIENTE:

$$U = 1.4 D + 1.7 L + 1.7 E_p + 1.7 P_W$$

SE REALIZO LA EJECUCIÓN DEL PROGRAMA, OBTENIENDOSE MOMENTOS, CORTES, CARGAS AXIALES Y TORSIÓN COMO SE INDICA EN LOS GRAFICOS QUE ADJUNTO.

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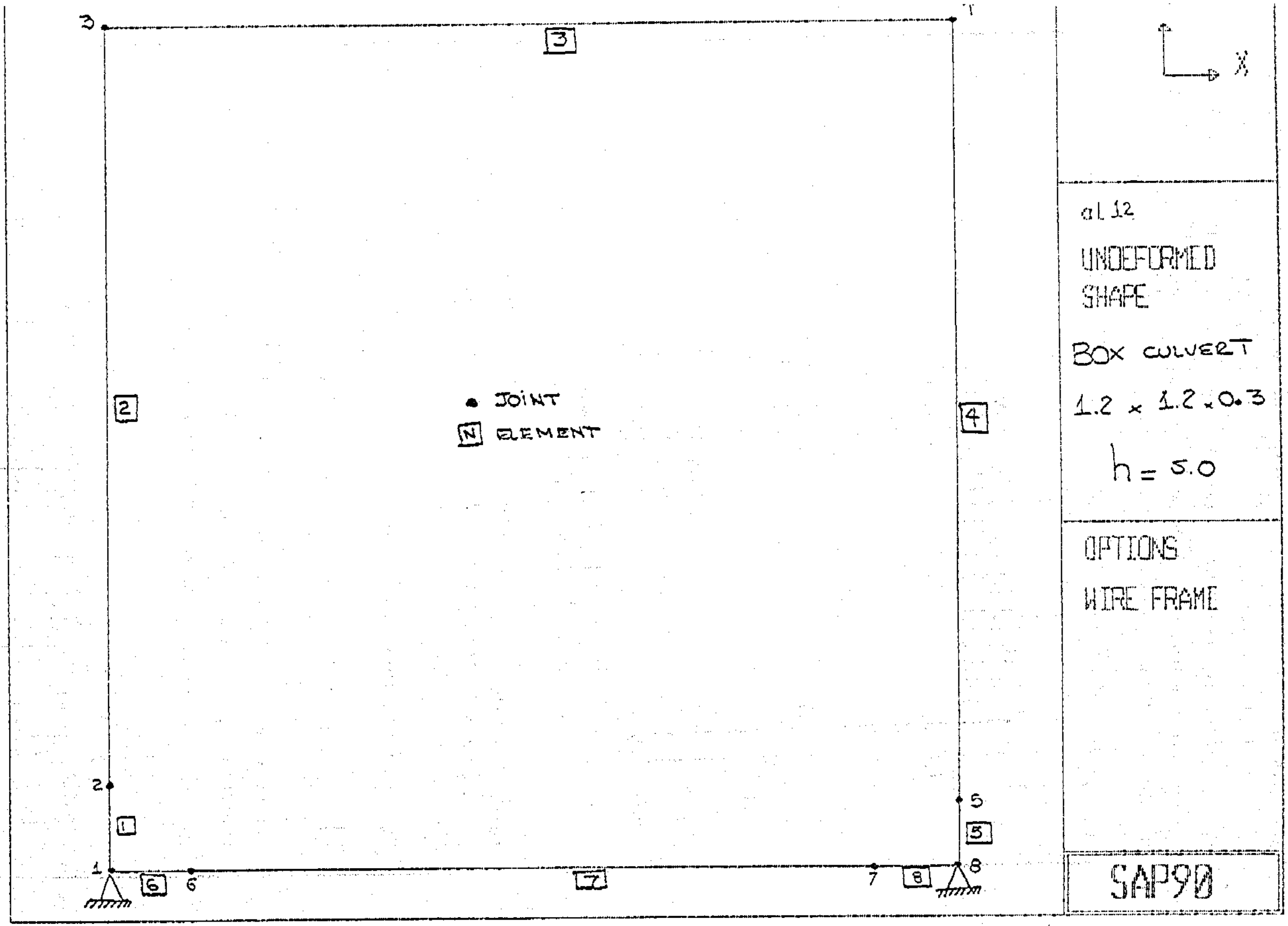
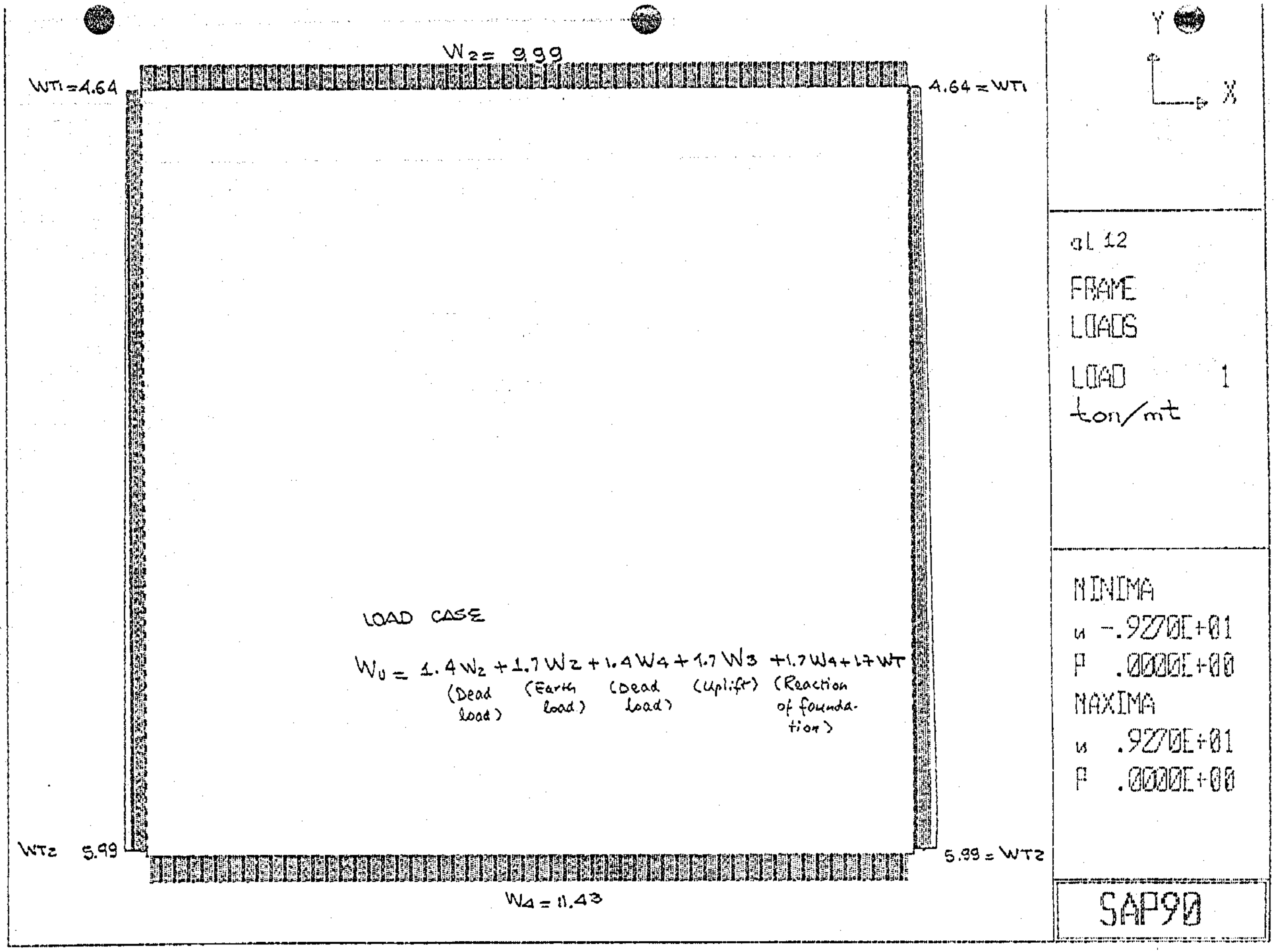
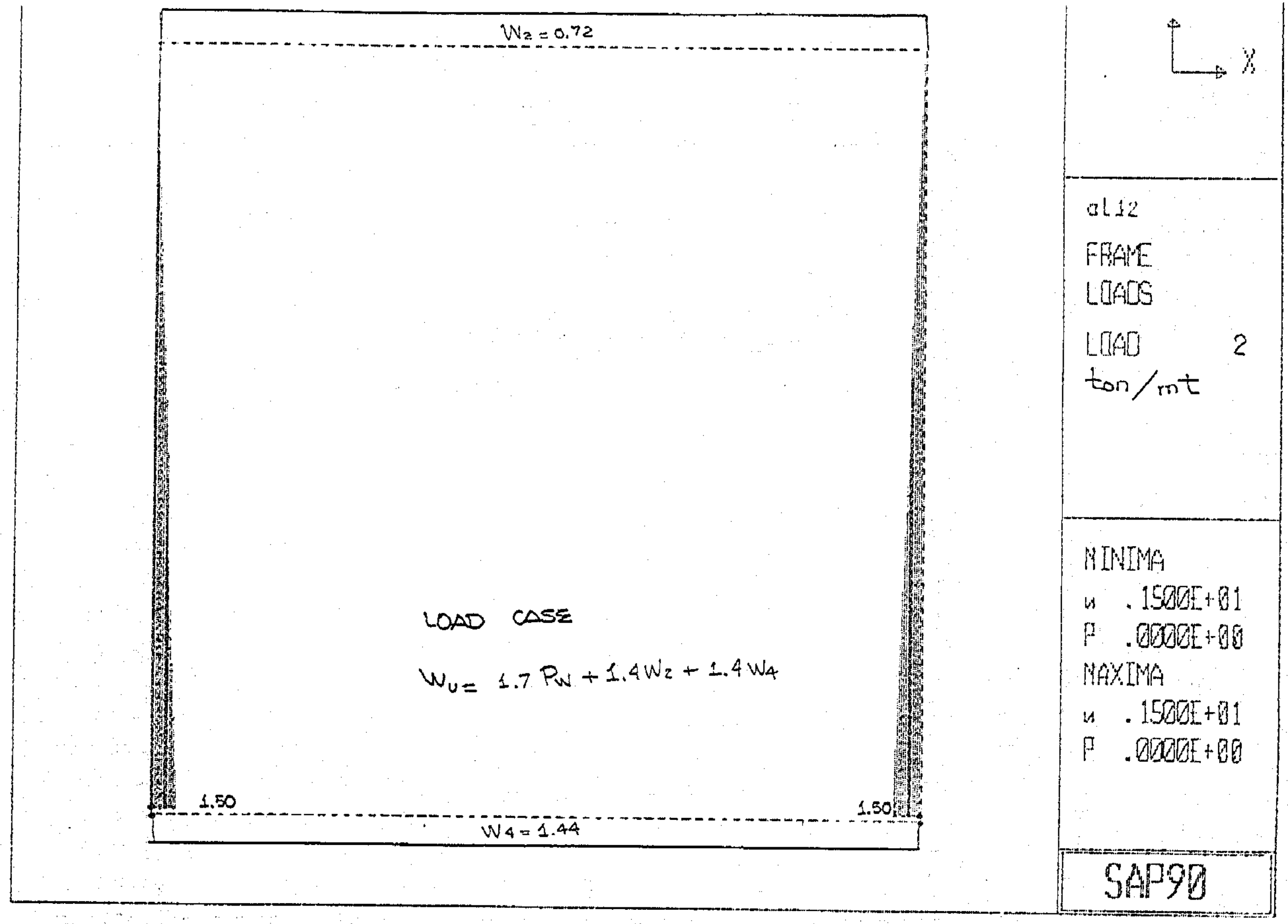


3
4
3
2
4
JOINT
N ELEMENT
2
1
1
6
6
7
7
8
8
5
5
X
aL12
UNDEFORMED
SHAPE
BOX CULVERT
1.2 x 1.2 x 0.3
h = 5.0
OPTIONS
WIRE FRAME
SAP90


W2 = 9.99
WT1 = 4.64
4.64 = WT1
WT2 5.99
5.99 = WT2
W4 = 11.43
LOAD CASE
Wu = 1.4 W2 + 1.7 W2 + 1.4 W4 + 1.7 W3 + 1.7 W4 + 1.7 WT
(Dead load) (Earth load) (Dead load) (Uplift) (Reaction of foundation)
Y
X
al 12
FRAME
LOADS
LOAD 1
ton/mt
MINIMA
u -.9270E+01
P .0000E+00
MAXIMA
u .9270E+01
P .0000E+00
SAP90


W2 = 0.72
LOAD CASE
Wu = 1.7 PW + 1.4 W2 + 1.4 W4
1.50
1.50
W4 = 1.44
X
al12
FRAME
LOADS
LOAD 2
ton/mt
MINIMA
M .1500E+01
P .0000E+00
MAXIMA
M .1500E+01
P .0000E+00
SAP90

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[illegible]
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[illegible] 77          [illegible]
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[illegible]             [illegible]
[illegible]             [illegible]
[illegible],[illegible] 50,0,1 49          rpw liq.
[illegible]=0,0,0,1.49,1 50               rpw der.
[illegible]=0,-[illegible],0,1.4900,-4.04  twt liq
[illegible]=0,-4.64,0,1.4900,-5.07         twt der.
[illegible] 2   N=1           [illegible]=1,0
[illegible] 3                 NSL=3,0,6
[illegible] 4                 nsl=1,2
[illegible] 5                 nsl=2,0,7
[illegible] 6
[illegible] 6   N=1
[illegible] 7                 nsl=3,4,0,5
[illegible] 8

[illegible]
C=1 7,1.4,0,1.7
c=0,1 4,1.7
```

STRUCTURAL ANALYSIS PROGRAM

VERSION [illegible]

FRAME ELEMENT FORCES

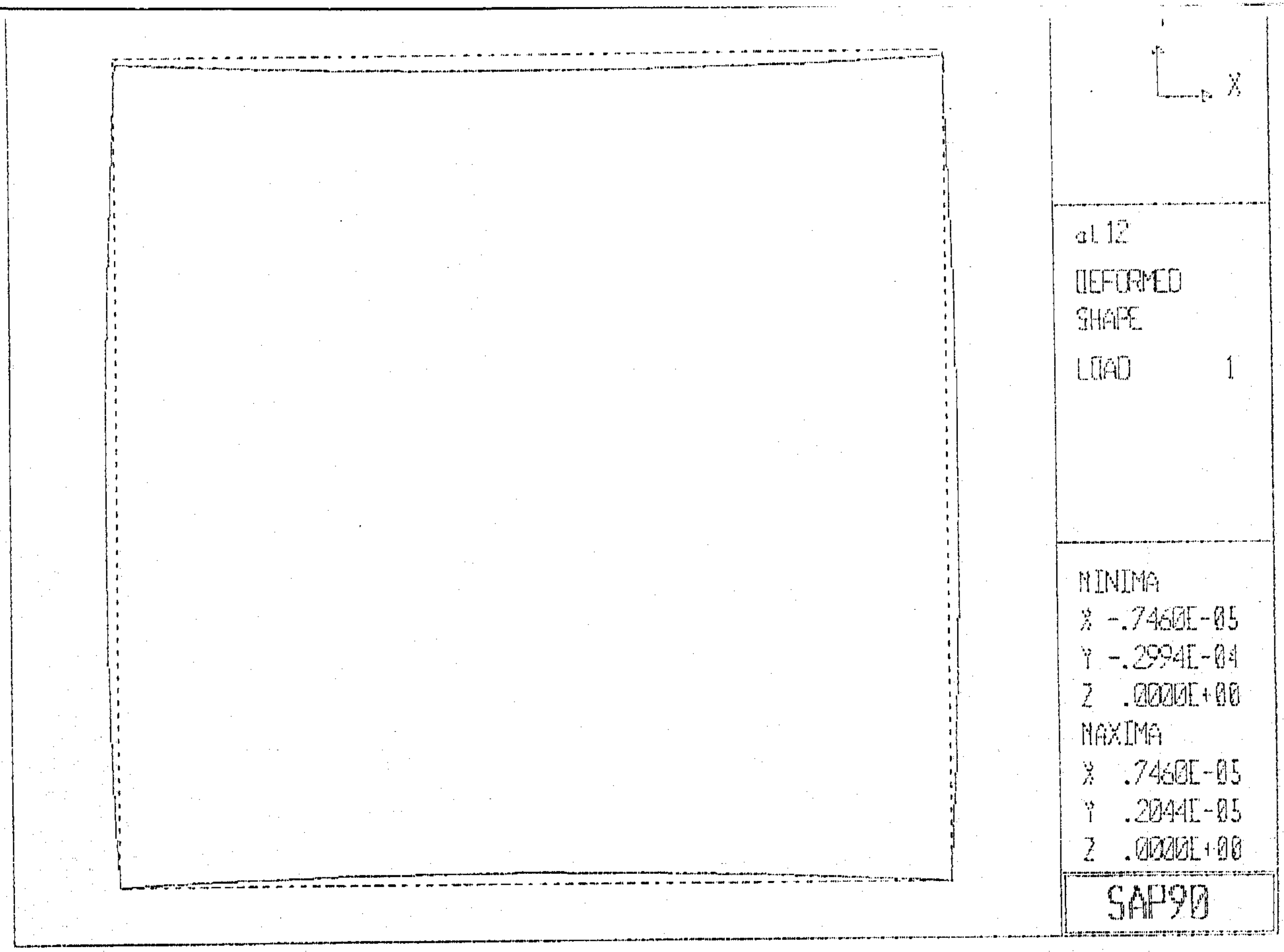
LOAD COND	DIST ENDI	1-2 PLANE SHEAR	1-2 PLANE MOMENT	AXIAL FORCE	1-3 PLANE SHEAR	1-3 PLANE MOMENT	AXIAL TORQ
1	.000			-12.575			
	.000	7.197	-2.684				
	.010	7.197	-2.612				
	.010			-12.575			
2	.000			-.756			
	.000	-1.094	-.191				
	.010	-1.094	-.202				
	.010			-.756			
1	.000			-12.575			
	.000	7.197	-2.612				
	.749	.000	.050				
	1.490	-6.266	-2.343				
	1.490			-12.575			
2	.000			-.756			
	.000	-1.094	-.202				
	.330	.000	-.467				
	1.490	.805	.054				
	1.490			-.756			
1	.000			-6.266			
	.000	12.575	-2.343				
	.750	.000	2.372				
	1.500	-12.575	-2.343				
	1.500			-6.266			
2	.000			.805			
	.000	.716	.054				
	.750	.000	.338				
	1.500	-.756	.054				
	1.500			.805			

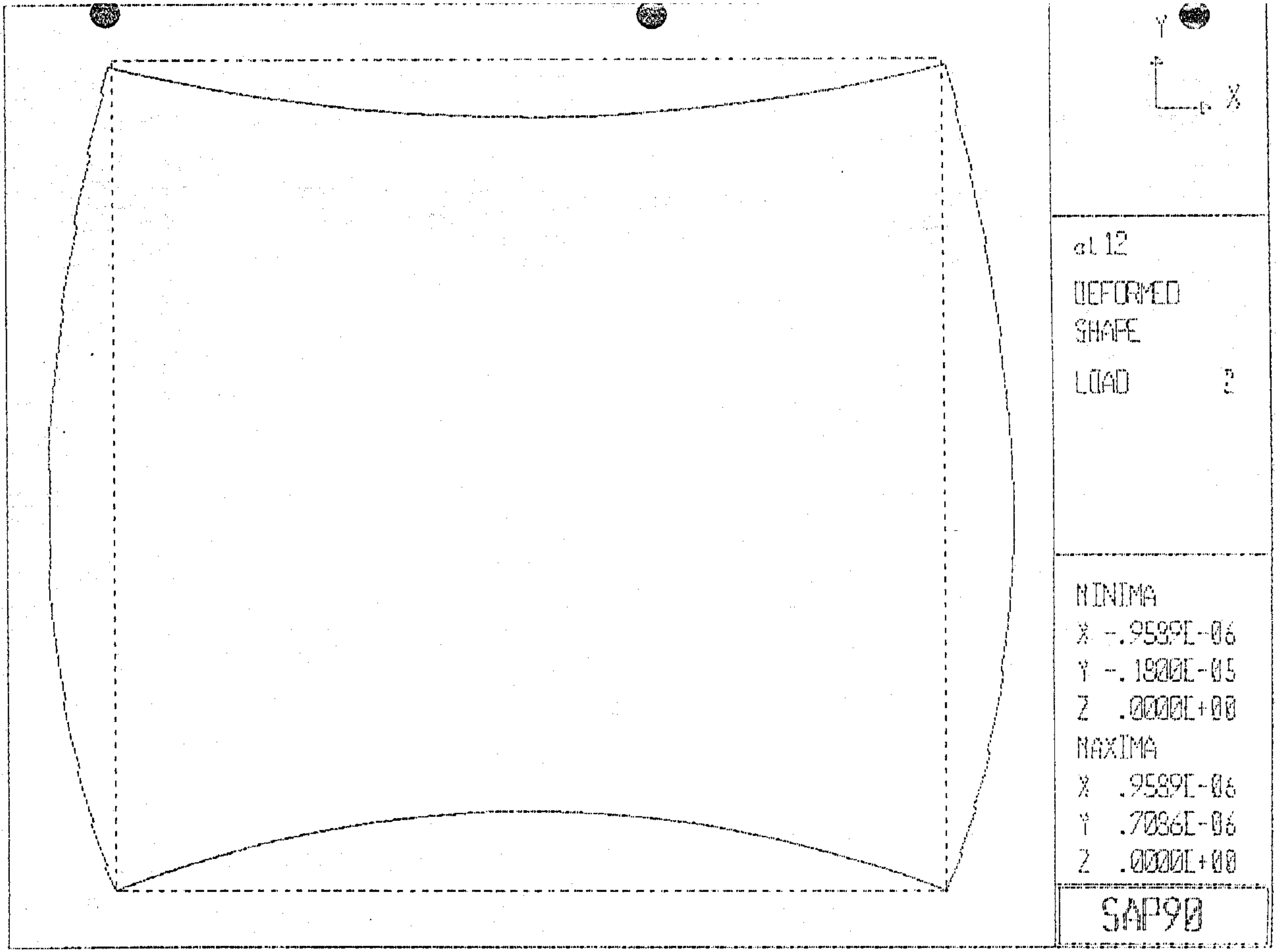
[illegible]	[illegible]			[illegible]
	[illegible]	[illegible]	[illegible]	
	[illegible]	[illegible]	[illegible]	
	[illegible]	[illegible]	[illegible]	
	[illegible]			[illegible]
[illegible]	[illegible]			[illegible]
	[illegible]	[illegible]	[illegible]	
	[illegible]	[illegible]	[illegible]	
	1.480	1.094	[illegible]	
	1.480			[illegible]

[illegible]

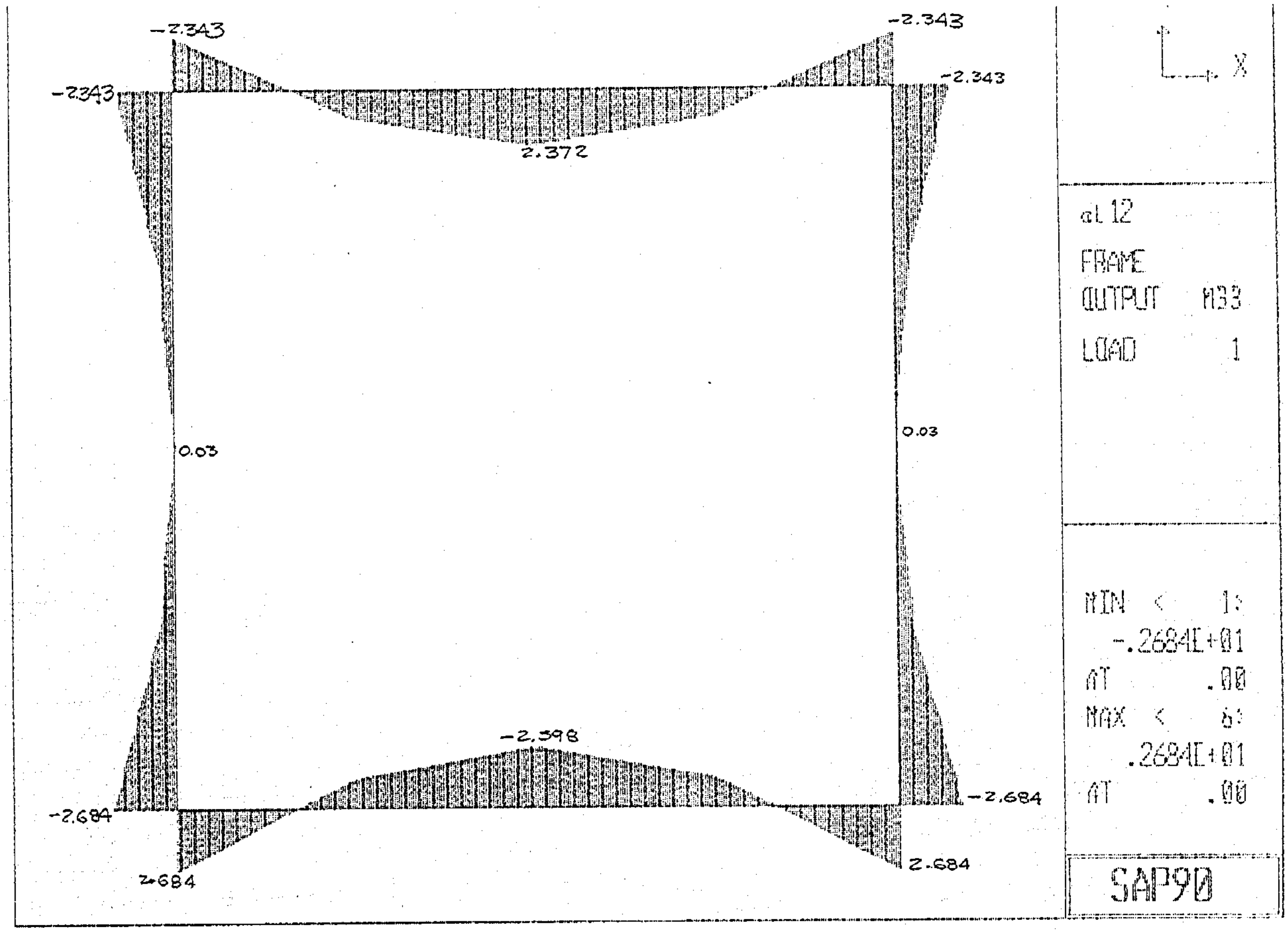
[illegible]

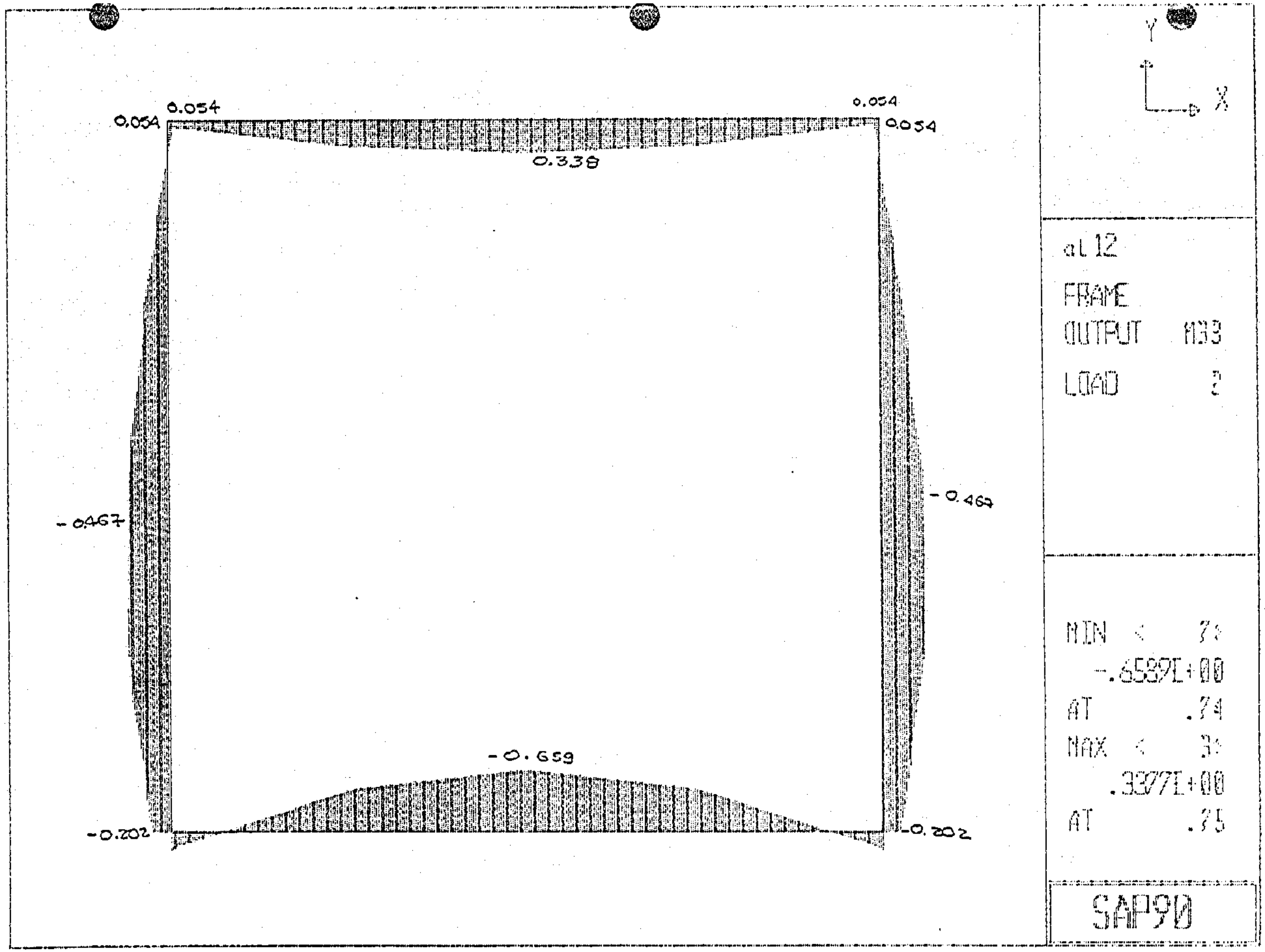
LOAD COND	DIST (FT)	1-2 PLANE SHEAR	1-2 PLANE MOMENT	AXIAL FORCE	1-3 PLANE SHEAR	1-3 PLANE MOMENT	TORSION
1	.000			-12.575			
	.000	-7.197	-[illegible].612				
	.010	-7.197	-2.684				
	.010			-12.575			
2	.000			-.756			
	.000	1.094	-.207				
	.010	1.094	-.191				
	.010			-.756			
1	.000			.000			
	.000	-13.899	2.684				
	.010	-13.899	2.545				
	.010			.000			
2	.000			.000			
	.000	-2.238	.191				
	.010	-2.238	.169				
	.010			.000			
1	.000			.000			
	.000	-13.899	2.545				
	.740	.000	-2.598				
	1.480	13.899	2.545				
	1.480			.000			
2	.000			.000			
	.000	-2.238	.169				
	.740	.000	-.659				
	1.480	2.238	.169				
	1.480			.000			
1	.000			.000			
	.000	13.899	2.545				
	.010	13.899	2.684				
	.010			.000			
2	.000			.000			
	.000	2.238	.169				
	.010	2.238	.191				
	.010			.000			

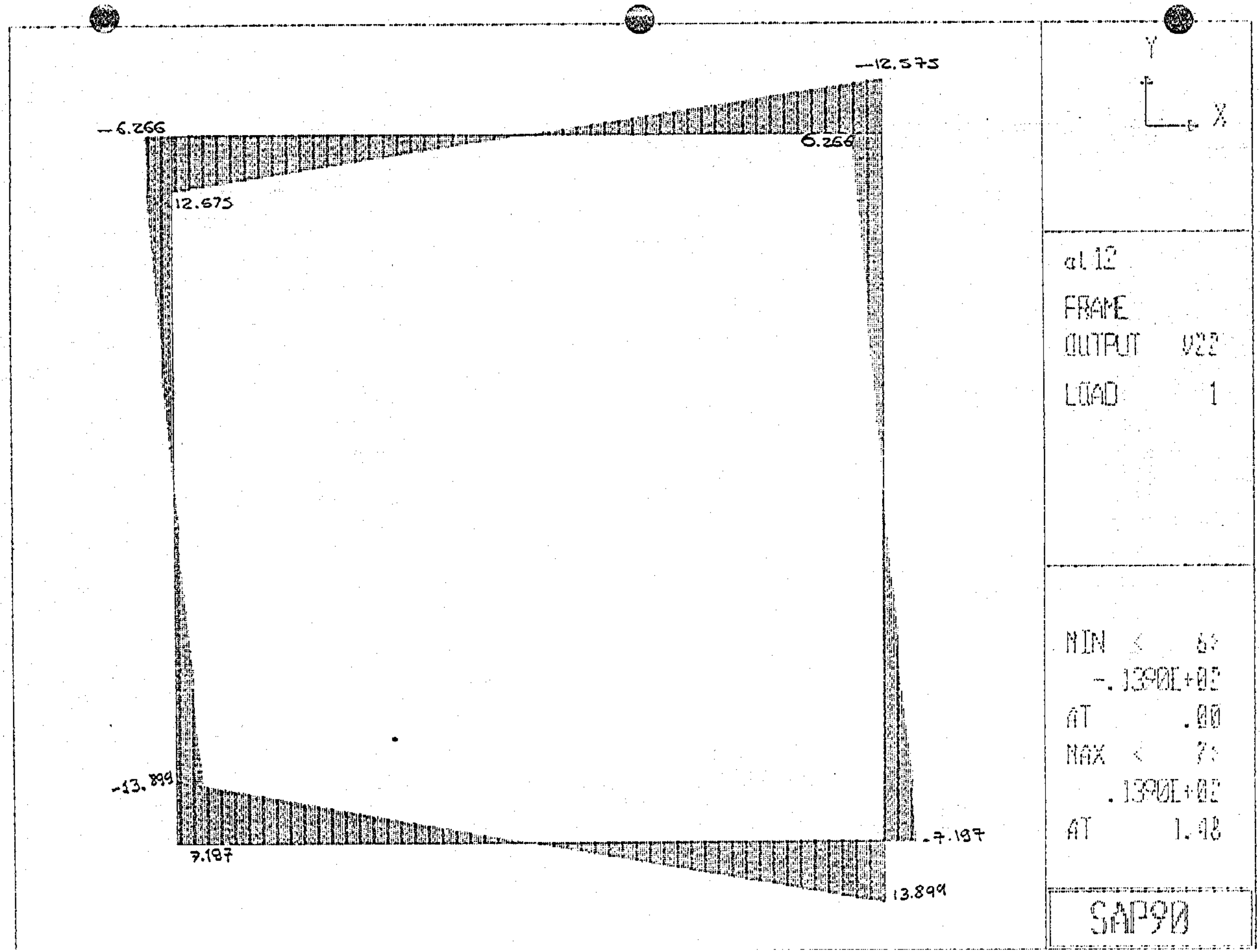

X
al 12
DEFORMED
SHAPE
LOAD 1
MINIMA
X -.7460E-05
Y -.2994E-04
Z .0000E+00
MAXIMA
X .7460E-05
Y .2044E-05
Z .0000E+00
SAP90

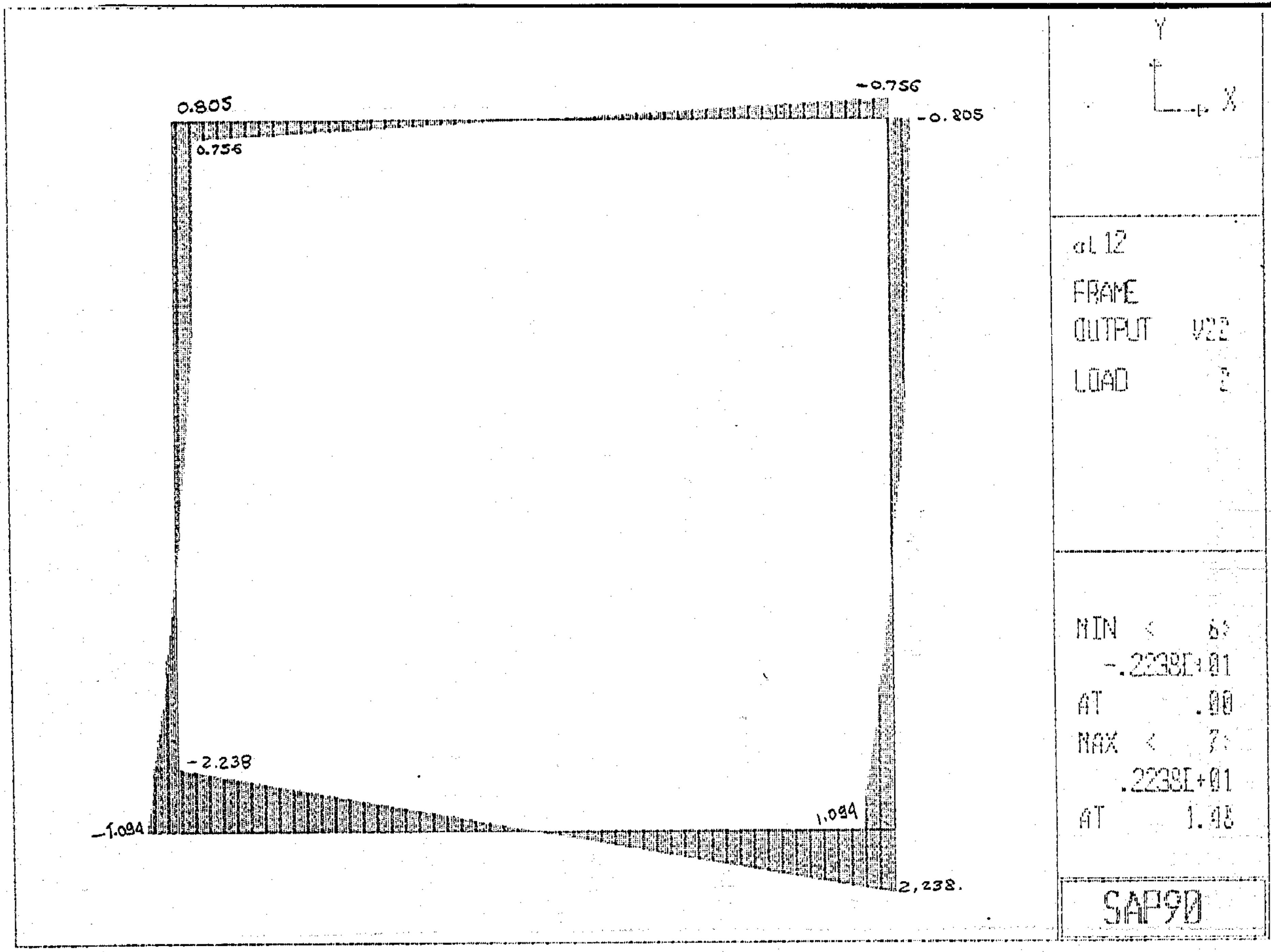


Y
X
al 12
DEFORMED
SHAPE
LOAD 2
MINIMA
X -.9589E-06
Y -.1900E-05
Z .0000E+00
MAXIMA
X .9589E-06
Y .7086E-06
Z .0000E+00
SAP90


-2.343
-2.343
-2.343
-2.343
2.372
0.03
0.03
-2.598
-2.684
-2.684
2.684
2.684
al 12
FRAME
OUTPUT M33
LOAD 1
MIN < 1>
-.2684E+01
AT .00
MAX < 6>
.2684E+01
AT .00
SAP90


Y
X
0.054
0.054
0.054
0.054
0.338
-0.467
-0.467
-0.659
-0.202
-0.202
al 12
FRAME
OUTPUT M33
LOAD 2
MIN < 7>
-.6589E+00
AT .74
MAX < 3>
.3377E+00
AT .75
SAP90


-6.266
-12.575
12.575
6.266
-13.899
7.187
-7.197
13.899
Y
X
al12
FRAME
OUTPUT V22
LOAD 1
MIN < 6>
-.1390E+02
AT .00
MAX < 7>
.1390E+02
AT 1.48
SAP90


Y
X
0.805
0.756
-0.756
-0.805
-2.238
-1.094
1.094
2,238.
al 12
FRAME
OUTPUT V22
LOAD 2
MIN < 6>
-.2238E+01
AT .00
MAX < 7>
.2238E+01
AT 1.48
SAP90

ANALYSIS AND DESIGN OF BOX CULVERT 1.2x1.2x0.3

5.- DATA FOR CALCULATION OF REINFORCEMENT

$f'_c = 180$ Kg/cm² $\quad M_U = 2.684$ ton-mt.

$f_y = 4200$ Kg/cm²

$b = 100$ cm

$d = 23$ cm

$r = 7$ cm

$$A_s = \frac{2.684 \times 10^5}{0.9 \times 4200 \left(23 - \frac{7}{2}\right)} = 3.64 \text{ cm}^2$$

$$a = \frac{3.64 \times 4.200}{0.85 \times 180 \times 100} = 1.0 \text{ cm}$$

$$A_s = \frac{2.684 \times 10^5}{0.9 \times 4200 \,(23 - 0.50)} = 3.16 \text{ cm}^2$$

$$\rho = \frac{A_s}{b\,d} = \frac{3.16}{100 \times 23} = 0.001372$$

$\rho < \rho_{min}$

$A_s = 0.0033\, b \cdot d$
$= 7.59$ cm² => 1Ø12@16
7Ø12

WITH BEAM DESIGN PROGRAM $\quad A_s = 7.66 \Rightarrow 7Ø12 = 1Ø12@16$

– SHEAR STRESS CHECK

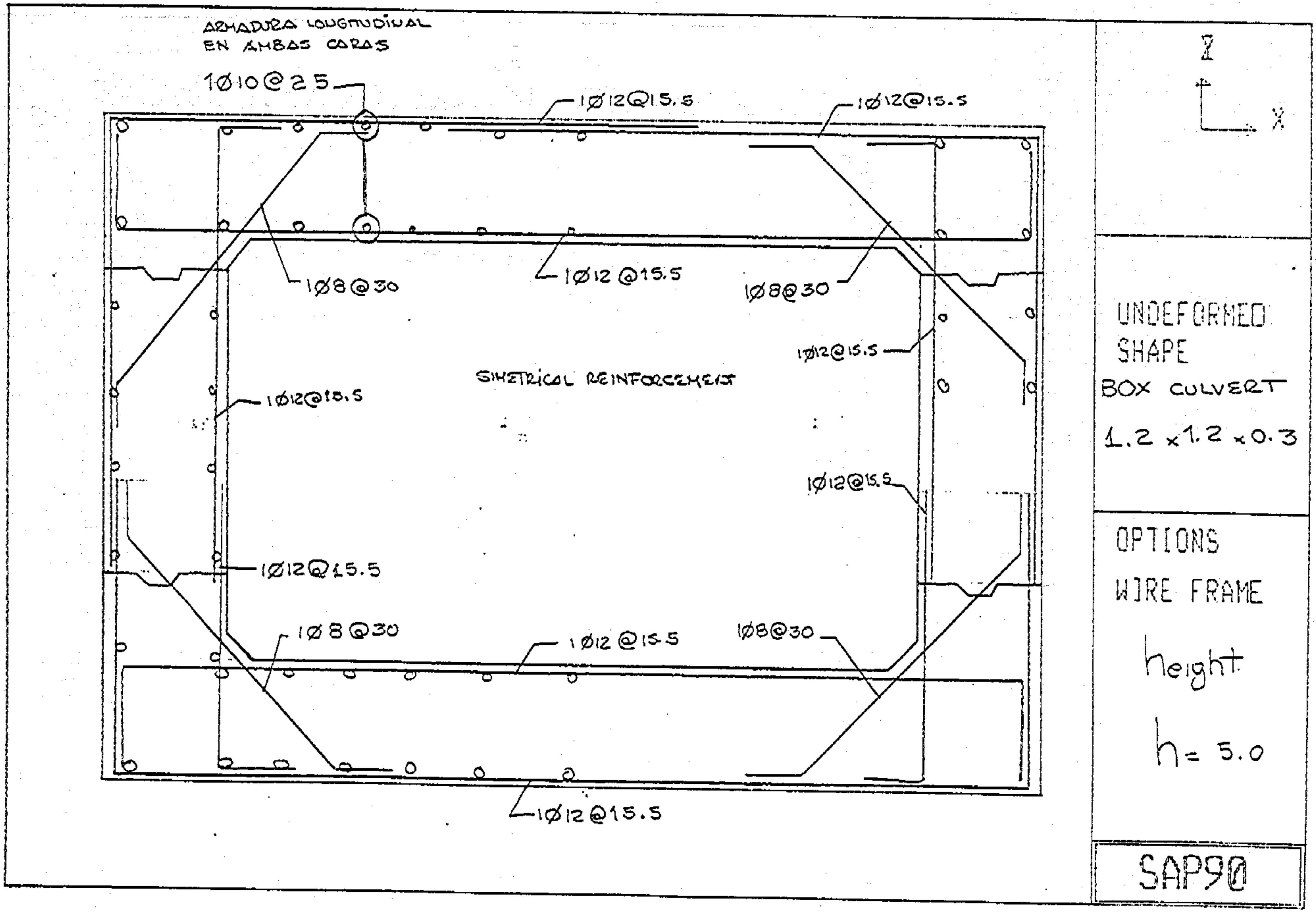
$V_U = 7.197$ ton

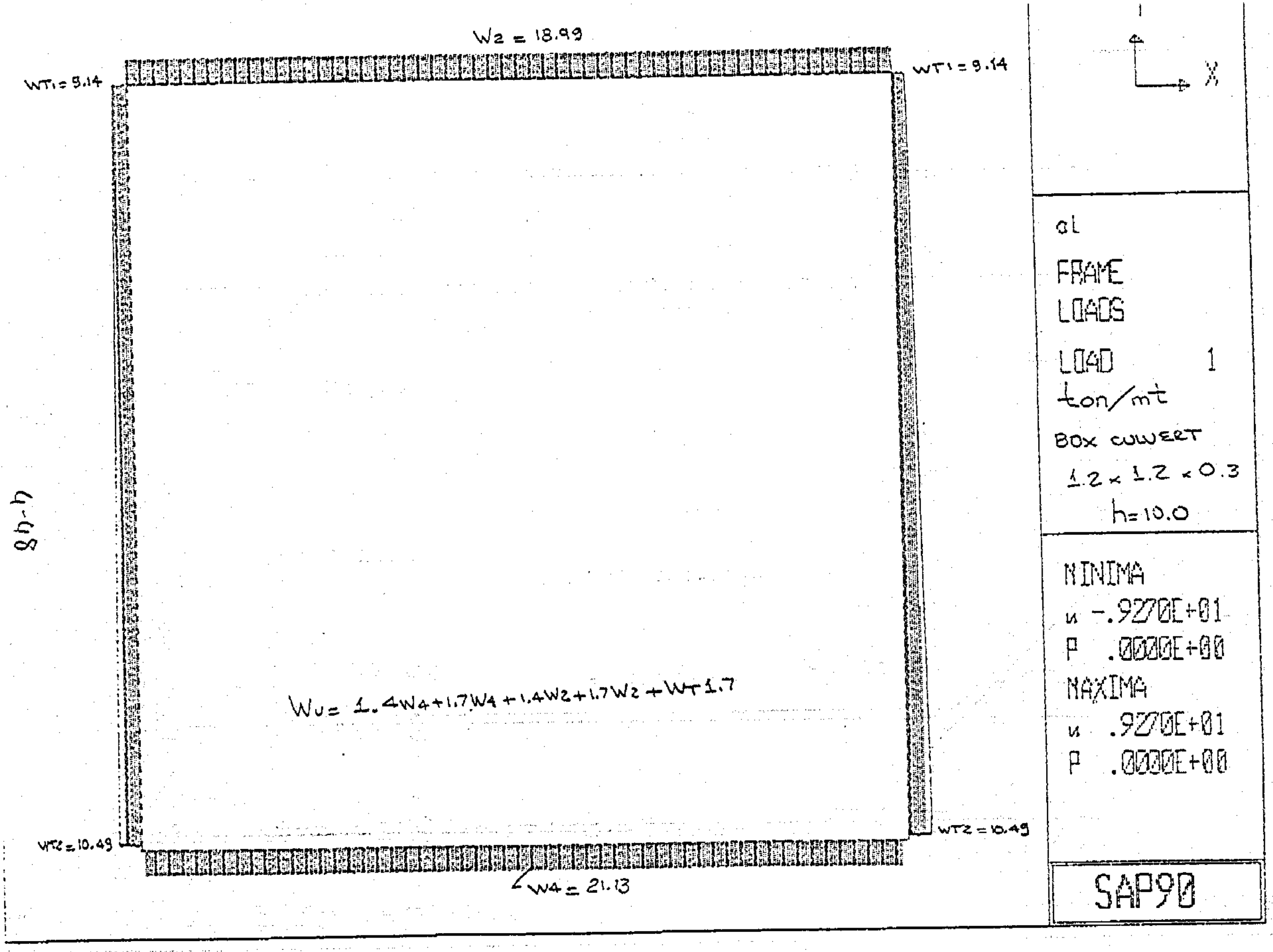
$V_c = 0.85 \cdot 0.53 \sqrt{180} \times 100 \times 23 = 13.901.41$ Kg. $\qquad V_c > V_U$ "OK"

$V_U = 13.899$ ton

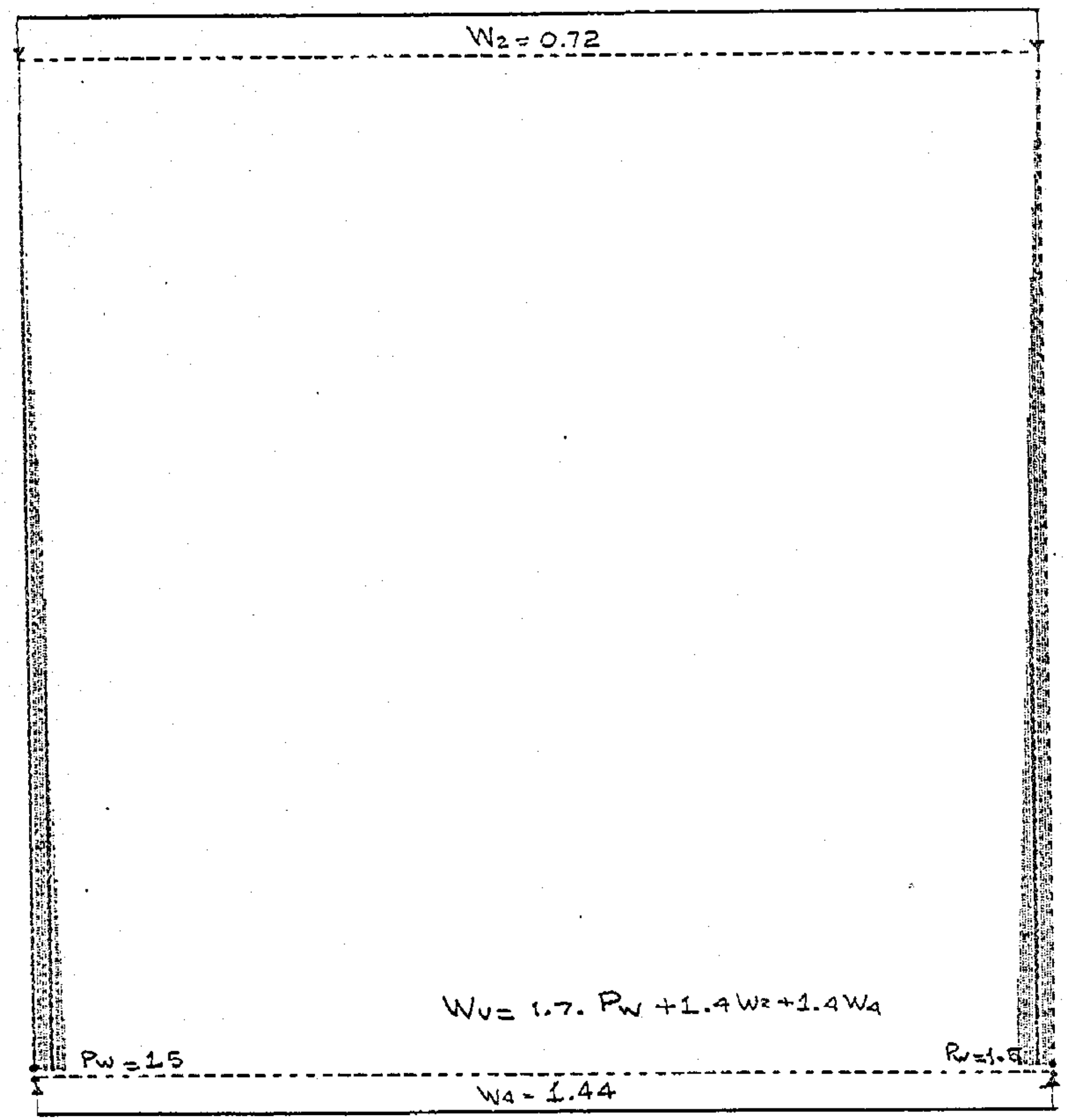
NO REQUIRED STIRRUPS

Revision	Checked by / Revisado por	Date (Fecha)	Approved by: / Aprobado por:	Date (Fecha)	Revision	Checked by: / Revisado por	Date (fecha)	Approbed by: / Aprobado por:	Date (Fecha) / Aprobado por:


ARMADURA LONGITUDINAL
EN AMBAS CARAS
1Ø10@25
1Ø12@15.5
1Ø12@15.5
1Ø8@30
1Ø12@15.5
1Ø8@30
1Ø12@15.5
SIMETRICAL REINFORCEMENT
1Ø12@15.5
1Ø12@15.5
1Ø12@15.5
1Ø8@30
1Ø12@15.5
1Ø8@30
1Ø12@15.5
Y
X
UNDEFORMED
SHAPE
BOX CULVERT
1.2 x 1.2 x 0.3
OPTIONS
WIRE FRAME
height
h = 5.0
SAP90



W2 = 18.99
WT1 = 5.14
WT1 = 9.14
WT2 = 10.49
WT2 = 10.49
W4 = 21.13
Wu = 1.4W4 + 1.7W4 + 1.4W2 + 1.7W2 + WT 1.7
Y
X
al
FRAME
LOADS
LOAD 1
ton/mt
BOX CULVERT
1.2 x 1.2 x 0.3
h = 10.0
MINIMA
u -.9270E+01
P .0000E+00
MAXIMA
u .9270E+01
P .0000E+00
SAP90



Y
X

al
FRAME
LOADS
LOAD 2
ton/mt

MINIMA
M .1500E+01
P .0000E+00
MAXIMA
M .1500E+01
P .0000E+00

SAP90

```
  ,=0
  Y=0
  Y=0

0,0,1,1,1,0

C=0100000

,1,50,0,1.49                 sw 120
,0,0,1,4900,1.50             sw 
,=10.49,0,1.4900,-9.14       sw 
,-0.14,0,1.4900,-10.49       sw 
                LP=1.0
        MSL=6.0,6
        nsl=1,2
        nsl=9.0,7

        nsl=2,4,0.5

,1,4,0,1.7
```

STRUCTURAL ANALYSIS PROGRAM

VERSION 5.40

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SAP90

FRAME ELEMENT FORCES

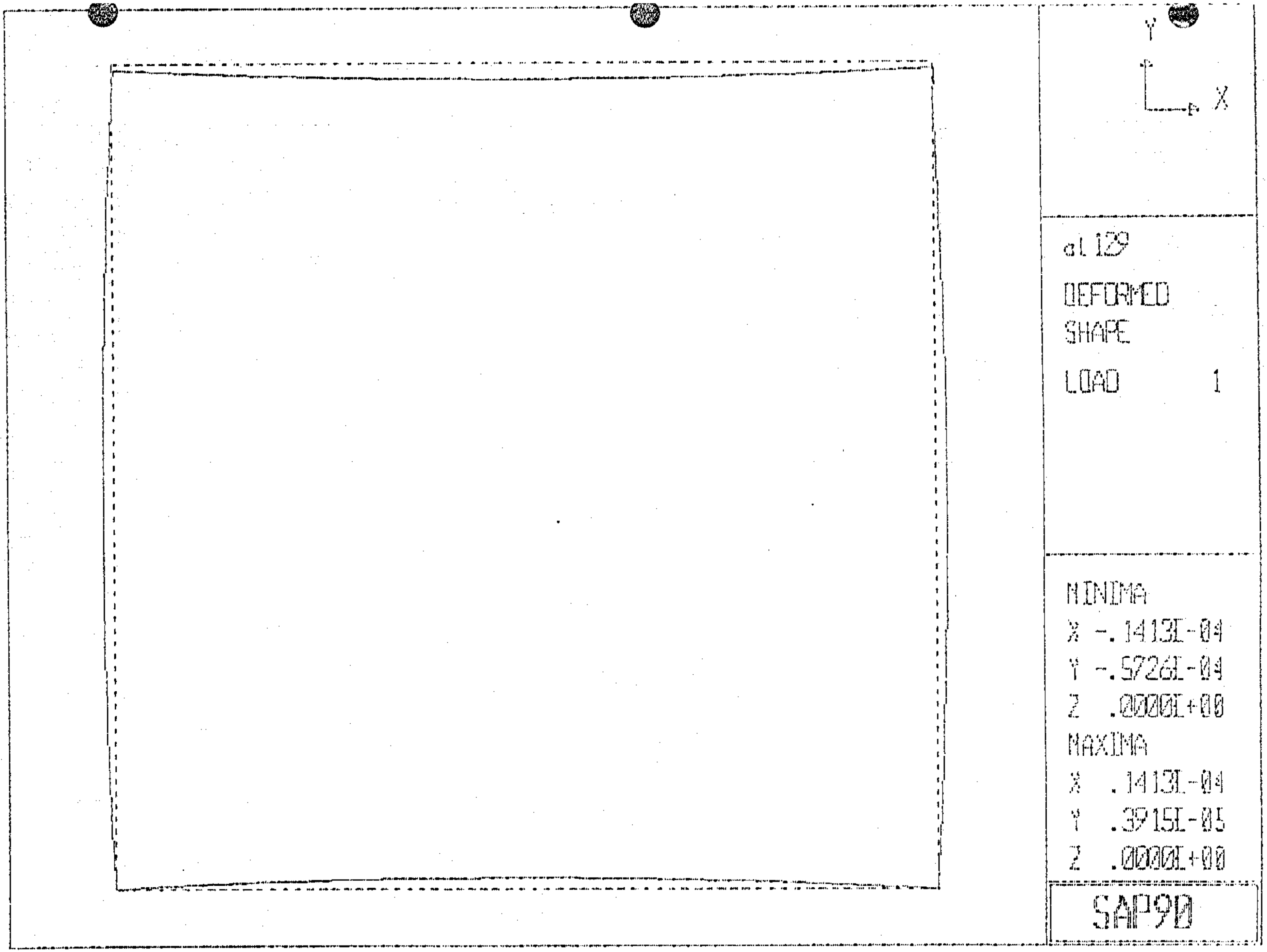
ELT ID	LOAD COMB	DIST ENDI	1-2 PLANE SHEAR	1-2 PLANE MOMENT	AXIAL FORCE	1-3 PLANE SHEAR	1-3 PLANE MOMENT	A
1								
	1	.000			-24.050			
		.000	12.993	-4.992				
		.010	12.993	-4.862				
		.010			-24.050			
	2	.000			-.756			
		.000	-1.094	-.191				
		.010	-1.094	-.202				
		.010			-.756			
2								
	1	.000			-24.050			
		.000	12.993	-4.862				
		.753	.000	-.025				
		1.490	-11.868	-4.449				
		1.490			-24.050			
	2	.000			-.756			
		.000	-1.094	-.202				
		.520	.000	-.467				
		1.490	.805	.054				
		1.490			-.756			
3								
	1	.000			-11.868			
		.000	24.050	-4.449				
		.750	.000	4.569				
		1.500	-24.050	-4.449				
		1.500			-11.868			
	2	.000			.805			
		.000	.756	.054				
		.750	.000	.338				
		1.500	-.756	.054				
		1.500			.805			

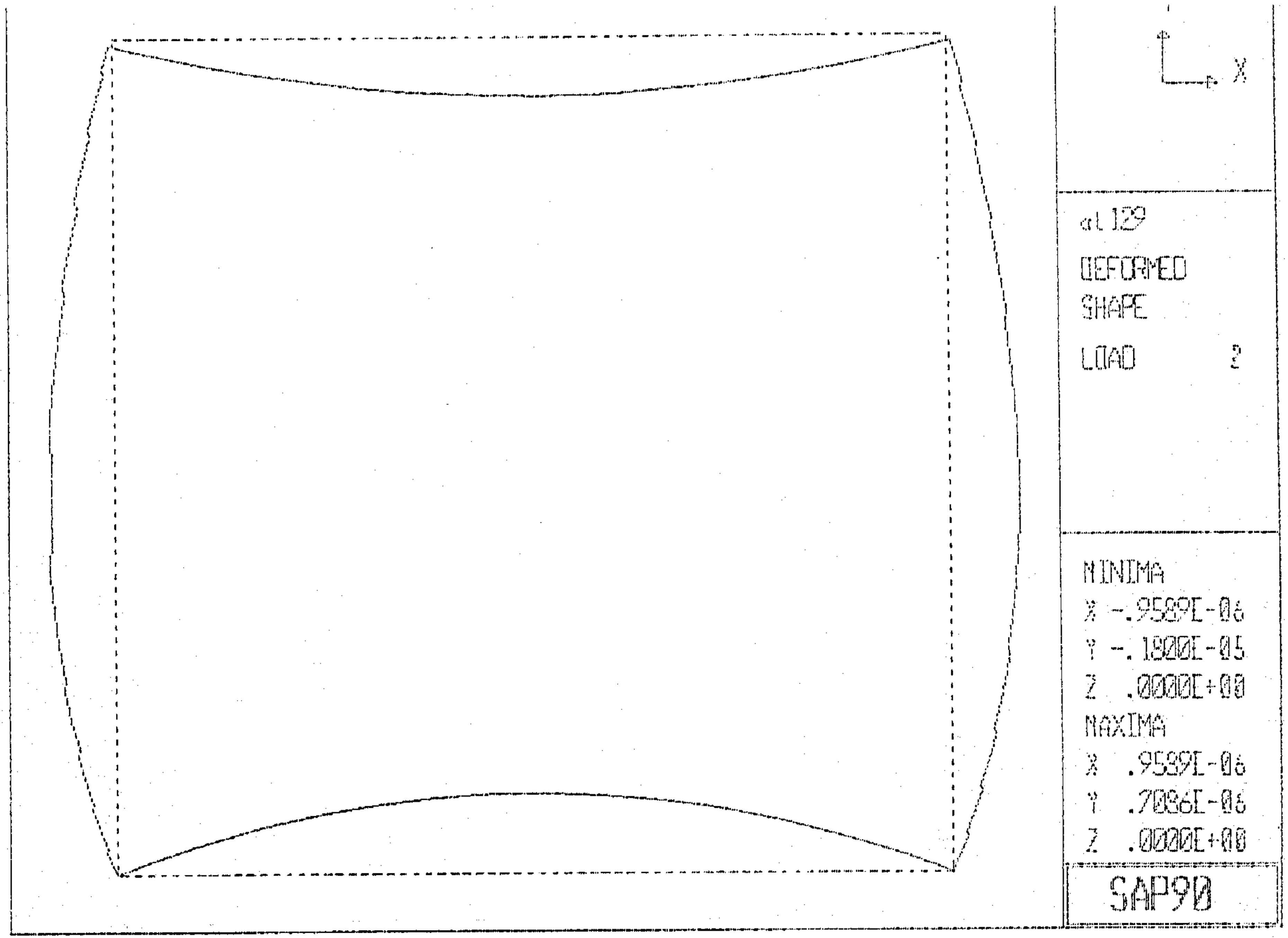
	.000	12.[illegible]	[illegible]	
	[illegible]	[illegible]	[illegible]	
	[illegible]	-12.[illegible]	-4.[illegible]	
	1.[illegible]			[illegible]
2	.000			[illegible]
	[illegible]	[illegible]	[illegible]	
	[illegible]	[illegible]	[illegible]	
	[illegible]	1.[illegible]	[illegible]	
	[illegible]			[illegible]

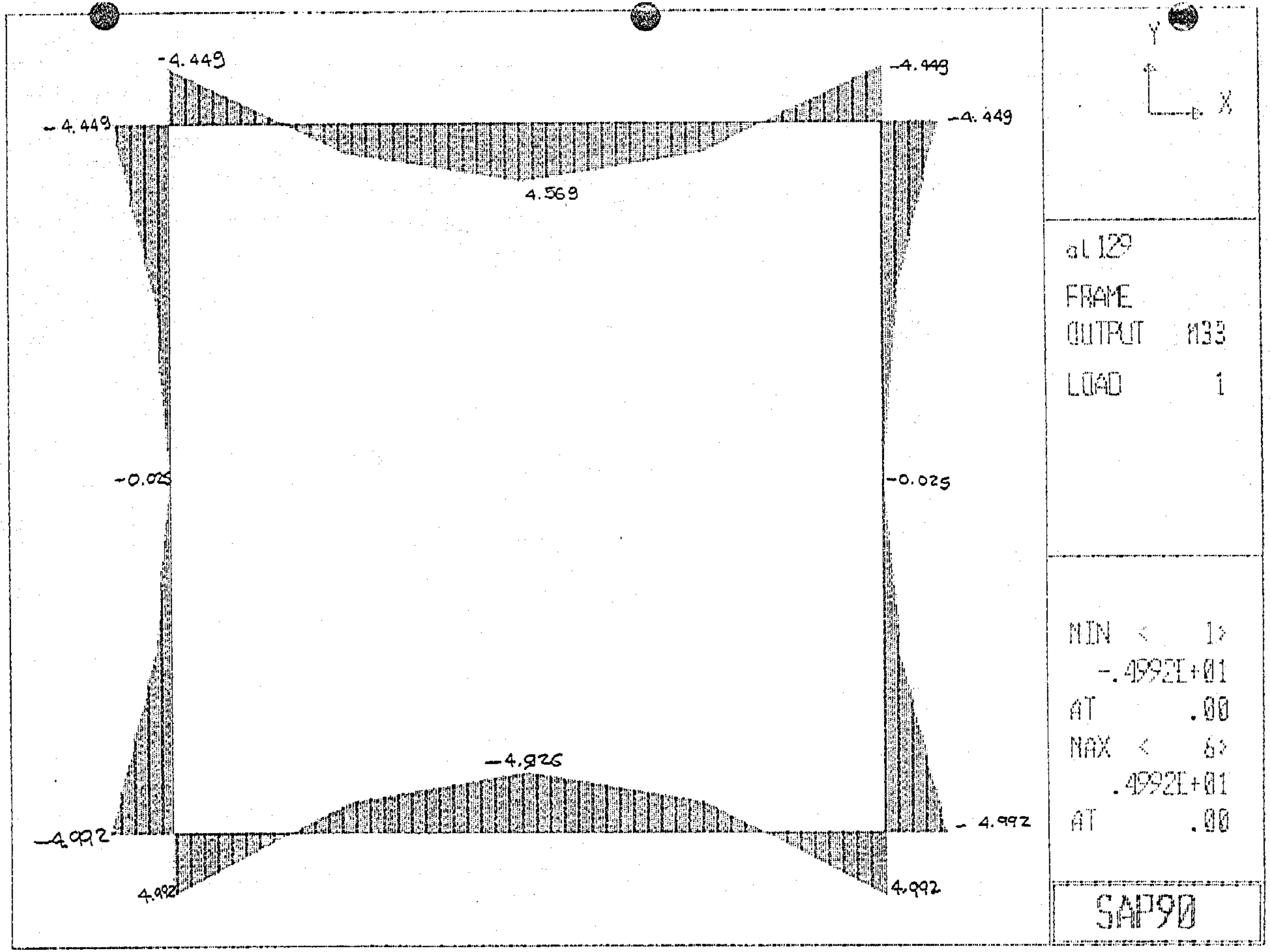
[illegible]

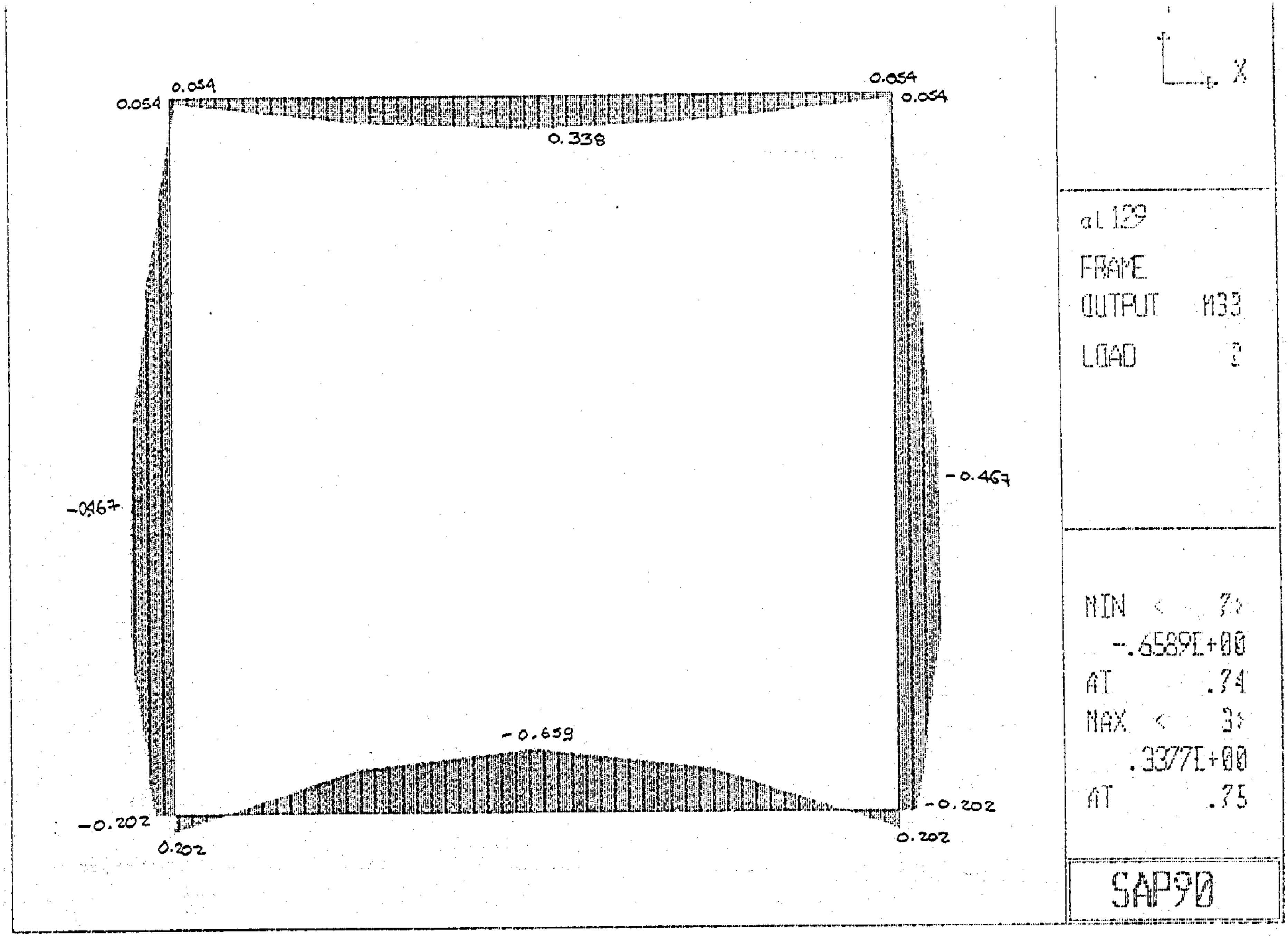
[illegible] F O R C E S

LOAD CASE	POINT (FT)	SHEAR	MOMENT	TORQUE	SHEAR	MOMENT	AXIAL FORCE
1	.000			-[illegible].000			
	.000	-12.99[illegible]	-4.8[illegible]				
	.010	-12.995	-4.992				
	.010			-[illegible].000			
2	.000			-.756			
	.000	1.094	-.[illegible]				
	.010	1.094	-.191				
	.010			-.756			
1	.000			.000			
	.000	-26.102	4.992				
	.010	-26.102	4.731				
	.010			.000			
2	.000			.000			
	.000	-2.238	.191				
	.010	-2.238	.169				
	.010			.000			
1	.000			.000			
	.000	-26.102	4.731				
	.740	.000	-4.924				
	1.480	26.102	4.731				
	1.480			.000			
2	.000			.000			
	.000	-2.238	.169				
	.740	.000	-.659				
	1.480	2.238	.169				
	1.480			.000			
1	.000			.000			
	.000	26.102	4.731				
	.010	26.102	4.992				
	.010			.000			
2	.000			.000			
	.000	2.238	.169				
	.010	2.238	.191				
	.010			.000			

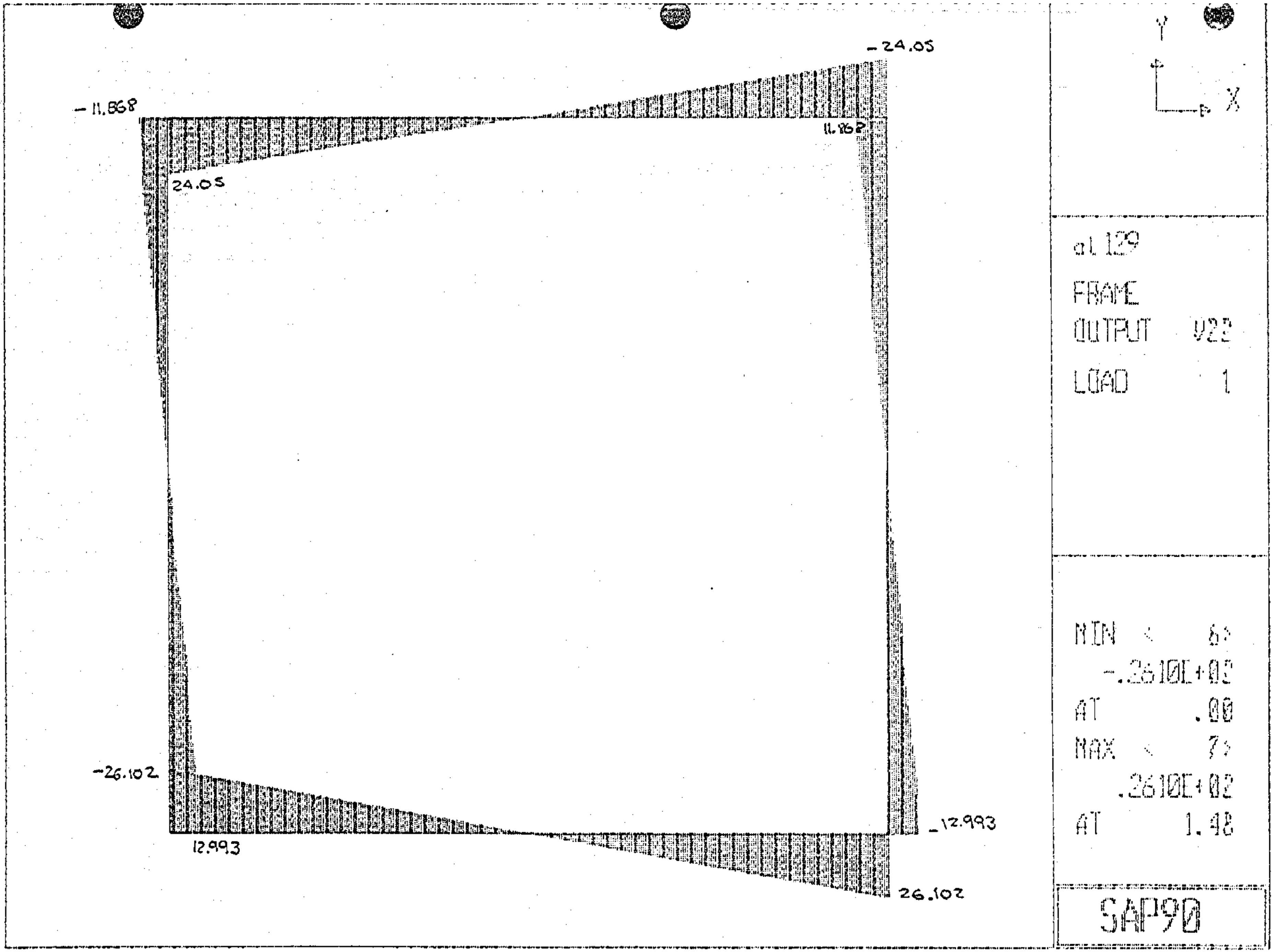

Y
X
al 129
DEFORMED
SHAPE
LOAD 1
MINIMA
X -.1413E-04
Y -.5726E-04
Z .0000E+00
MAXIMA
X .1413E-04
Y .3915E-05
Z .0000E+00
SAP90

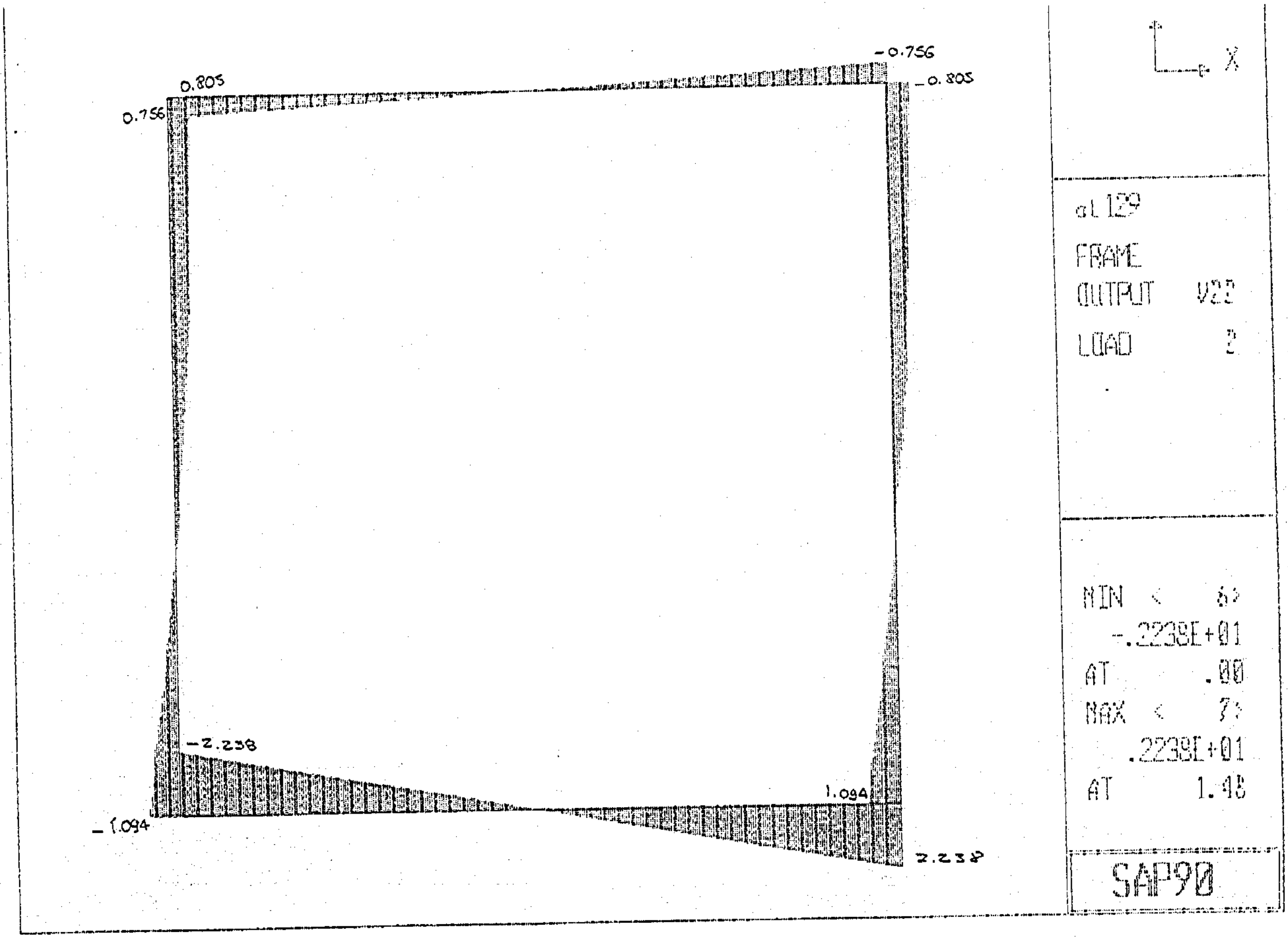

X
al 129
DEFORMED
SHAPE
LOAD 2
MINIMA
X -.9589E-06
Y -.1800E-05
Z .0000E+00
MAXIMA
X .9589E-06
Y .7086E-06
Z .0000E+00
SAP90


Y
X
-4.449
-4.449
-4.449
-4.449
4.569
-0.025
-0.025
-4.926
-4.992
-4.992
4.992
4.992
al129
FRAME
OUTPUT M33
LOAD 1
MIN < 1>
-.4992E+01
AT .00
MAX < 6>
.4992E+01
AT .00
SAP90



0.054
0.054
0.054
0.054
0.338
-0.467
-0.467
-0.659
-0.202
0.202
-0.202
0.202
X
al 129
FRAME
OUTPUT M33
LOAD 2
MIN < 7>
-.6589E+00
AT .74
MAX < 3>
.3377E+00
AT .75
SAP90


Y
X
al 129
FRAME
OUTPUT V22
LOAD 1
MIN < 6>
-.2610E+02
AT .00
MAX < 7>
.2610E+02
AT 1.48
SAP90
-11.868
-24.05
11.868
24.05
-26.102
12.993
-12.993
26.102


0.805
0.756
-0.756
-0.805
-2.238
-1.094
1.094
2.238
X
al 129
FRAME
OUTPUT V22
LOAD 2
MIN < 6>
-.2238E+01
AT .00
MAX < 7>
.2238E+01
AT 1.48
SAP90

– DATA FOR REINFORCEMENT.

$f'_c = 180$ Kg/cm²

$f_y = 4200$ Kg/cm²

$b = 100$ cm

$d = 23$ cm

$r = 7$ cm

$M_u = 4.992$ t-m

$$A_s = \frac{4.992 \times 10^5}{0.9 \cdot 4.200\,(23-2.0)} = 6.288$$

$$a = \frac{6.288 \times 4200}{0.85 \times 180 \times 100} = 3.10$$

$$A_s = \frac{4.992 \times 10^5}{0.9 \cdot 4200\,(21.5)} = 6.142 \qquad \rho_{min} = \frac{14}{f_y} = \frac{14}{4{,}200} = 0.00333$$

$A_s = 7.66$ cm² ⇒ 8Ø12 ⇒ 1Ø12 @ 15.5

CALCULATION WITH PROGRAM

– SHEAR STRESS CHECK

$V_u = 12.993$ ton

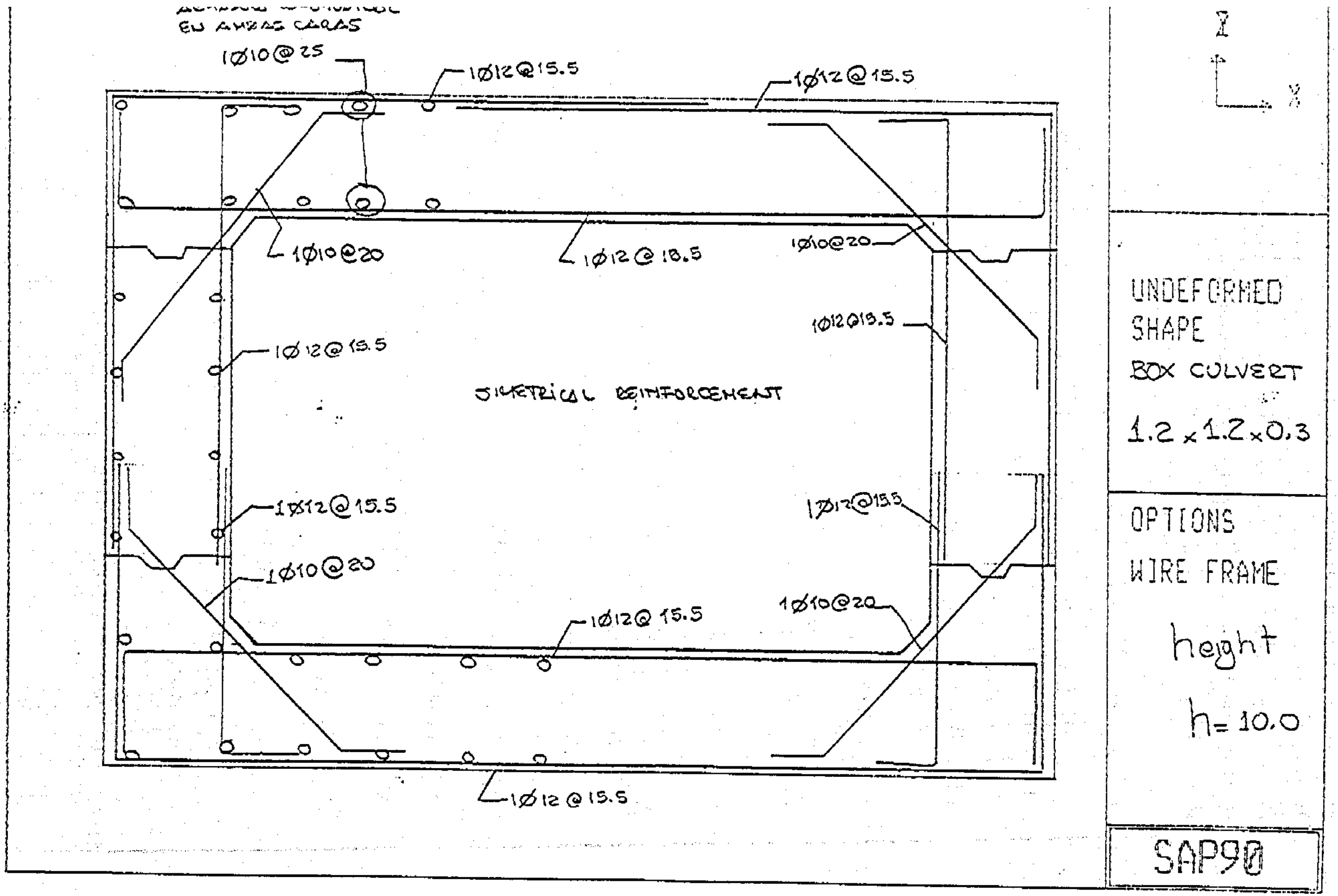
$V_c = 0.85 \times 0.53 \sqrt{180} \times 100 \times 23 = 13.901.41$ Kg. $\qquad V_c > V_u$.

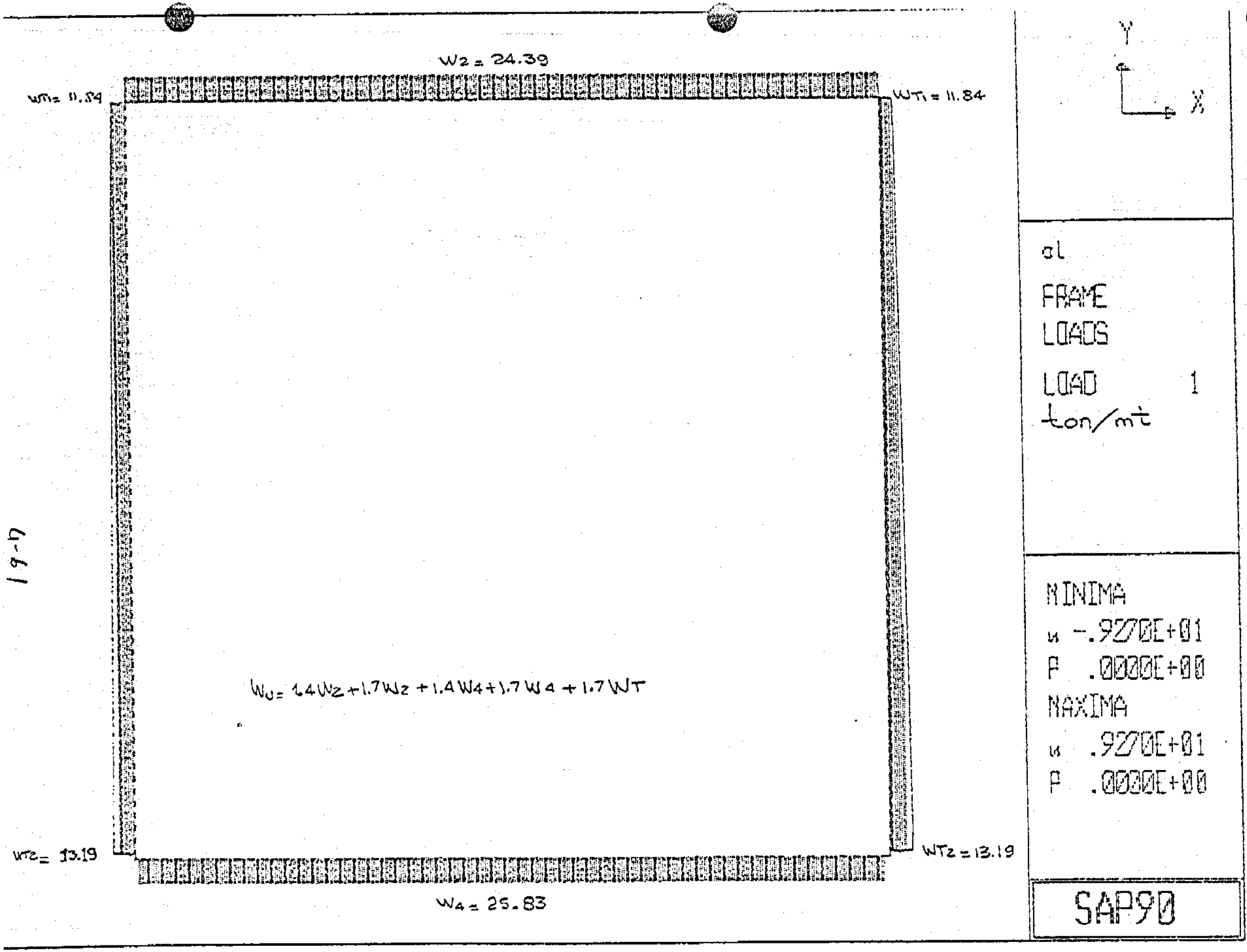
$V_u = 26.102$ ton

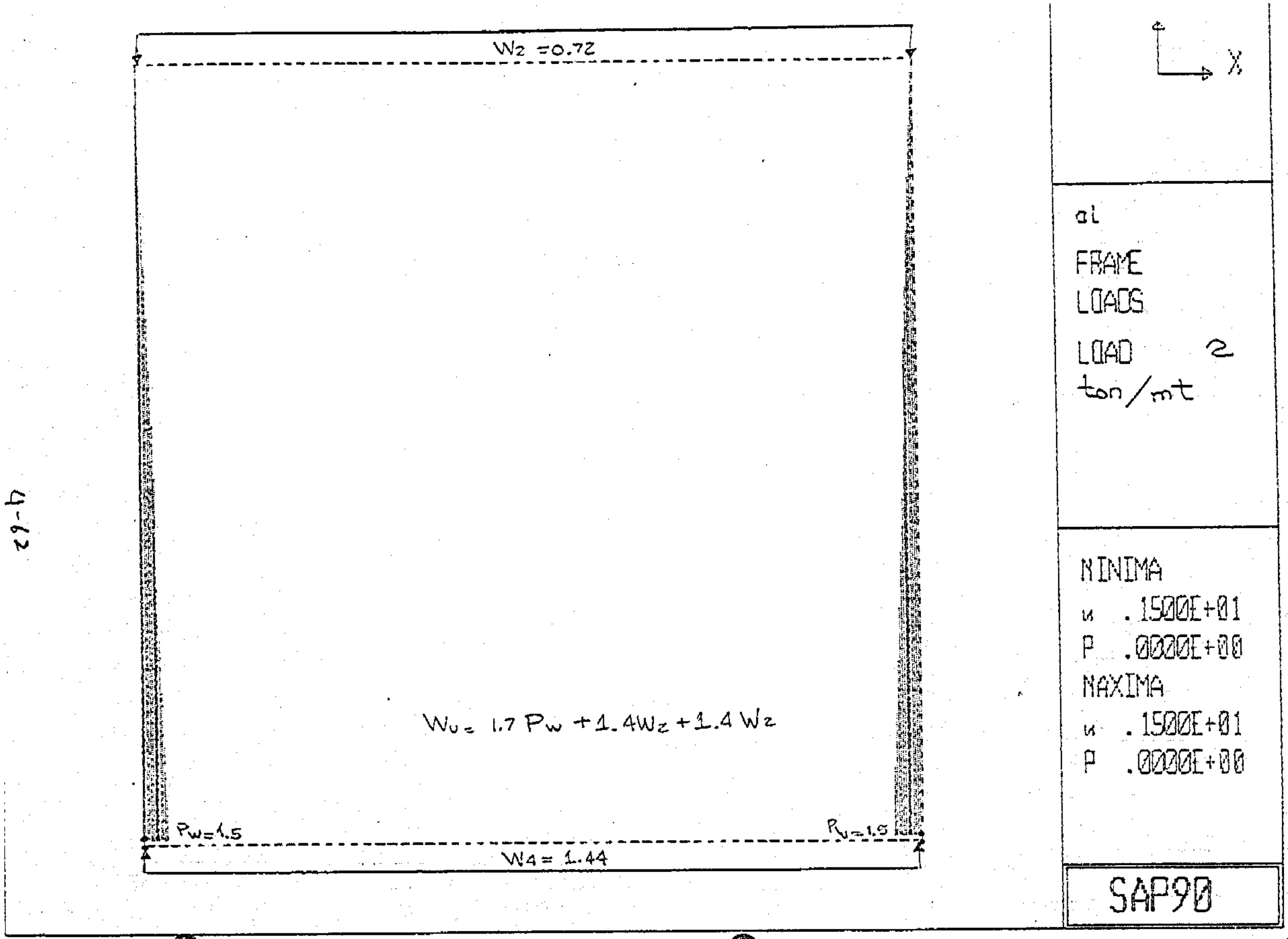
$V_s = 26.102 - 13.9 = 12.20$ ton

$$A_v = \frac{12.20 \times 10^3}{0.85 \times 4200 \times \sin 45} = 4.83 \text{ cm}^2 \Rightarrow 1Ø10 @ 20$$

Revision	Checked by / Revisado por	Date (Fecha)	Approved by: / Aprobado por:	Date (Fecha)	Revision	Checked by: / Revisado por	Date (fecha)	Approbed by: / Aprobado por:	Date (Fecha) / Aprobado por:


EN AMBAS CARAS
1Ø10@25
1Ø12@15.5
1Ø12@15.5
1Ø10@20
1Ø12@15.5
1Ø10@20
1Ø12@15.5
1Ø12@15.5
SIMETRICAL REINFORCEMENT
1Ø12@15.5
1Ø12@15.5
1Ø10@20
1Ø10@20
1Ø12@15.5
1Ø12@15.5
UNDEFORMED
SHAPE
BOX CULVERT
1.2 x 1.2 x 0.3
OPTIONS
WIRE FRAME
height
h= 10.0
SAP90


W2 = 24.39
WT1 = 11.84
WT1 = 11.84
WU = 1.4W2 + 1.7W2 + 1.4W4 + 1.7W4 + 1.7WT
WT2 = 13.19
WT2 = 13.19
W4 = 25.83
Y
X
al
FRAME
LOADS
LOAD 1
ton/mt
MINIMA
u -.9270E+01
P .0000E+00
MAXIMA
u .9270E+01
P .0000E+00
SAP90


W2 =0.72
Wu = 1.7 Pw + 1.4W2 + 1.4 W2
Pw=1.5
Pw=1.5
W4 = 1.44
X
al
FRAME
LOADS
LOAD 2
ton/mt
MINIMA
w .1500E+01
P .0000E+00
MAXIMA
w .1500E+01
P .0000E+00
SAP90

```
[illegible]
SYSTEM
L=1

JOINTS
1   X=0   Y=[illegible]   Z=[illegible]
2           Y=[illegible]
3           Y=1.50
4   X=1.50  Y=1.50
5   X=1.50  Y=.01
6   X=.01   Y=0
7   X=1.49  Y=0
8   X=1.50  Y=0

RESTRAINTS
1 8 1   R=0,0,1,1,1,0
1 8 7   R=1,1,1,1,1,0

FRAME
NM=2 NL=9
1 SH=R T=.30,1 E=2100000.
2 SH=R T=.30,1
1 WL=0,-23.67      :w2
2 WL=0,-0.72       :w7
3 WL=0,23.67       :w4
4 WL=0,2.16        :w4
5 WL=0             :w3
6 TRAP=0,11.50,0,1.49              :pw izq.
7 TRAP=0,0,0,1.490,1.30            :pw der.
8 TRAP=0,-13.19,0,1.4900,-11.64    :wt izq.
9 TRAP=0,-11.64,0,1.4900,-13.19.   :wt der.
1 1 2   M=1          LP=1,0
2 2 3           NSL=3,0,6
3 3 4           nsl=1,2
4 4 5           nsl=2,0,7
5 5 8
6 1 6   M=1
7 6 7           nsl=5,4,0.3
8 7 8

COMBO
1 C=1.7,1.4,0,1.7
2 C=0,1.4,1.7
```

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STRUCTURAL ANALYSIS PROGRAMS

VERSION 5.40

SANTAPULLA 1 2X1.2 h=16.0

F R A M E   E L E M E N T   F O R C E S

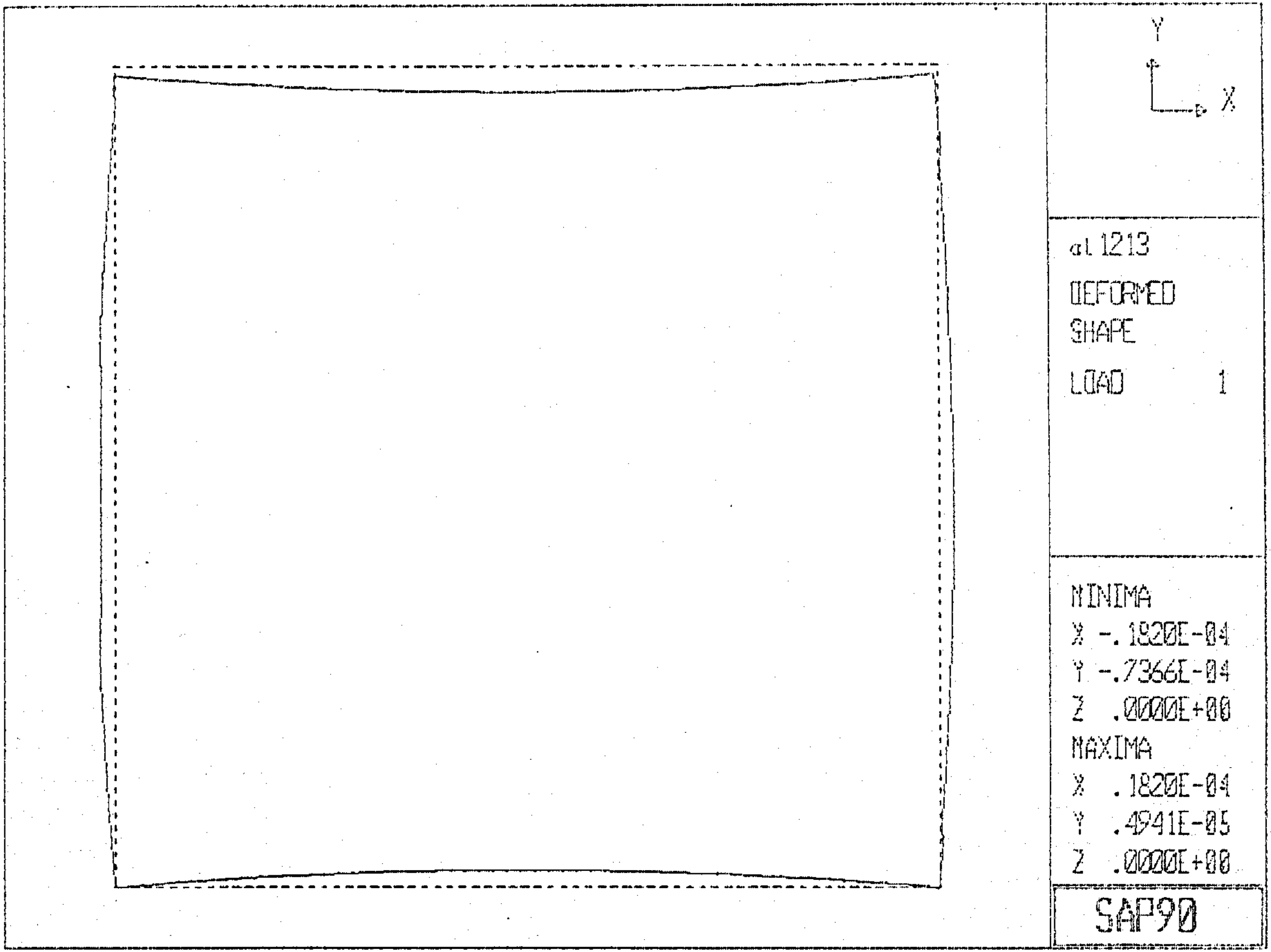
ELT ID	LOAD COND	DIST ENDI	1-2 PLANE SHEAR	1-2 PLANE MOMENT	AXIAL FORCE	1-3 PLANE SHEAR	1-3 PLANE MOMENT	AXIAL TORQ
1								
	1	.000			-30.935			
		.000	16.414	-6.305				
		.010	16.414	-6.141				
		.010			-30.935			
	2	.000			-.756			
		.000	-1.094	-.191				
		.010	-1.094	-.202				
		.010			-.756			
2								
	1	.000			-30.935			
		.000	16.414	-6.141				
		.751	.000	-.029				
		1.490	-15.287	-5.726				
		1.490			-30.935			
	2	.000			-.756			
		.000	-1.094	-.202				
		.520	.000	-.467				
		1.490	.805	.054				
		1.490			-.756			
3								
	1	.000			-15.287			
		.000	30.935	-5.726				
		.750	-.000	5.875				
		1.500	-30.935	-5.726				
		1.500			-15.287			
	2	.000			.805			
		.000	.756	.054				
		.750	.000	.338				
		1.500	-.756	.054				
		1.500			.805			
4								

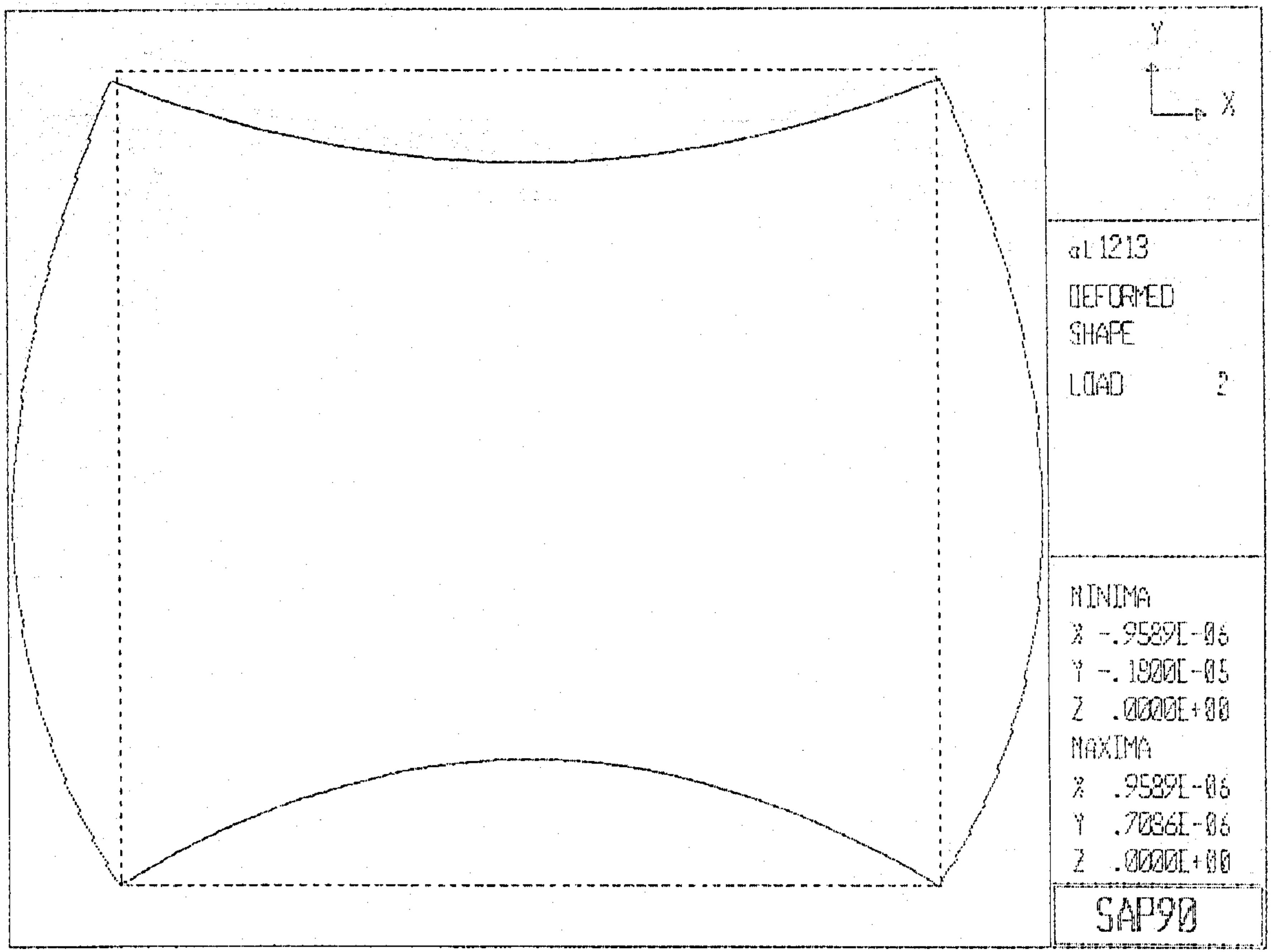
[illegible]	.000			[illegible]
	.000	[illegible]	[illegible]	
	.740	.000	[illegible]	
	1.480	[illegible]	[illegible]	
	1.480			[illegible]
[illegible]	.000			[illegible]
	.000	[illegible]	[illegible]	
	.740	.000	[illegible]	
	1.480	[illegible]	[illegible]	
	1.480			[illegible]

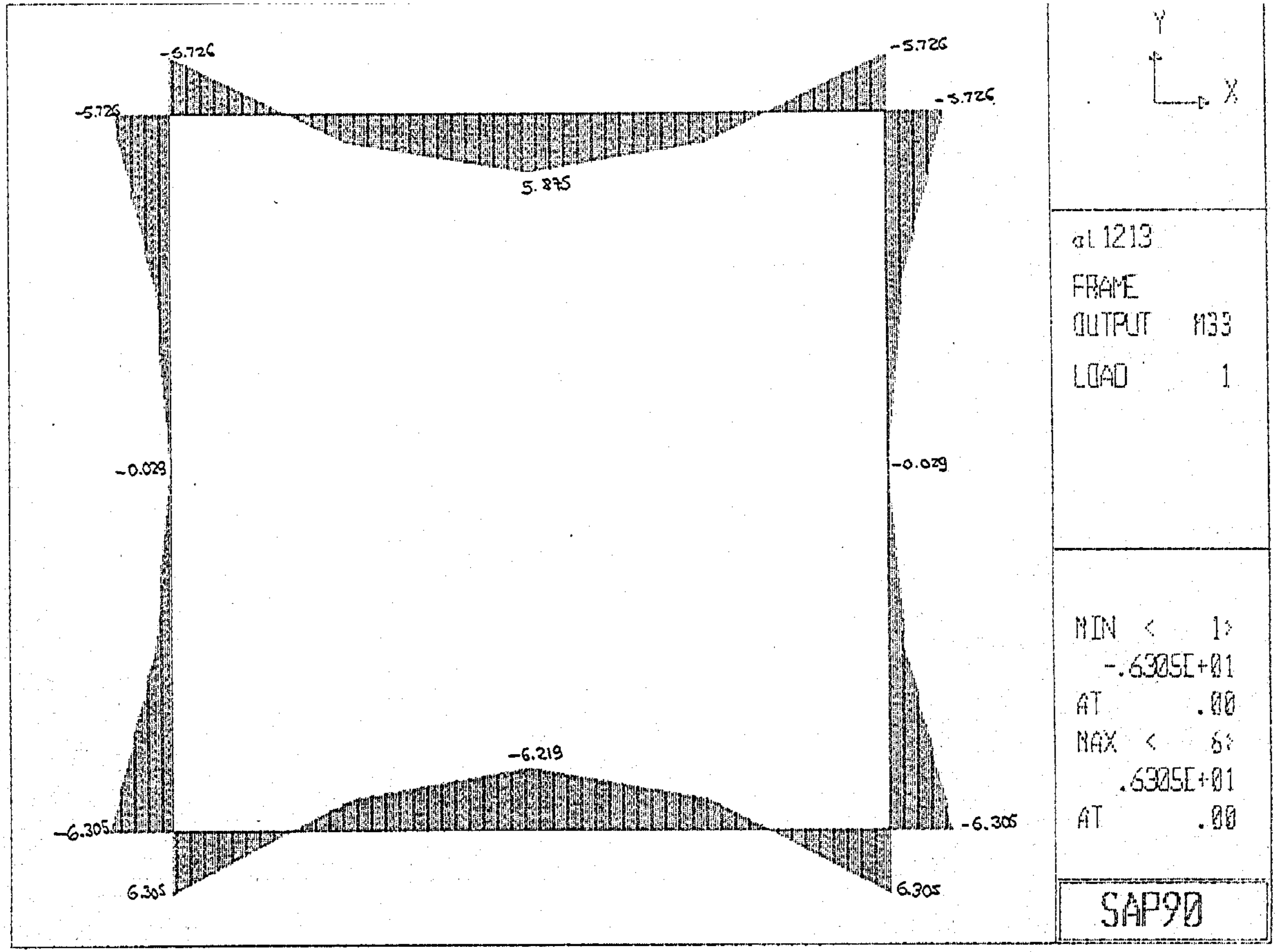
[illegible]

F R A M E E L E M E N T F O R C E S

ELT LOAD ID COMB	DIST ENDI	1-2 PLANE SHEAR	1-2 PLANE MOMENT	AXIAL FORCE	1-3 PLANE SHEAR	1-3 PLANE MOMENT	AXIAL TORQ
[illegible]							
1	.000			-30.935			
	.000	16.414	-6.141				
	.010	-16.414	-6.305				
	.010			30.935			
2	.000			-.756			
	.000	1.094	.202				
	.010	1.094	-.191				
	.010			-.756			
6							
1	.000			.000			
	.000	-32.958	6.305				
	.010	-32.958	5.976				
	.010			.000			
2	.000			.000			
	.000	-2.238	.191				
	.010	-2.238	.169				
	.010			.000			
7							
1	.000			.000			
	.000	-32.958	5.976				
	.740	.000	-6.219				
	1.480	32.958	5.976				
	1.480			.000			
2	.000			.000			
	.000	-2.238	.169				
	.740	.000	-.659				
	1.480	2.238	.169				
	1.480			.000			
8							
1	.000			.000			
	.000	32.958	5.976				
	.010	32.958	6.305				
	.010			.000			
2	.000			.000			
	.000	2.238	.169				
	.010	2.238	.191				
	.010			.000			


Y
X
al 1213
DEFORMED
SHAPE
LOAD 1
MINIMA
X -.1820E-04
Y -.7366E-04
Z .0000E+00
MAXIMA
X .1820E-04
Y .4941E-05
Z .0000E+00
SAP90


Y
X
al 1213
DEFORMED
SHAPE
LOAD 2
MINIMA
X -.9589E-06
Y -.1900E-05
Z .0000E+00
MAXIMA
X .9589E-06
Y .7086E-06
Z .0000E+00
SAP90


-5.726
-5.726
-5.726
-5.726
5.875
-0.029
-0.029
-6.219
-6.305
-6.305
6.305
6.305
Y
X
aL1213
FRAME
OUTPUT M33
LOAD 1
MIN < 1>
-.6305E+01
AT .00
MAX < 6>
.6305E+01
AT .00
SAP90

4-68

47