



UNIVERSIDAD NACIONAL AUTÓNOMA DE MÉXICO

FACULTAD DE INGENIERÍA

---

ANÁLISIS Y DISEÑO DE UNA ESTRUCTURA METALICA  
EDIFICIO DE PROCESO METIL MERCAPTANO

TRABAJO ESCRITO  
QUE PARA OBTENER EL TÍTULO DE  
INGENIERO CIVIL  
PRESENTA  
JESUS PONCE SANDOVAL



Universidad Nacional  
Autónoma de México



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## INTRODUCCION

El trabajo consiste en el análisis y diseño de una estructura metálica para un proceso definido dentro del complejo industrial, el cual se localiza en Cosoleacaque Ver.

La finalidad de esta planta es la producción de alimentos destinados a ~~las diferentes especies animales.~~

El edificio en estudio consta de 5 niveles principales en los cuales se localizan todos los equipos que componen esta estructura.

La cimentación de este edificio es a base de zapatas aisladas con trabes de liga en las dos direcciones principales, la solución estructural es a base de trabes de acero de alma llena apoyadas en columnas de acero, los entrepisos son de placa antiderrapante. Con el objeto de proporcionar mayor rigidez a la estructura bajo efectos de cargas sísmicas, se decidió contraventear el edificio en el sentido longitudinal con diagonales dispuestas en "V" invertida.

En el sentido transversal, los marcos son rígidos.

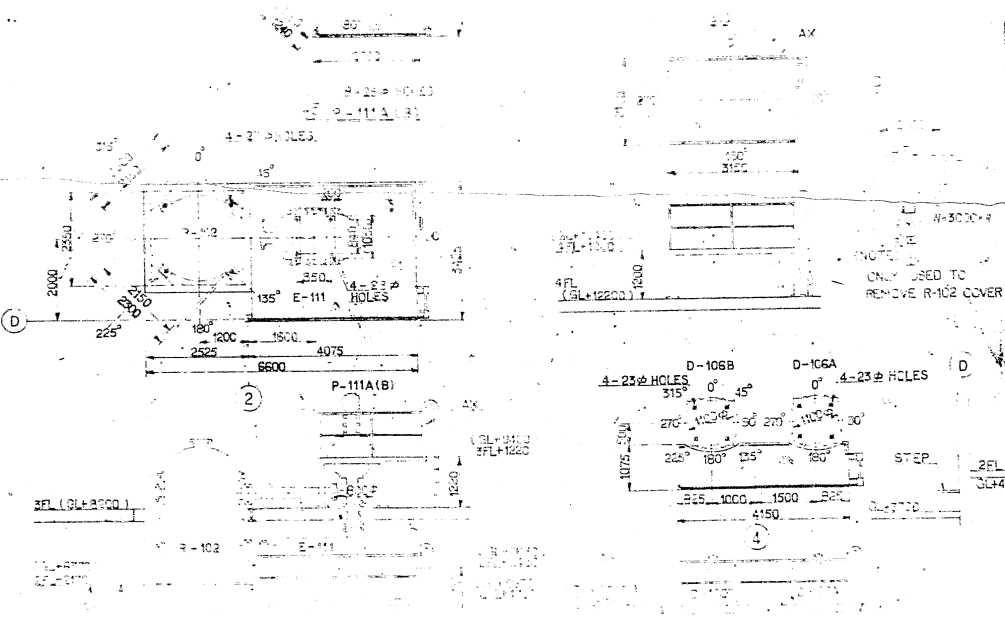
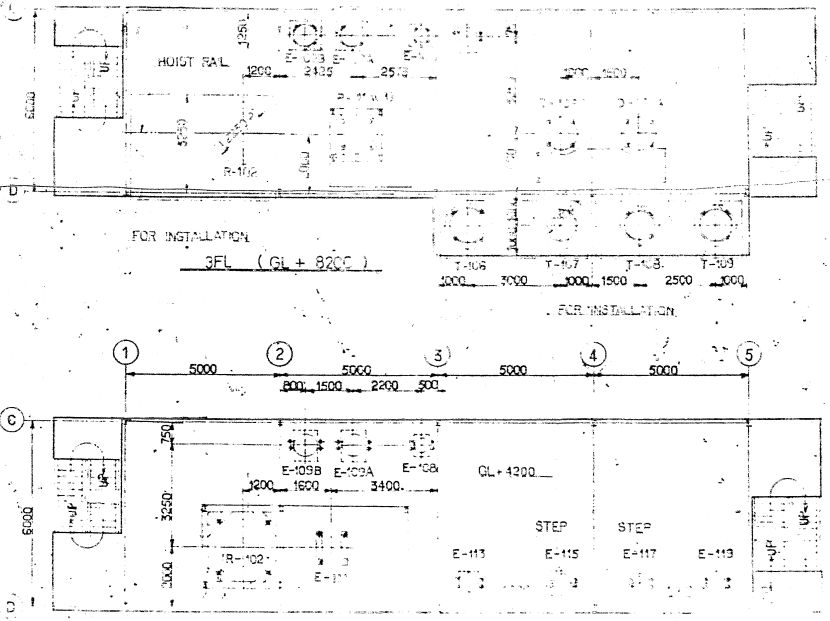
Los materiales que se usaron fueron los siguientes:

En trabes, columnas y contraventeo, acero estructural A-36.

Soldadura electrodos de la serie E-70XX en tornillos, acero A-307 para montaje.

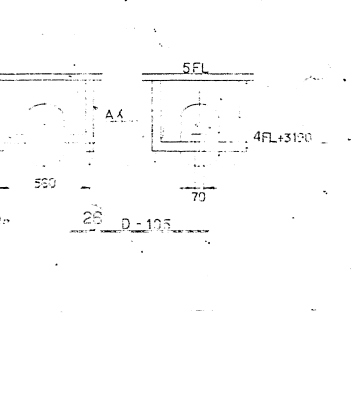
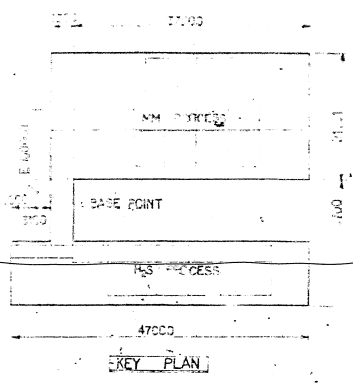
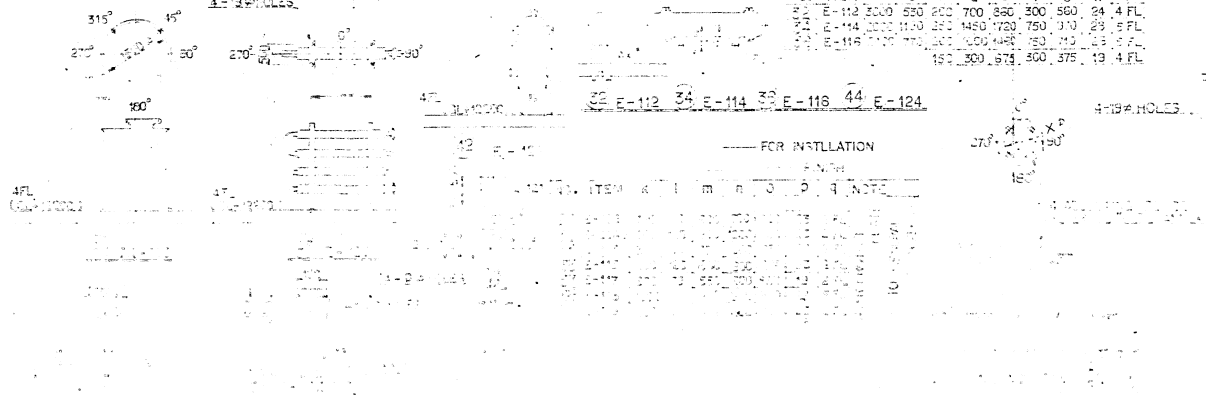
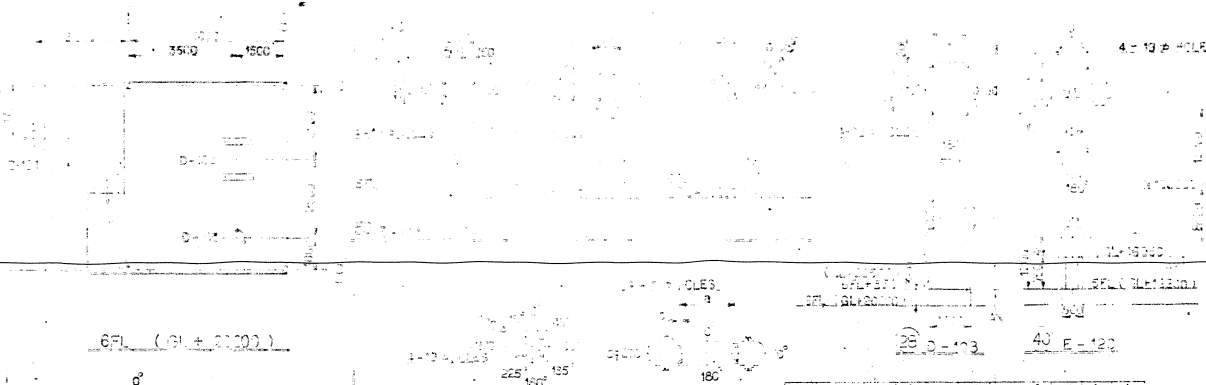
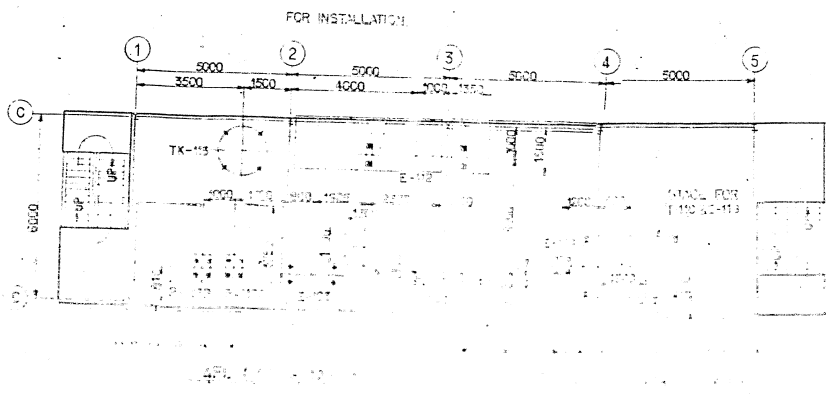
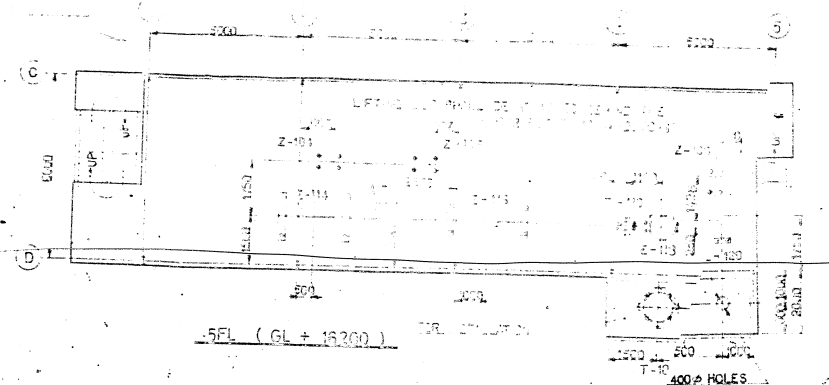
Para el diseño de los elementos de acero se siguieron los criterios del A.I.S.C.

A continuación se muestra el plano de la ingeniería básica con el cual se desarrollo todo el trabajo.



NO.	DESCRIPTION	EQUIP. LIST	QTY	UNIT	PRICE	AMOUNT	REMARKS
52	Z-106		1	0.2	0.25	0.5	5FL+800
53	Z-105		1	0.2	0.25	0.5	5FL+800
54	Z-104		1	0.2	0.25	0.5	5FL+800
55	TK-213		1	(1.0)	(4.8)	4.8	4FL+1000
56	T-107		1	0.15	0.2	0.3	5FL 475
57	T-106		1	3.0	4.5	13.5	GL+8200
58	T-105		1	3.5	5.0	17.5	GL+8200
59	T-104		1	2.3	4.2	10.0	GL+8200
60	T-103		1	3.1	4.2	13.0	GL+8200
61	R-102		1	0.2	15.9	3.18	2FL+550
62	P-113A B		141	0.47	0.49	1.0	4FL+450
63	P-111A B		141	3.35	3.35	4.7	4FL+450
64	E-124		1	0.8	0.8	1.0	4FL+675
65	E-123		1	0.8	0.8	1.5	4FL+1000
66	E-122		1	(0.6)	(1.0)	1.5	4FL+1100
67	E-121		1	1.9	2.0	3.0	3FL+290
68	E-120		1	1.3	1.5	2.0	5FL+1200
69	E-119		1	0.7	1.0	1.0	2FL+240
70	E-118		1	1.7	2.0	3.0	5FL+70
71	E-117		1	0.5	0.35	0.5	2FL+120
72	E-116		1	5.1	5.1	9.0	5FL+1450
73	E-115		1	0.9	1.0	1.5	2FL+150
74	E-114		1	10.0	13.0	16.5	5FL+170
75	E-113		1	0.8	0.85	1.5	2FL+150
76	E-112		1	3.4	3.8	5.5	4FL+850
77	E-111		1	(2.0)	(3.9)	4.0	4FL+3000
78	E-108A B		2	2.5	2.5	4.5	2FL+2200
79	E-108		1	1.2	1.2	2.0	2FL+2200
80	D-103		1	1.6	2.4	3.5	5FL+1750
81	D-102A B		2	(0.6)	(2.3)	3.0	3FL+1500
82	D-102		1	(0.05)	(0.05)	(0.5)	4FL+350
83	D-105		1	(0.1)	(0.25)	(0.5)	4FL+345

ED. ... (FOR CIVIL ENGINEERING)

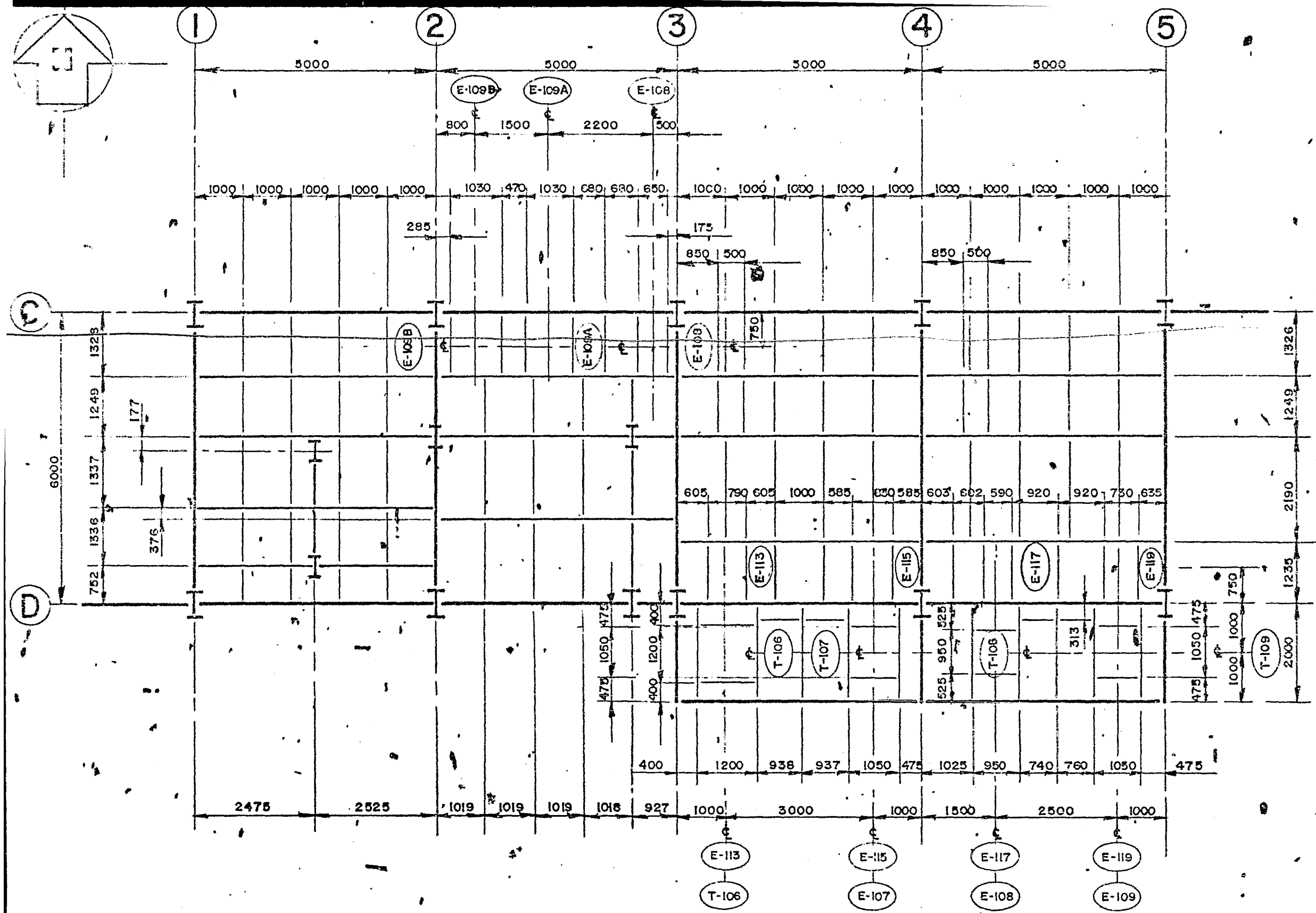


## CAPITULO 1

### ESTRUCTURACION

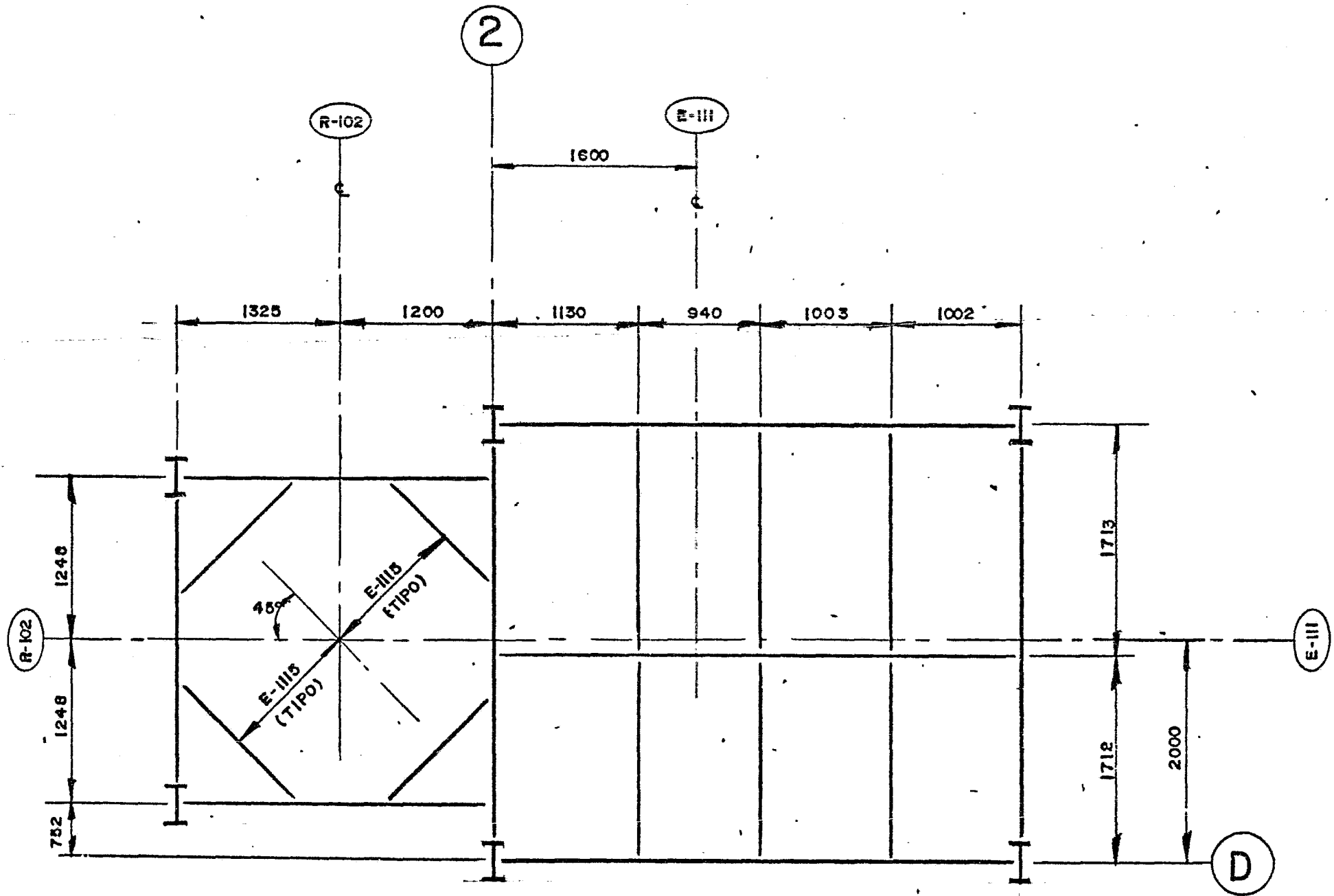
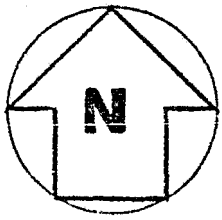
Dado que la estructura tiene tableros con dimensiones normales, se con-  
~~cluyó que una estructuración tradicional de trabes secundarias y prin-~~  
cipales sería la adecuada.

A continuación se muestran cada uno de los niveles con la localización de trabes. La finalidad de realizar una estructuración lo más homogénea posible es: Evitar un número elevado de perfiles en longitud y para que el taller elabore los planos de fabricación y montaje con el menor número de detalles.



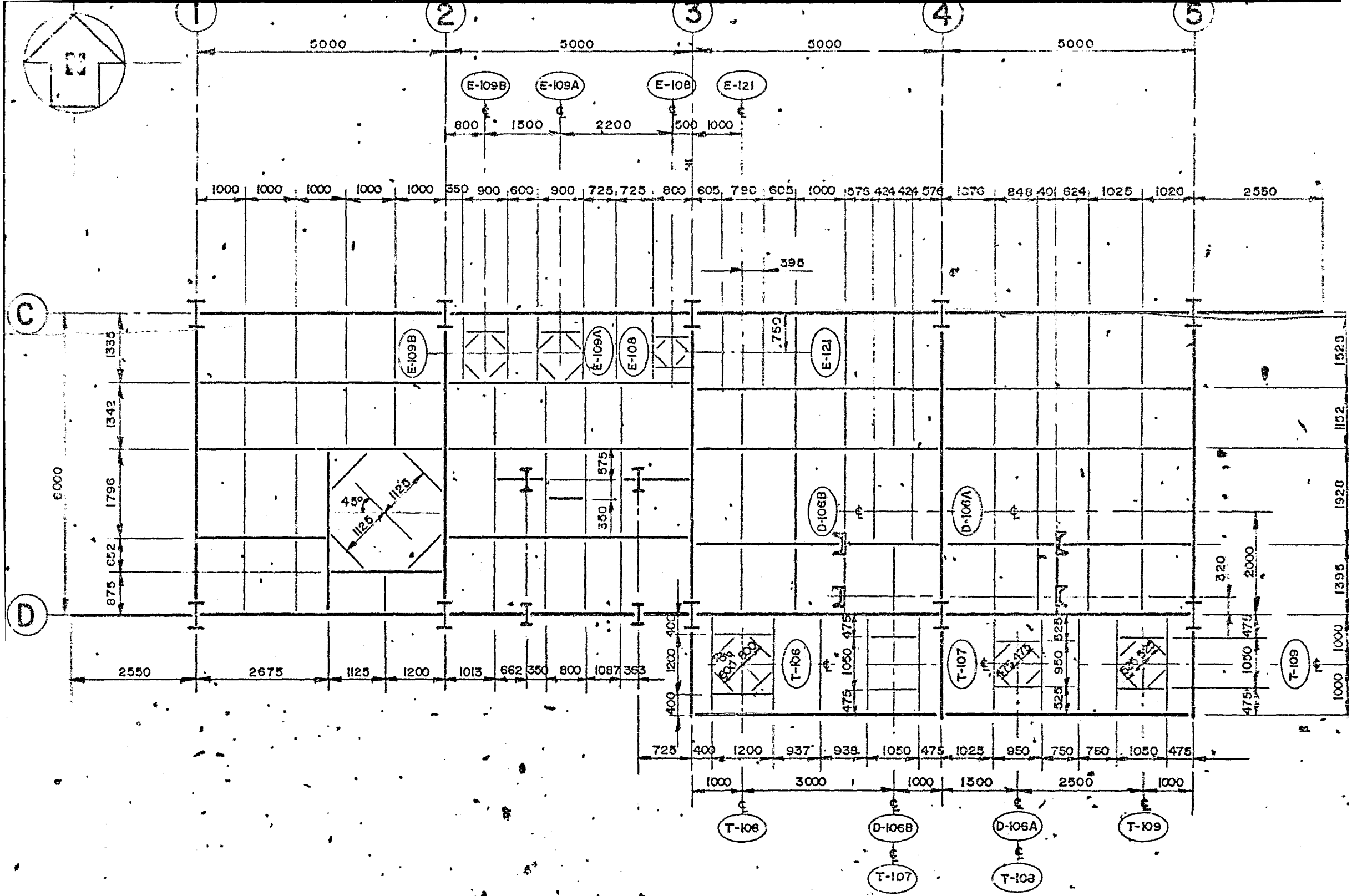
**PLANTA NIVEL 2 T/S.+9.694 Y T/S+9.194**

**F. S. T. R. U. C. T. U. R. A. C. I. O. N.**



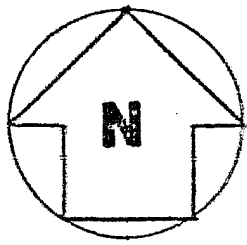
PLANTAS T/S + 11.864 Y T/S + 11.444  
ESTRUCTURACION





PLANTA NIVEL 3 T/S+13.649

ESTRUCTURACION



P-IIA/B

ε

3178

2425

P-III A/B

2000

865

868

868

1675

1250

750

750

655 (TIPO)

655 (TIPO)

45°(TIPO)

4

2000

2325

1075

320

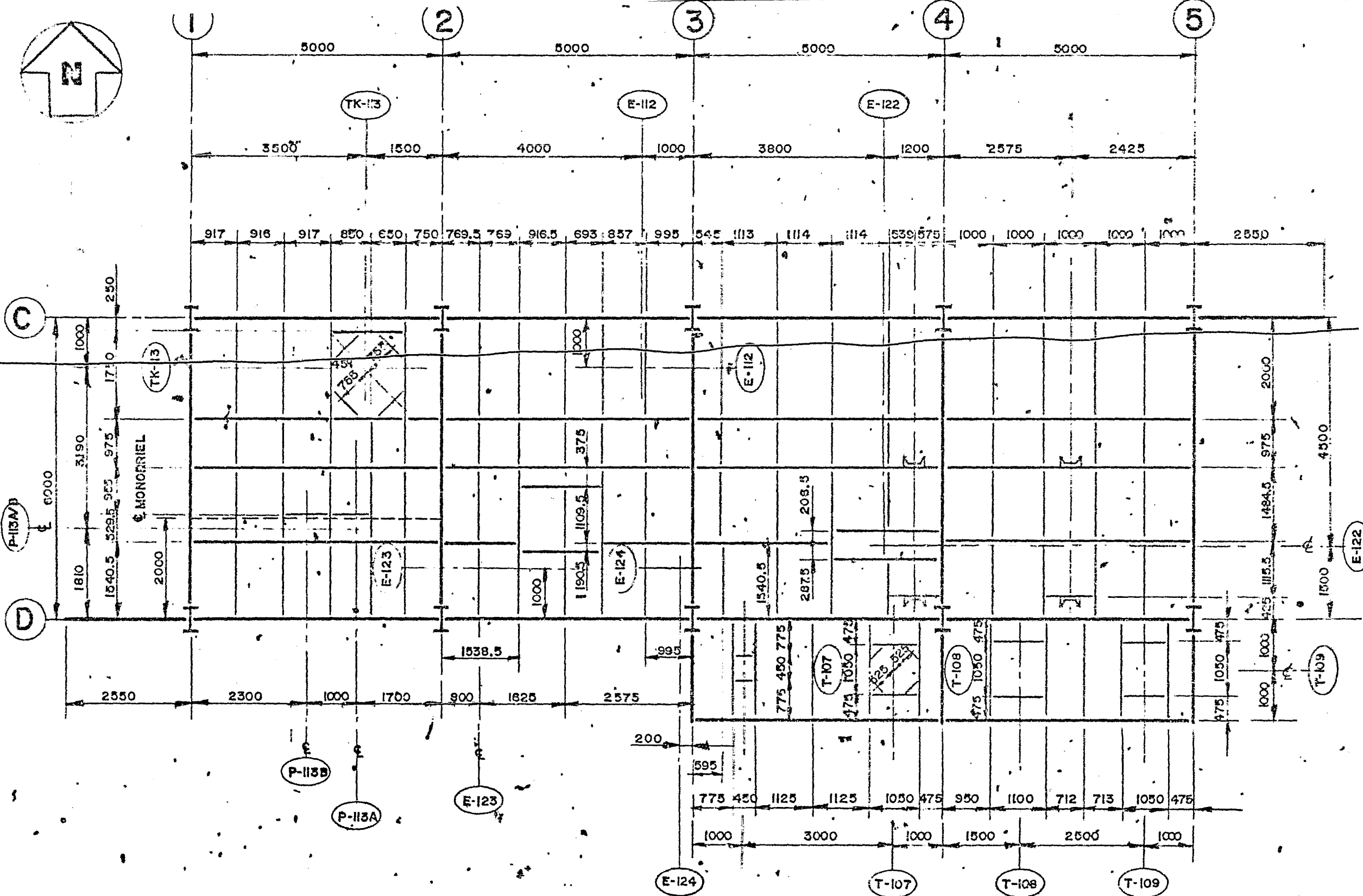
2162

2163

D

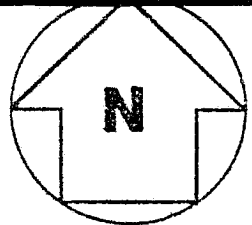
PLANTA T/S + 14.914  
ESTRUCTURACION

PLANTA T/S + 15.194  
ESTRUCTURACION

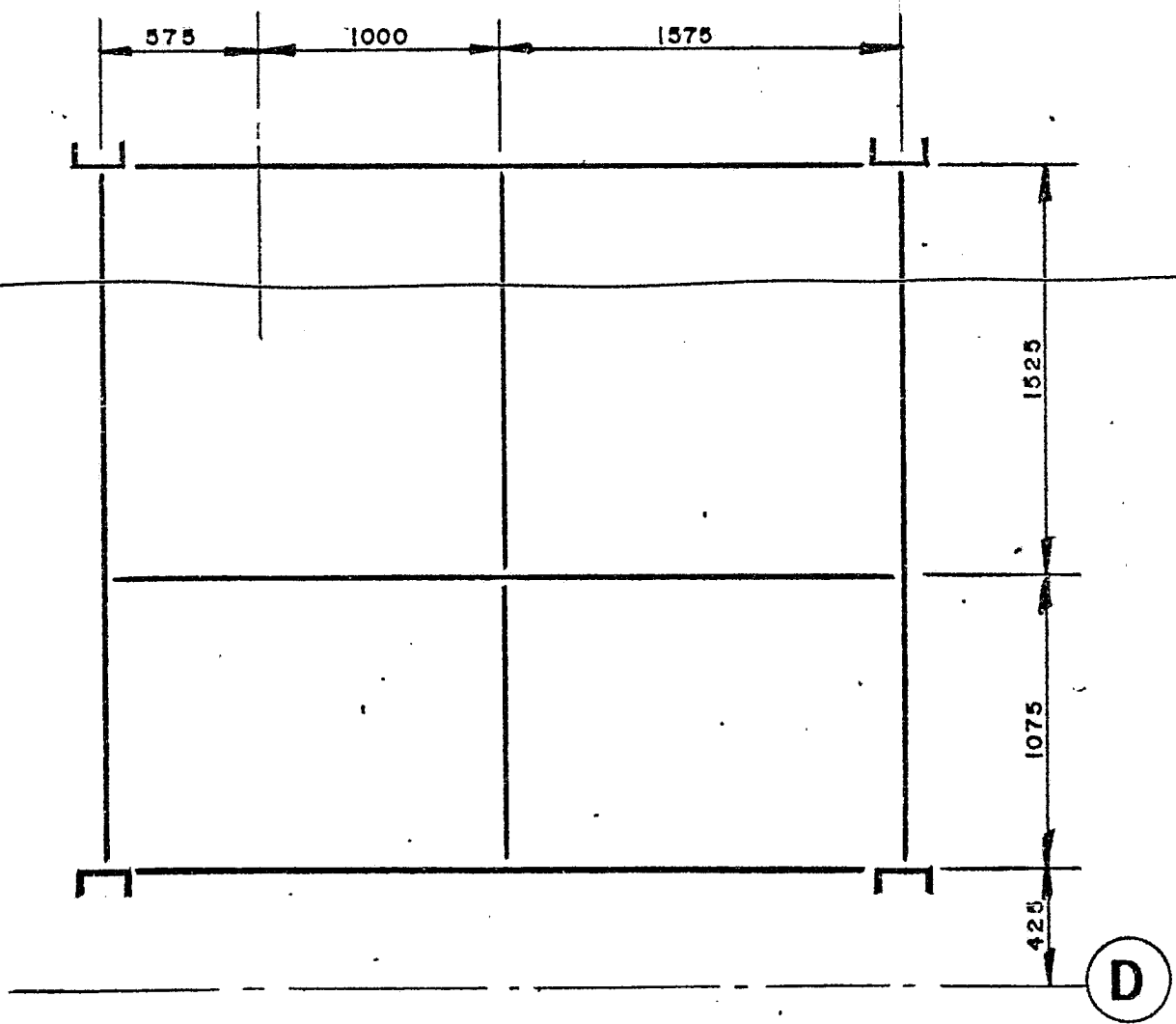


PLANTA NIVEL 4 T/S +17.694

ESTRUCTURACION



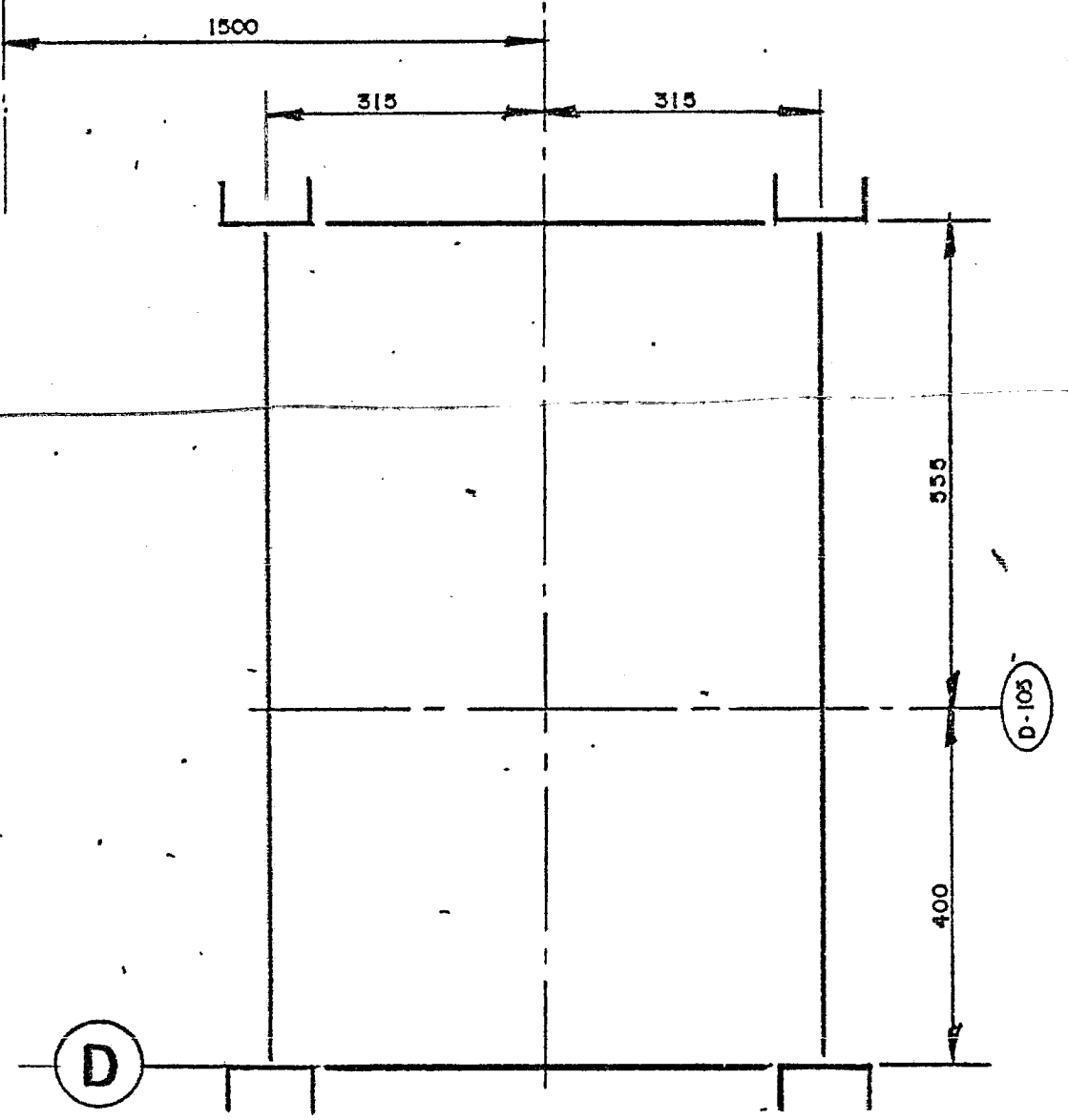
4



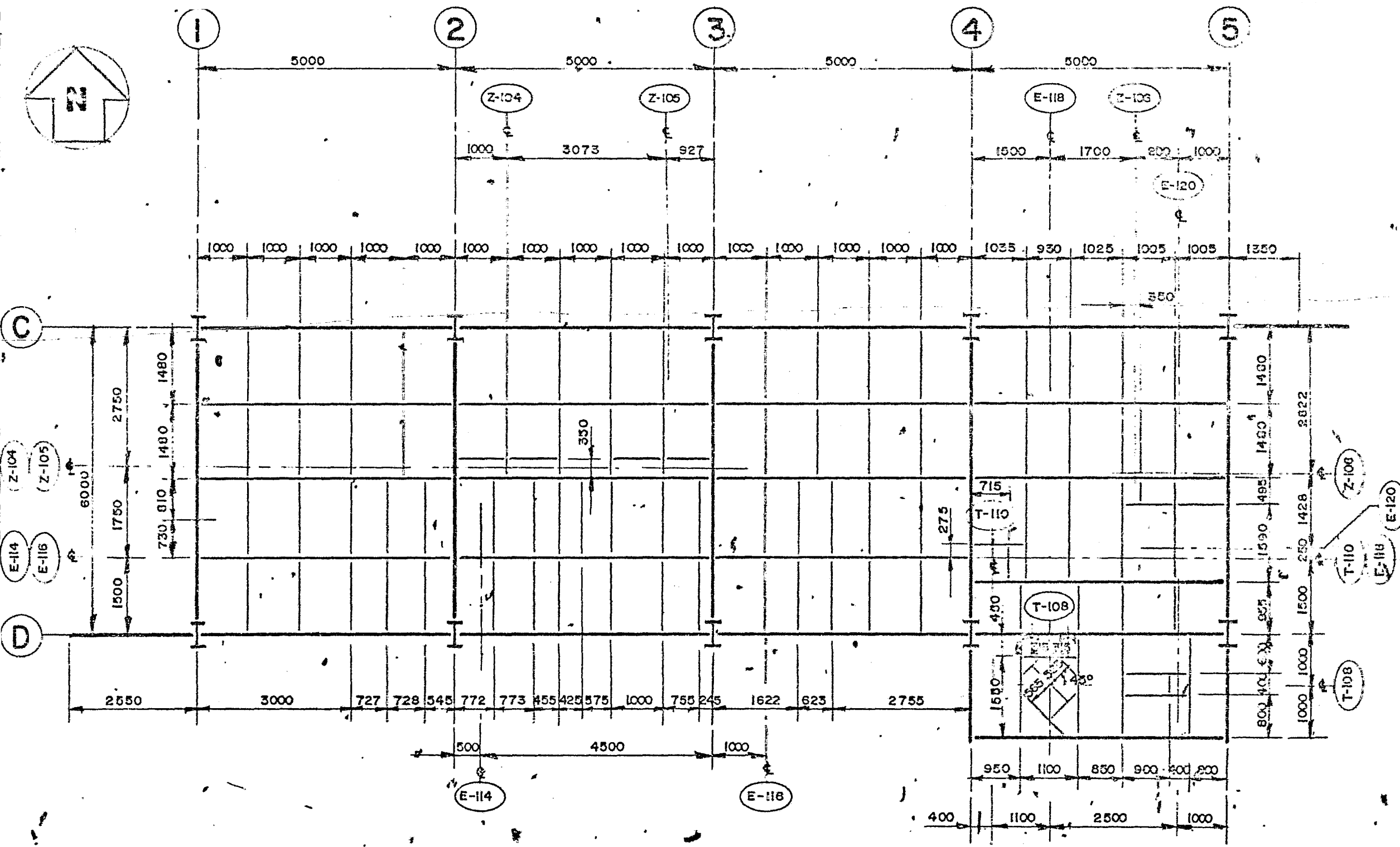
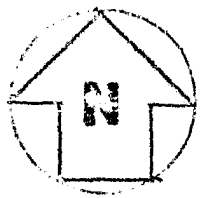
PLANTA T/S + 18.894  
ESTRUCTURACION

4

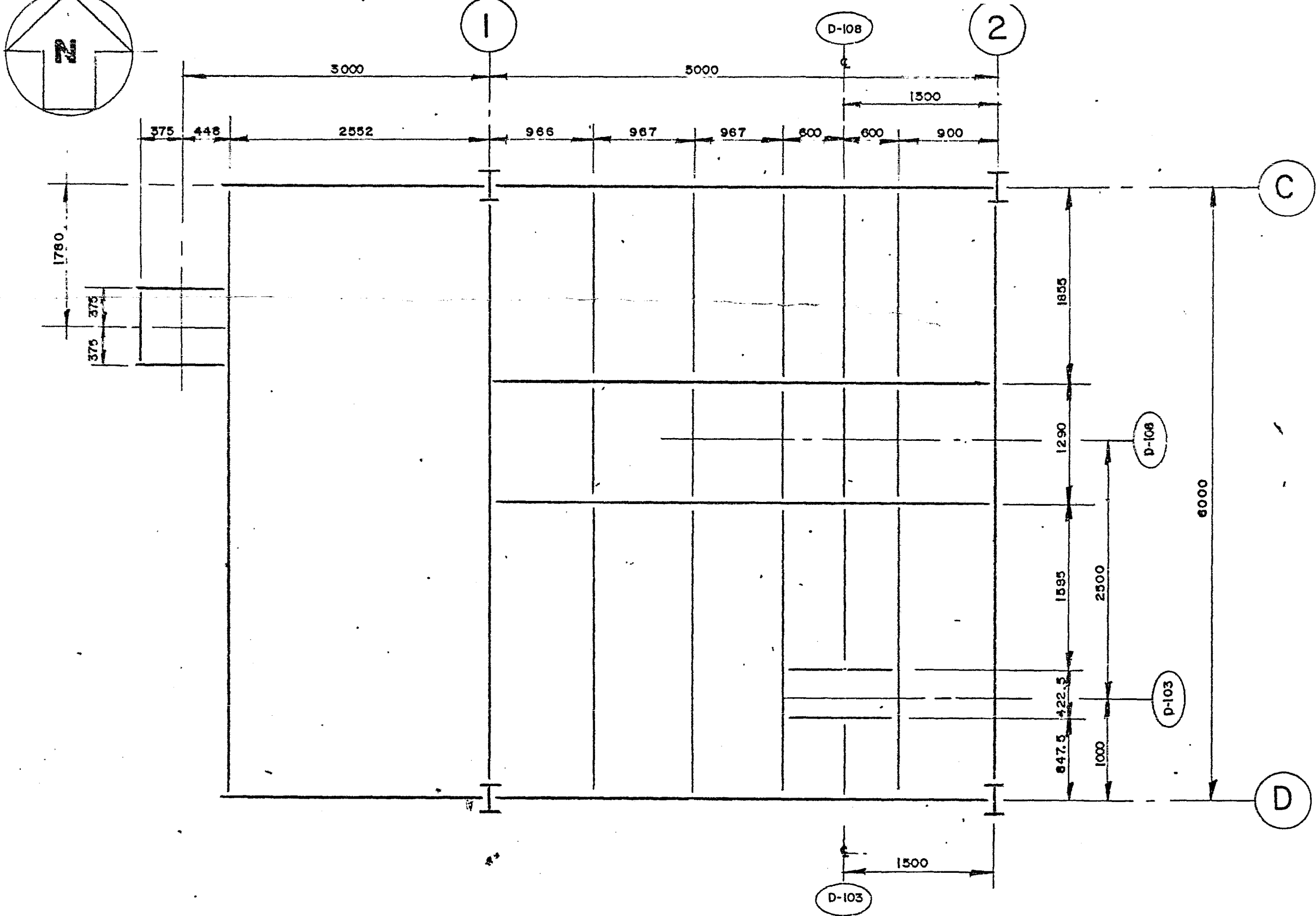
D-105



PLANTA T/S + 20.884  
ESTRUCTURACION



**PLANTA NIVEL + 5 T/S + 21.694**  
**ESTRUCTURACION**



PLANTA +6 - T/S 25.694

ESTRUCTURACION

## CAPITULO 2

### ANÁLISIS DE CARGAS

El sistema de piso será a base de placa con un espesor de 6 mm.

#### - Carga Muerta

$$W \text{ Placa espesor 6 mm. } \frac{8000 \text{ kg/m}^3 \times 0.006}{=} = 50 \text{ kgs/m}^2$$

$$W \text{ Estructura } \frac{\text{Para diseño de trabes pincipales}}{\text{considerar el p.p. de éstas}} \frac{50 \text{ kgs/m}^2}{100 \text{ kgs/m}^2}$$

Peso propio de equipos ver plano de ingeniería básica  
(se usan los pesos para diseño)

La W Estructura es sólo para trabes secundarias, para facilidad de la bajada de cargas.

#### - Carga Viva

$$W \text{ Pisos} = \frac{\text{(Para plantas industriales no esta especificada en los reglamentos mexicanos)}}{500 \text{ kgs/m}^2}$$

$$W \text{ Escaleras } \frac{500 \text{ kgs/m}^2}{\text{ó una carga concentrada movil de } \underline{\hspace{2cm}} \text{ 500 kgs}} \\ \text{la más desfavorable.}$$

- No se considera reducción por carga viva para combinación con sismo.

- No se analiza viento por considerar más critico sismo con cargas de diseño para equipos por lo que se esta despreciando viento en equipo vacio y por ser un edificio descubierto.

∴ Cargas de Diseño

Para Estructura

$$C.M = 100 \text{ kgs/m}^2$$

$$C.V = \frac{500}{\text{m}} \text{ kgs/m}^2$$

$$W = 600 \text{ kgs/m}^2 \text{ para traveses secundarias}$$

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Para Escaleras

$$C.M = 100 \text{ kgs/m}^2$$

$$C.V = \frac{500}{\text{m}} \text{ kgs/m}^2 \quad \text{ó}$$

$$600 \text{ kgs/m}^2$$

C.M = 100 kgs/m<sup>2</sup> y una carga viva concentrada - móvil de 500 kgs.

La condición más crítica para las escaleras.



## CARGAS ACCIDENTALES

### S I S M O:

Para el Cálculo de las fuerzas sísmicas se utilizó el Manual de Obras Civiles de la Comisión Federal de Electricidad, última revisión.

La estructura está localizada en:  
Cosoleacaque, Veracruz.

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Clasificación de la estructura:  
Grupo B

Estructuración:  
Tipo 1

Zona sísmica: B

Tipo de Suelo: III

Coefficiente sísmico básico:  
 $C_B = 0.24$

Factor de ductilidad:  
 $Q = 2$

Método de análisis:  
Estático

Coefficiente sísmico reducido:

$$\frac{C_B}{Q} = \frac{0.24}{2} = 0.12 > a_0 = 0.06$$

Se hizo un análisis y diseño preliminar con el objeto de determinar es-cuadrías aproximadas para los elementos de la estructura y poder reali-zar un análisis más exacto haciendo uso de dichas escuadrías. El aná--lisis por carga vertical se elaboró con el método tradicional de distri-bución de momentos de H. Cross.

El análisis por cargas accidentales (sismo) se realizó con el método --estático, que supone que la estructura forma un sistema discreto de ma-sas concentradas en cada nivel, que la aceleración que sufre cada masa, producida por una excitación sísmica, varía linealmente con la altura -de dicha masa sobre la base del edificio, siendo nula en la base y máxima en la parte más alta de la estructura y, por último, que el cortante en la base es igual al producto del peso total del edificio por el coefi--ciente sísmico especificado.

Una vez conocidos los cortantes sísmicos preliminares en cada nivel de cada marco se procedió a determinar los elementos mecánicos en las dis-tintas secciones, aplicando el método aproximado de Bowman, que consis-te en suponer conocidos los puntos de inflexión en las columnas. En el sentido longitudinal se idealizaron los marcos como una armadura y se -consideró que el contraventeo tomaba toda la fuerza sísmica.

Conocidos los elementos mecánicos tanto por cargas gravitacionales como por cargas accidentales, se procedió al diseño de los diferen-tes miembros de la estructura. Siguiendo los criterios de diseño mencionados -con anterioridad, vale la pena aclarar que cada sección se diseñó para las condiciones más desfavorables, bien fuera por cargas gravitaciona--les o por la combinación de éstas con las accidentales.

# BAJADAS DE CARGAS Y ANALISIS DE TRABES SECUNDARIAS POR NIVEL

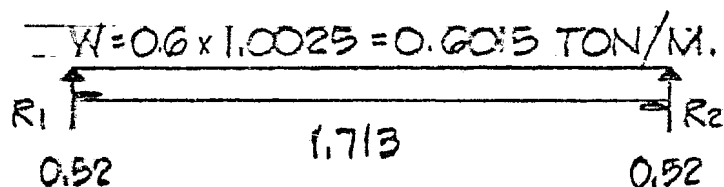
NIVEL T/S + 11.444

CARGAS DE DISEÑO

$$W = 600 \text{ KGS./m}^2 = 0.6 \text{ TON/M}^2$$

PESO PROPIO DE EQUIPOS

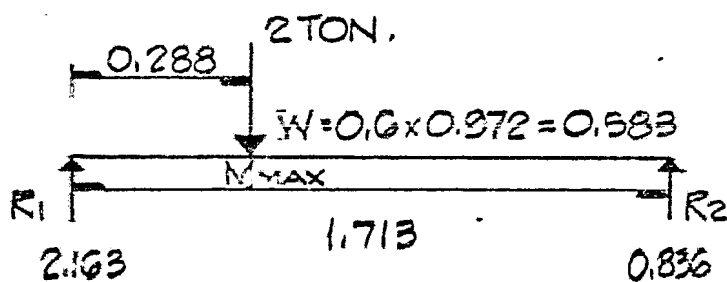
T-1



$$M = \frac{Wl^2}{8} = 0.2206 \text{ TON-M.}$$

$$R_1 = R_2 = \frac{Wl}{2} = 0.5152 = 0.52 \text{ TON.}$$

T-1A

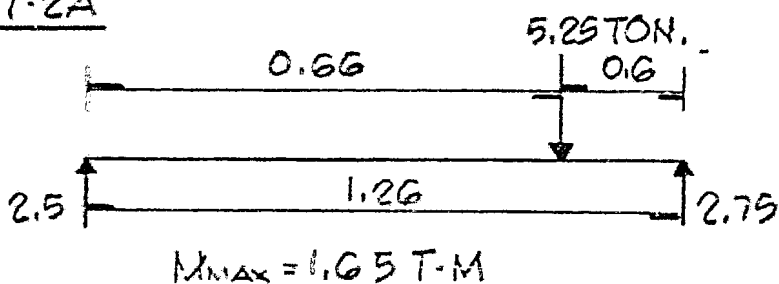


$$R_1 = \frac{0.583 \times 1.713}{2} + \frac{2(1.713 - 0.288)}{1.713} = 2.16 \text{ TON}$$

$$R_2 = 2 + 0.583 \times 1.713 - 2.16 = 0.84 \text{ TON.}$$

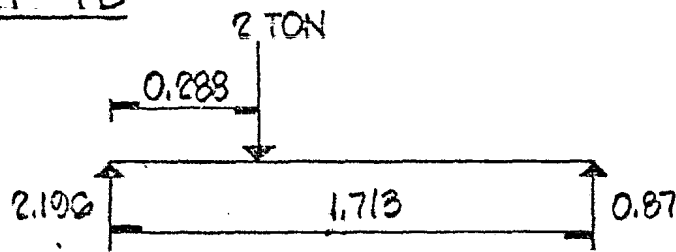
$$M_{\text{MAX}} = 2.16 \times 0.288 - 0.583 \times \frac{0.288^2}{2} = 0.6 \text{ TON-M.}$$

T-2A



A PARTIR DE LA T-13 LOS ELEMENTOS MECANICOS SE CALCULAN CON EL MODULO DE ANALISIS ESTRUCTURAL DE LA CALCULADORA HP-41CV.

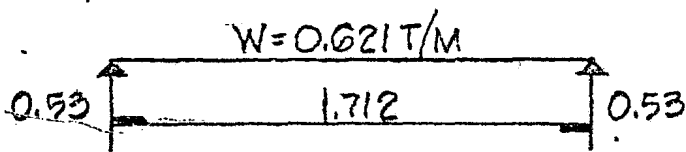
T-1B



$$W = 0.6 \times 1.035 = 0.621 \text{ T/M}$$

$$M_{\text{MAX}} = 0.61 \text{ T-M}$$

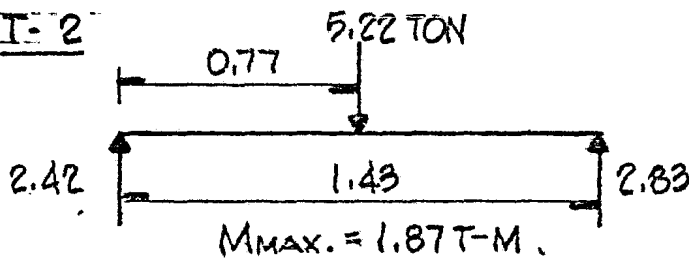
T-1C



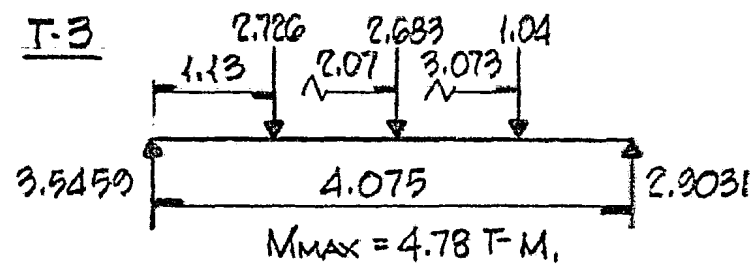
$$W = 0.6 \times 1.035 = 0.621 \text{ T/M}$$

$$M_{\text{MAX}} = 0.23 \text{ T-M}$$

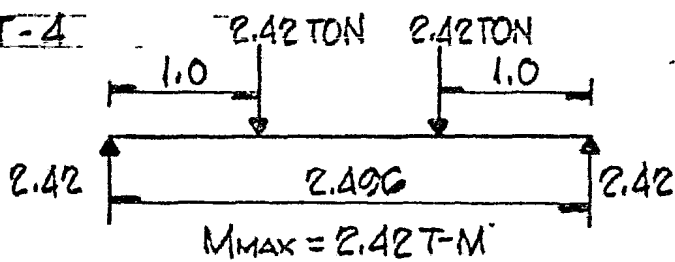
T-2



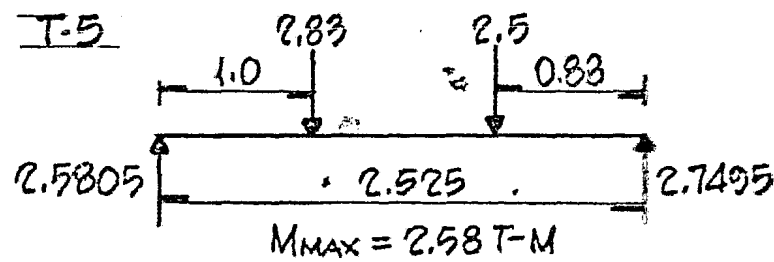
T-3



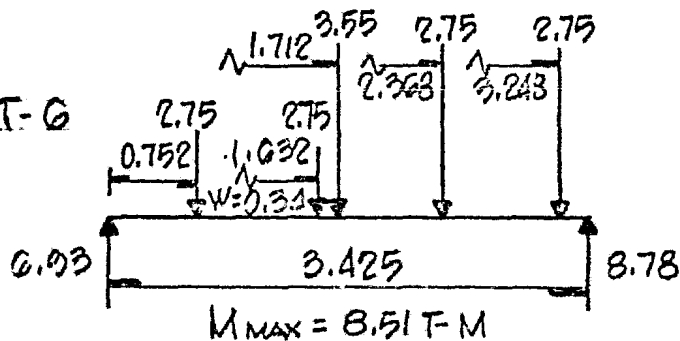
T-4



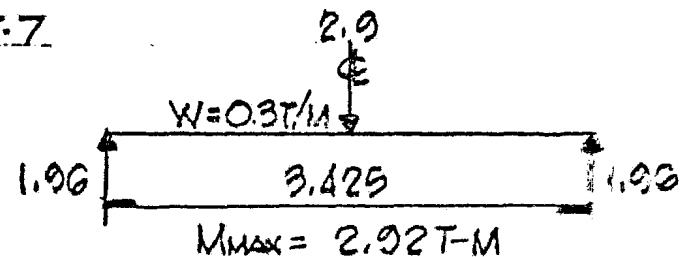
T-5



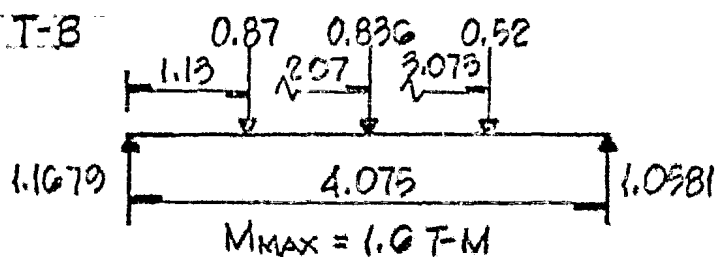
T-6



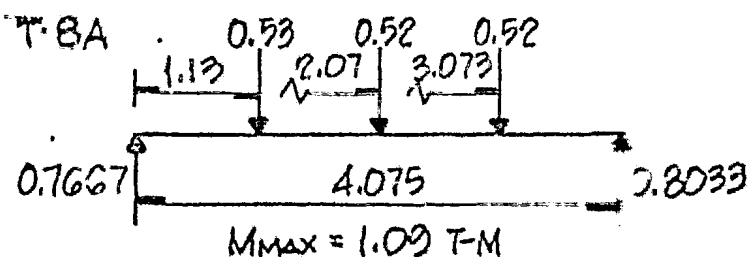
T-7



T-8



T-8A

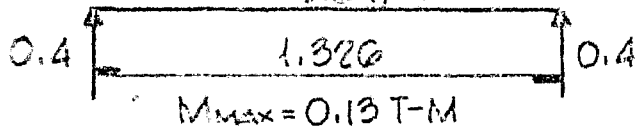


NIVEL T/3 + 9.694 Y T/3 + 9.194

T-9

$$W = 0.6 \times 1.0 = 0.6 T/M$$

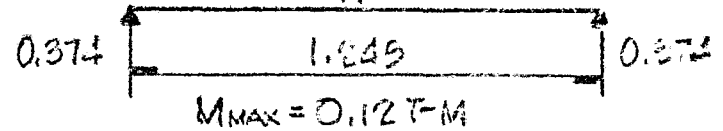
$$W = 0.6 T/M$$



T-10

$$W = 0.6 \times 1.0 = 0.6 T/M$$

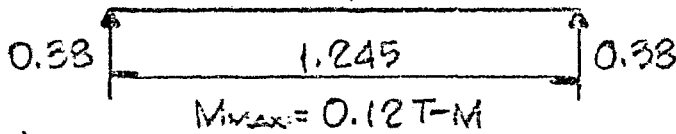
$$W$$



T-11

$$W = 0.6 \times 1.019 = 0.61 T/M$$

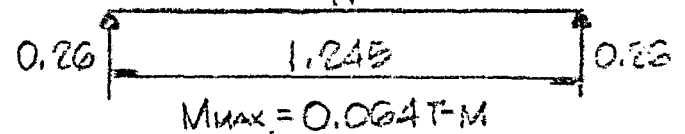
$$W$$



T-12

$$W = 0.6 \times 0.675 = 0.41 T/M$$

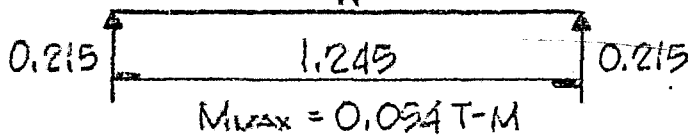
$$W$$



T-13

$$W = 0.6 \times 0.575 = 0.345 T/M$$

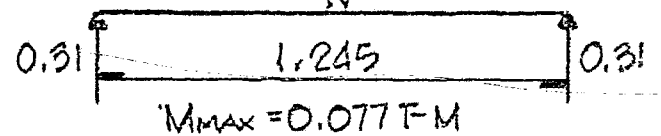
$$W$$



T-14

$$W = 0.825 \times 0.6 = 0.495$$

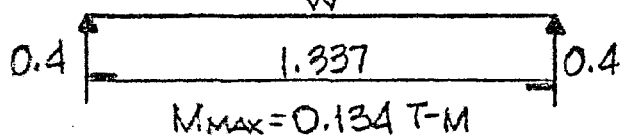
$$W$$



T-15

$$W = 0.6 \times 1.0 = 0.6 T/M$$

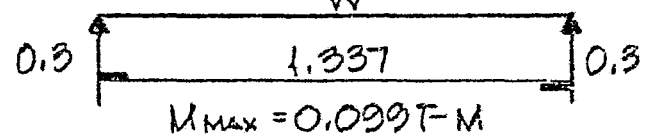
$$W$$



T-16

$$W = 0.6 \times 0.7375 = 0.443 T/M$$

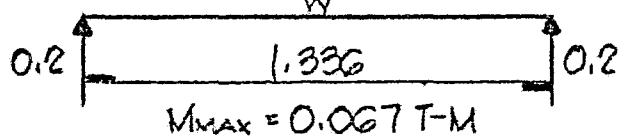
$$W$$



T-16A

$$W = 0.6 \times 0.5 = 0.3 T/M$$

$$W$$

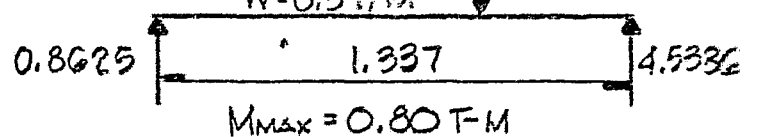


T-17

$$W = 0.6 \times 0.5 = 0.3$$

$$1.16 = 5.0 \times 0.177$$

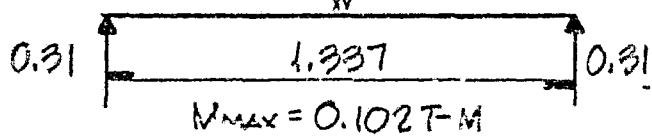
$$W = 0.3 T/M$$



T-18

$$W = 0.6 \times 0.7625 = 0.4575$$

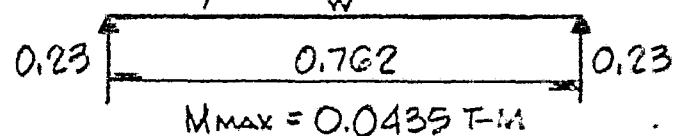
$$W$$



T-19

$$W = 0.6 \times 1.0 = 0.6 T/M$$

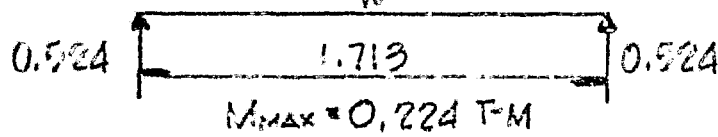
$$W$$



T-20

$$W = 0.6 \times 1.019 = 0.61 T/M$$

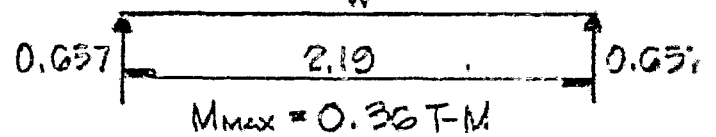
$$W$$



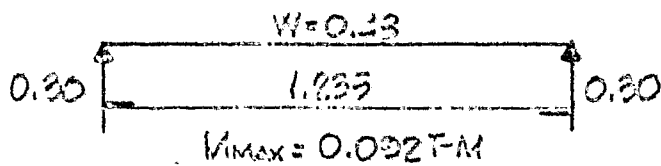
T-21

$$W = 0.6 \times 1.0 = 0.6 T/M$$

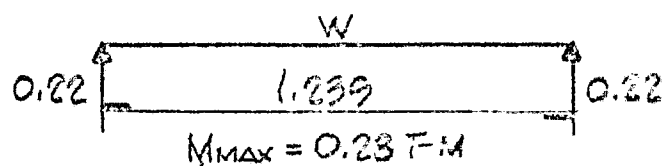
$$W$$



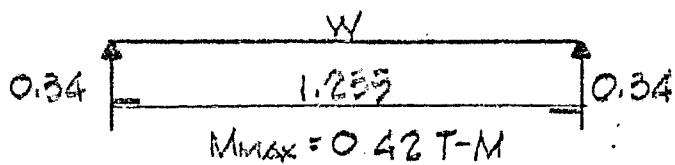
T-22 Y T-23  $W = 0.6 \times 0.8025 = 0.48 T/M$



T-24  $W = 0.6 \times 0.603 = 0.36 T/M$



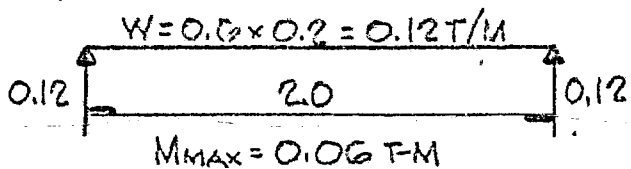
T-25  $W = 0.6 \times 0.92 = 0.552$



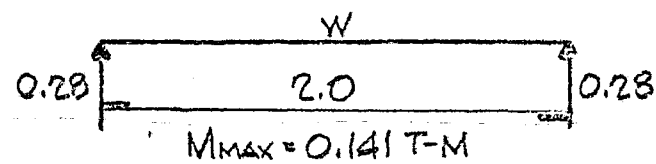
T-26, T-27 Y T-28

UNICAMENTE PESO PROPIO

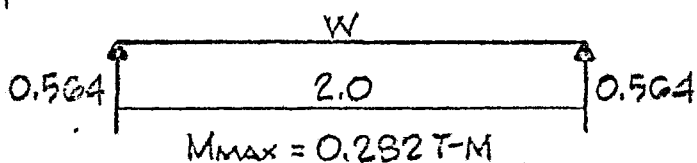
T-29



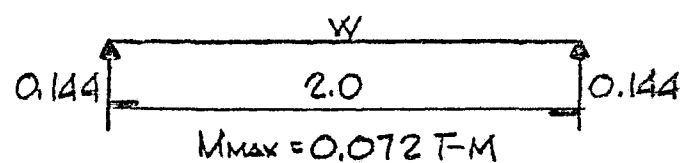
T-30  $W = 0.6 \times 0.469 = 0.28 T/M$



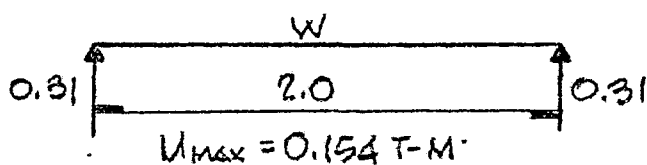
T-31  $W = 0.6 \times 0.94 = 0.564 T/M$



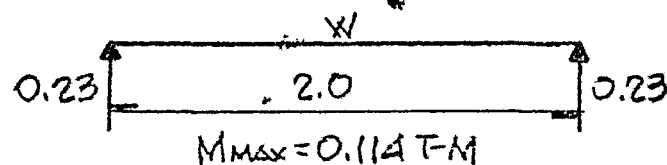
T-32  $W = 0.6 \times 0.24 = 0.144$



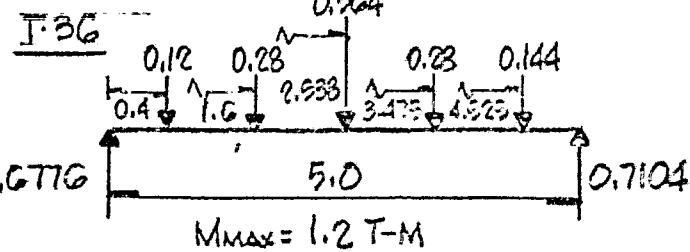
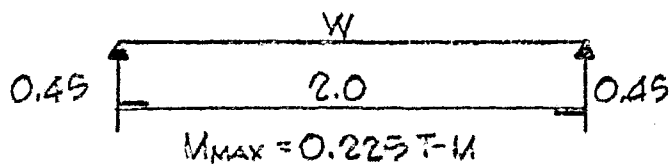
T-33  $W = 0.6 \times 0.153 = 0.31 T/M$



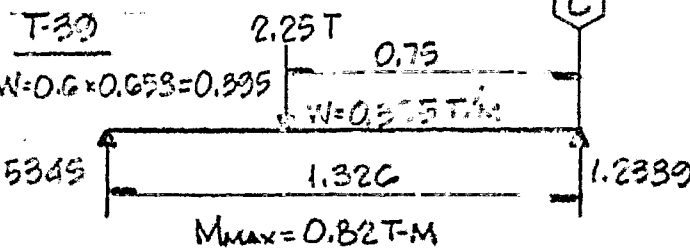
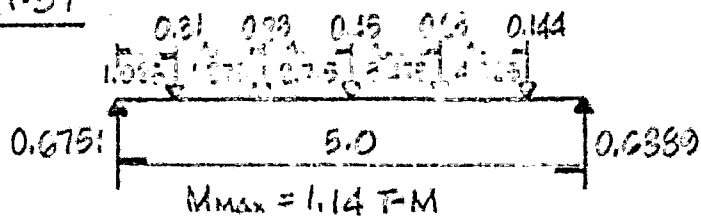
T-34  $W = 0.6 \times 0.38 = 0.23 T/M$

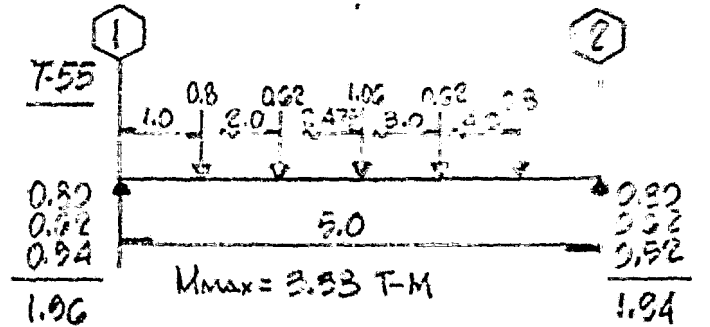
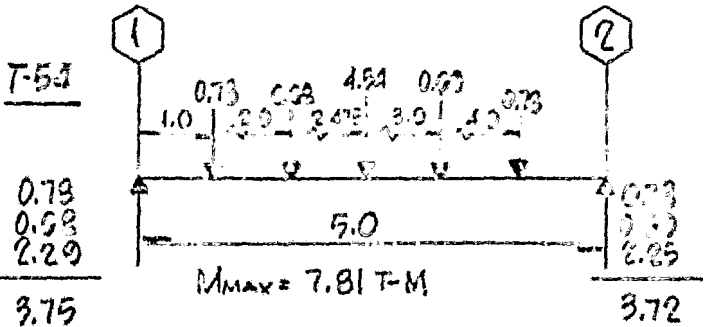
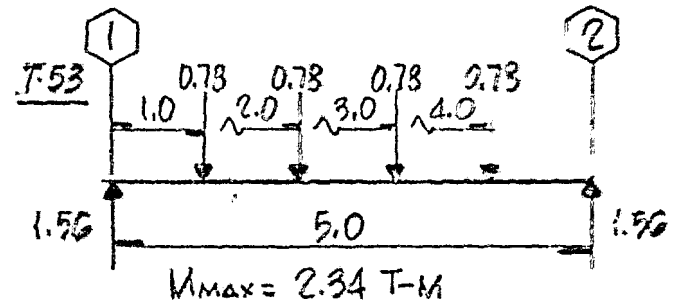
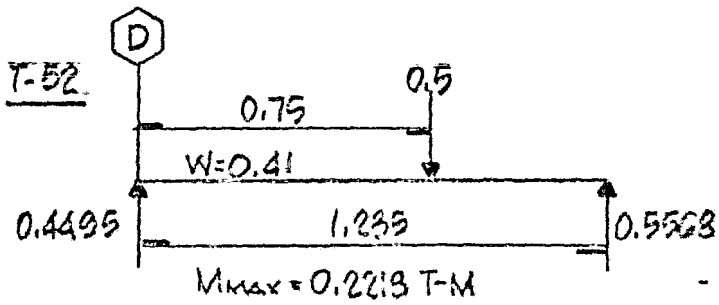
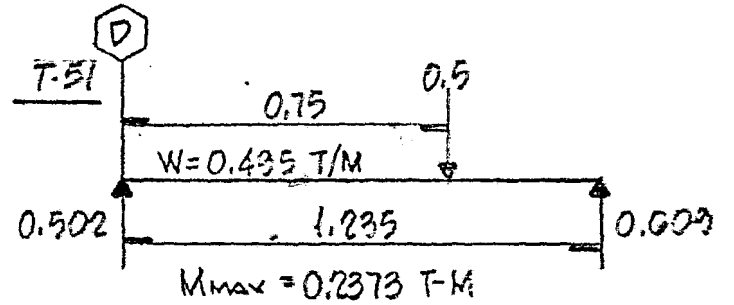
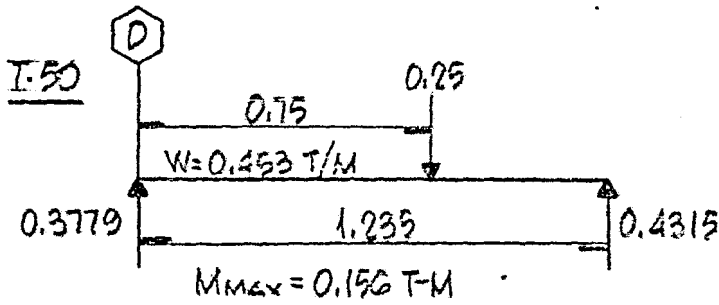
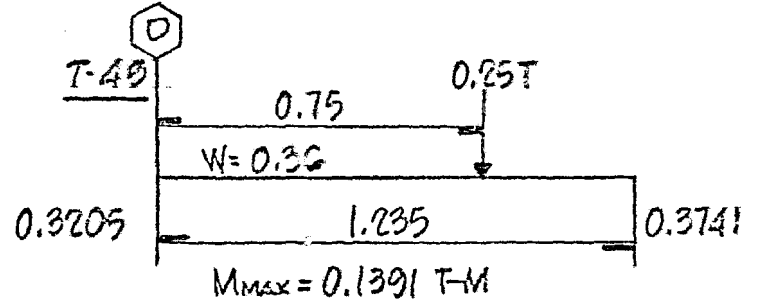
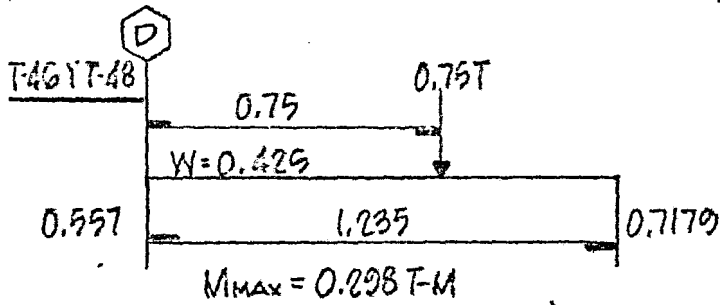
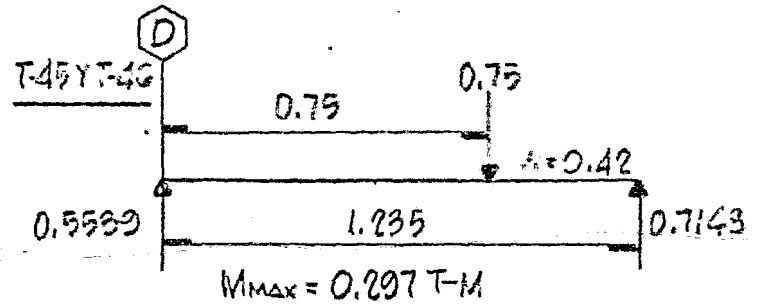
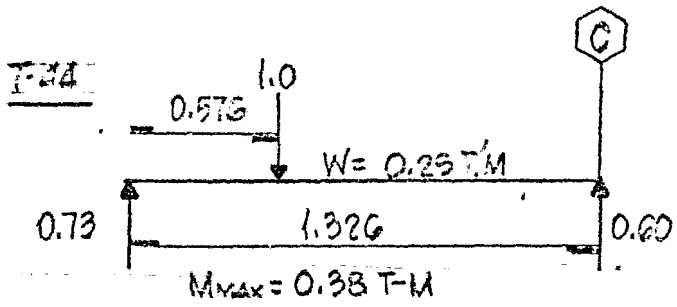
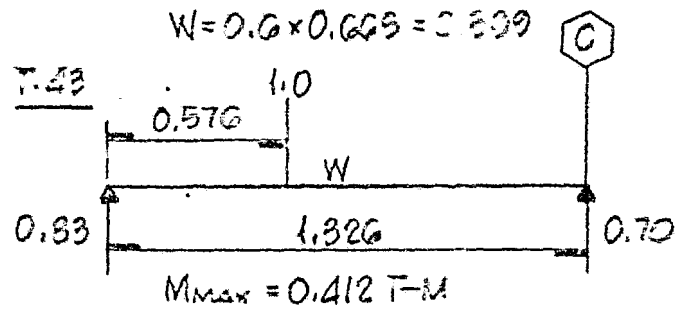
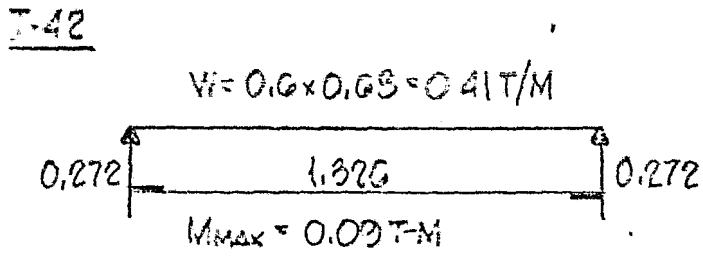
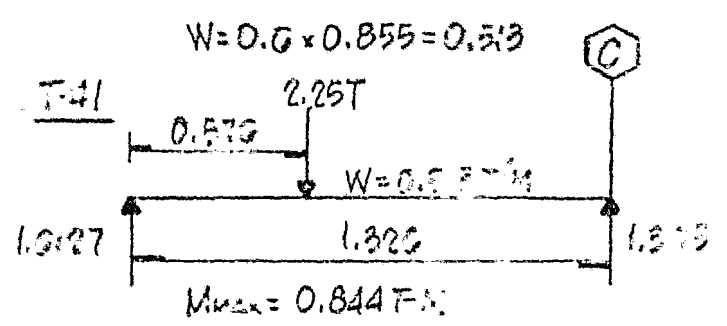
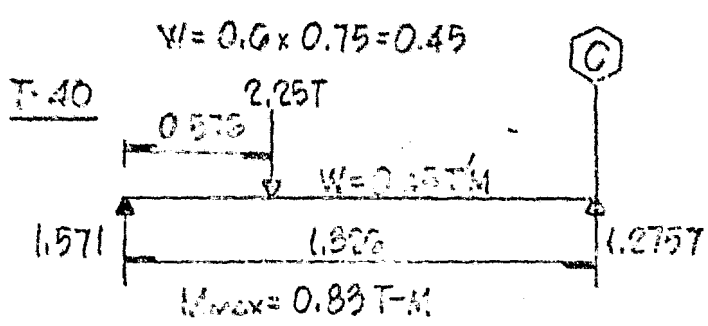


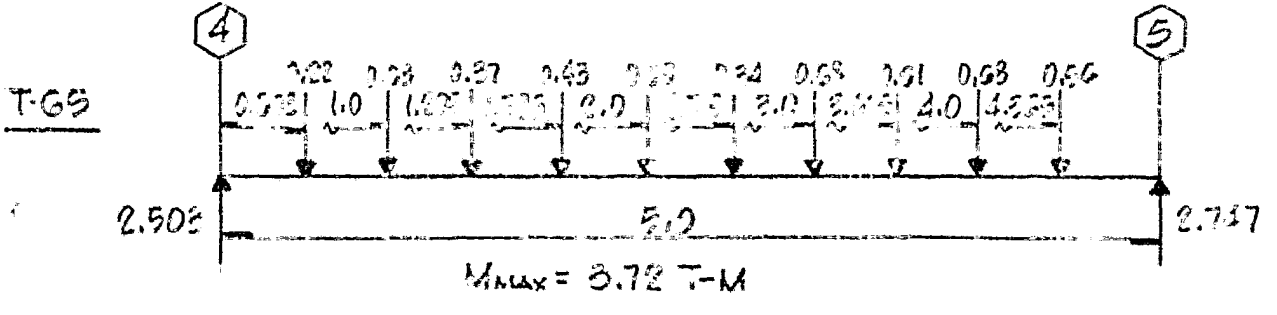
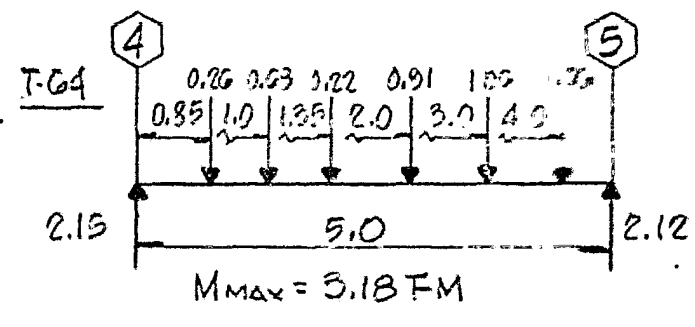
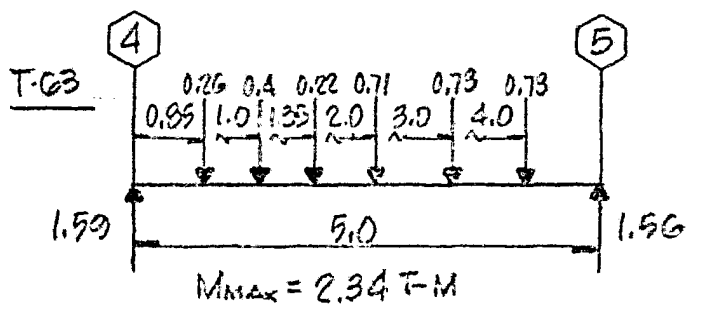
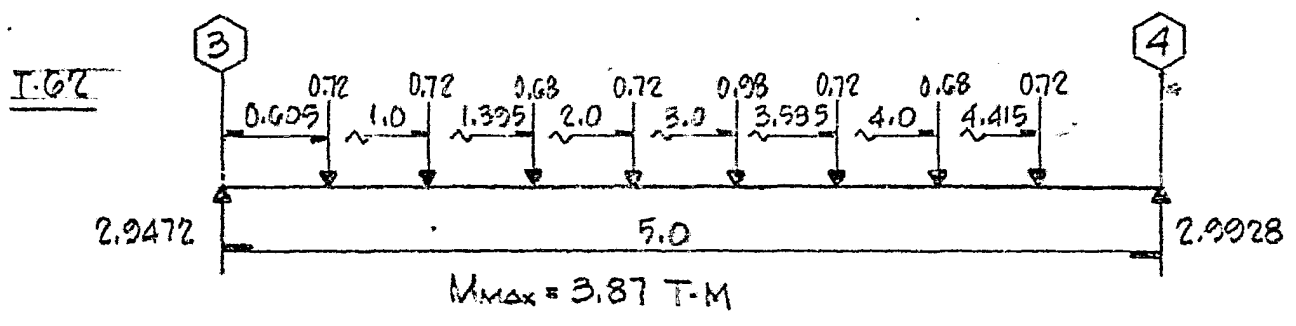
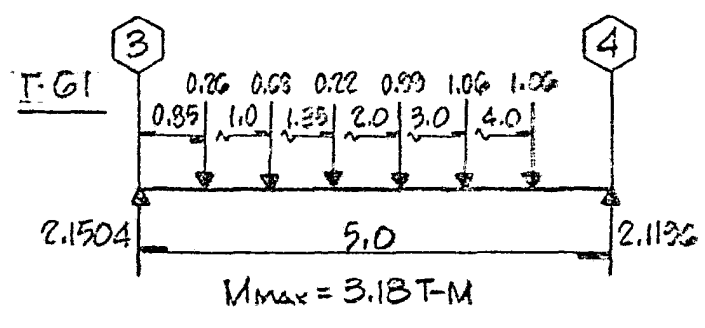
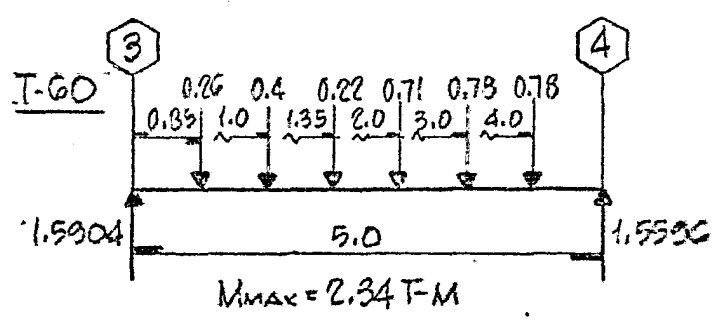
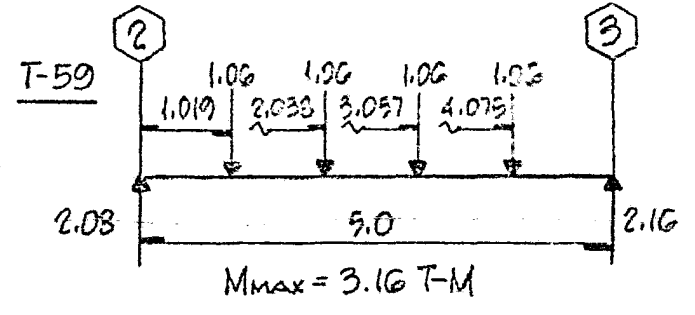
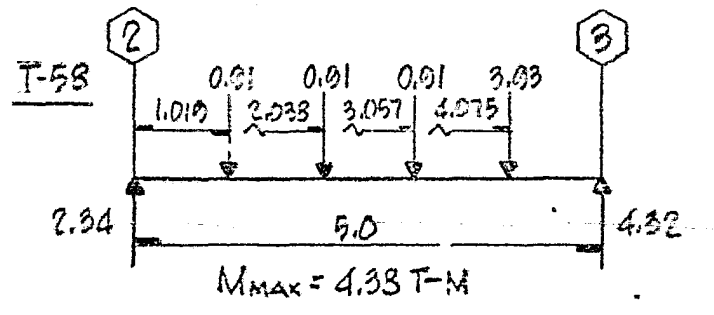
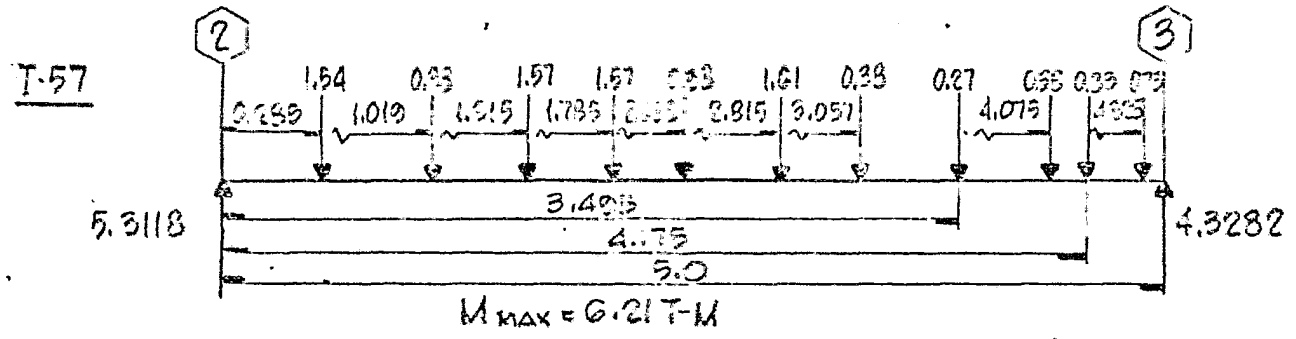
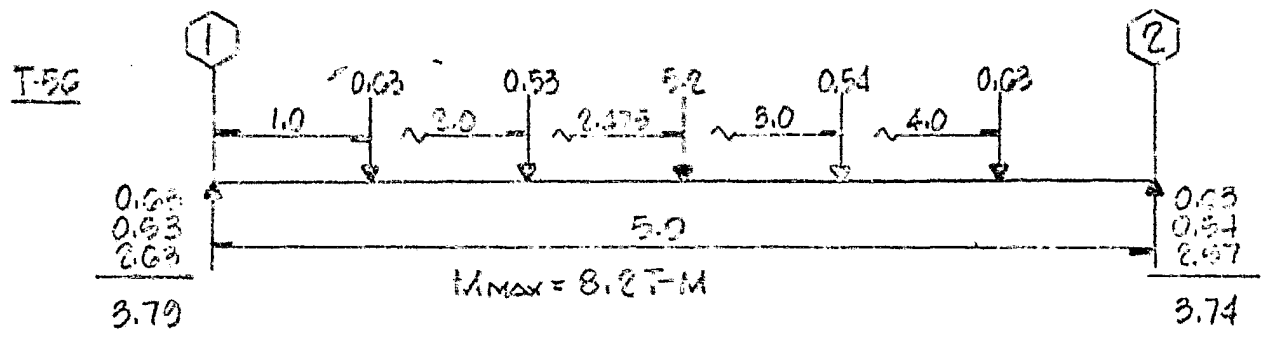
T-35  $W = 0.6 \times 0.75 = 0.45 T/M$



T-37





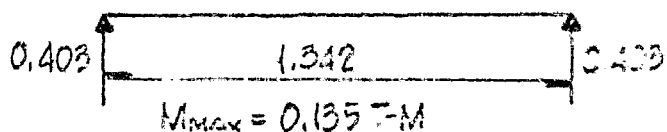




# NIVEL T/S + 13.624

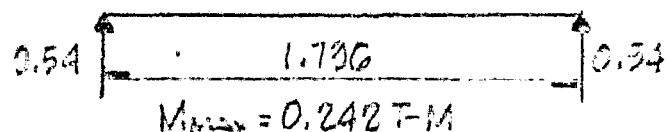
T-66

$W = 0.6 \times 1.0 = 0.6 \text{ T/M}$



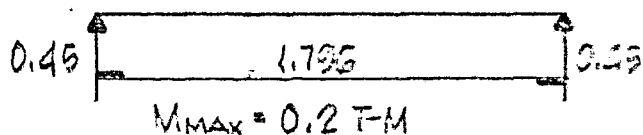
T-67

$W = 0.6 \times 1.0 = 0.6 \text{ T/M}$



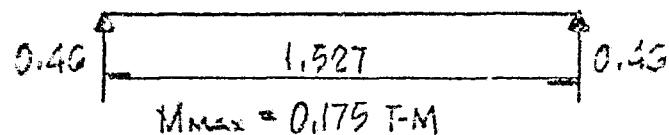
T-68

$W = 0.6 \times 0.34 = 0.2 \text{ T/M}$



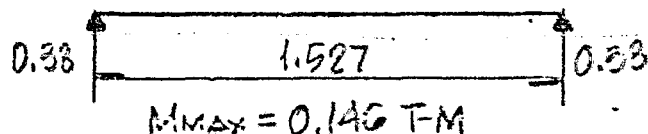
T-69

$W = 0.6 \times 1.0 = 0.6 \text{ T/M}$



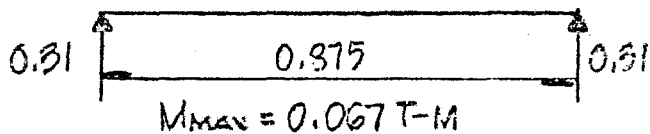
T-70

$W = 0.6 \times 0.34 = 0.2 \text{ T/M}$

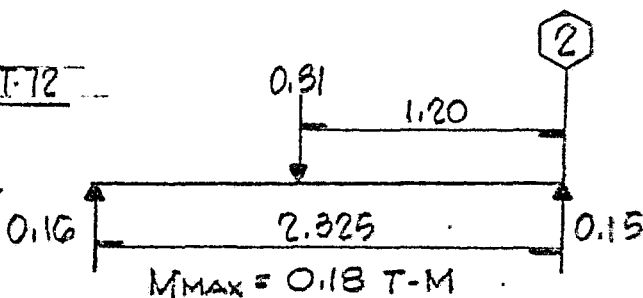


T-71

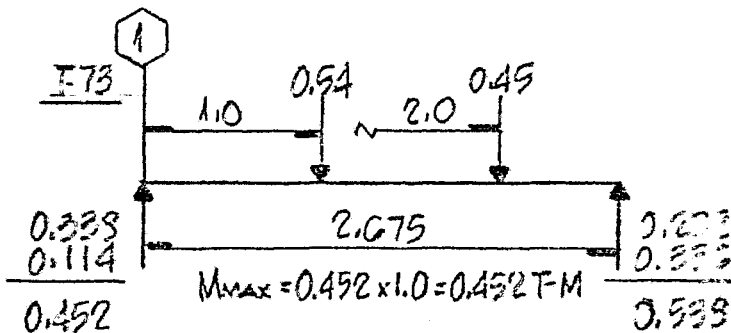
$W = 0.6 \times 1.163 = 0.7 \text{ T/M}$



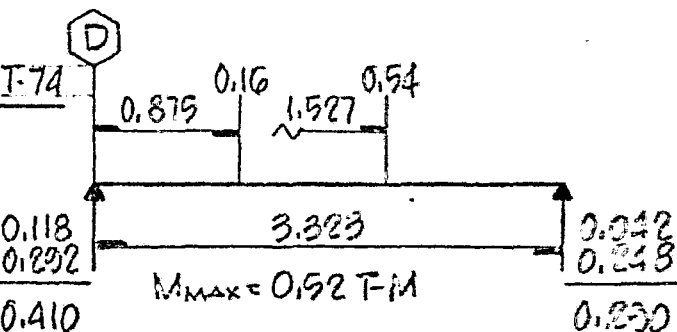
T-72



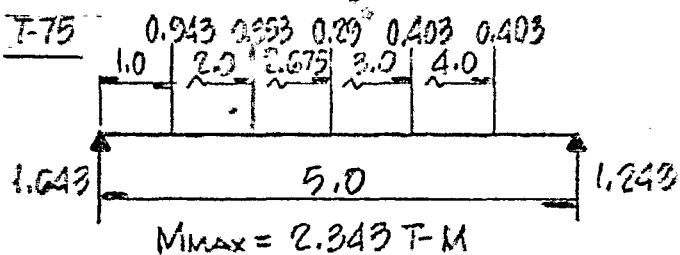
T-73



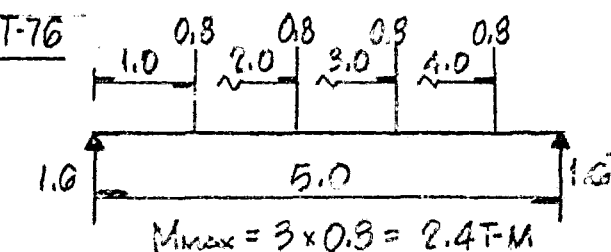
T-74



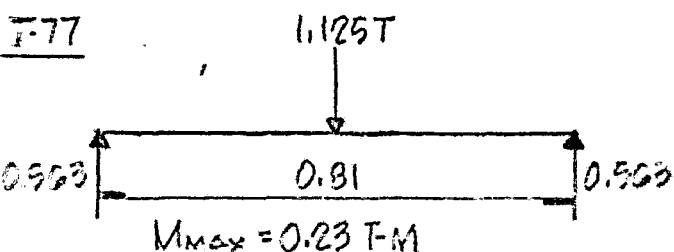
T-75



T-76

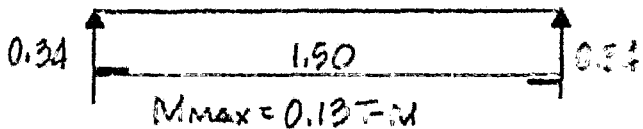


T-77

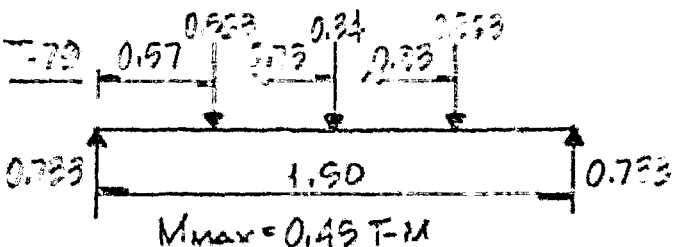


T-78

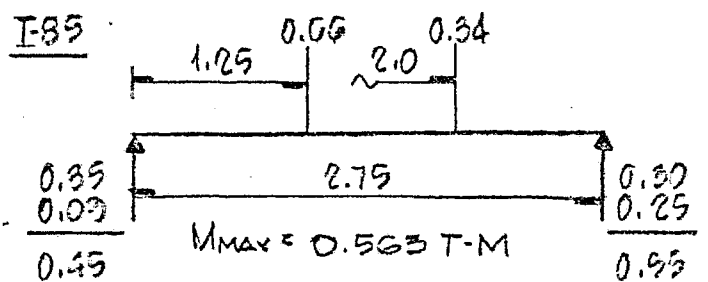
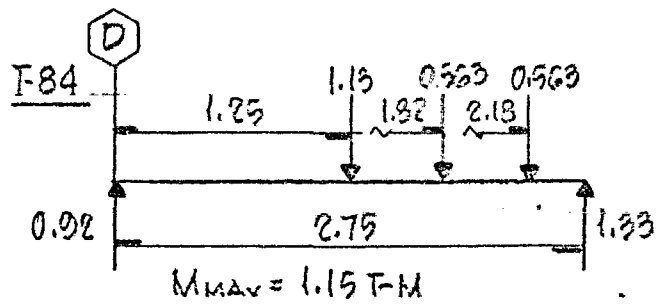
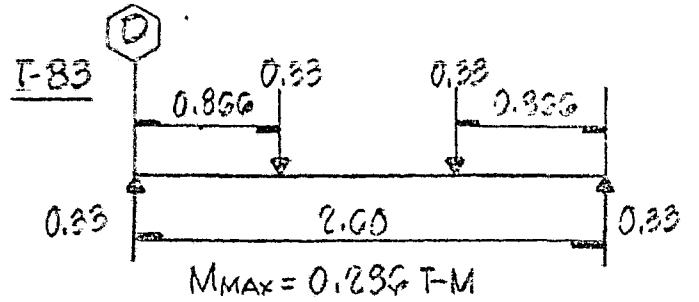
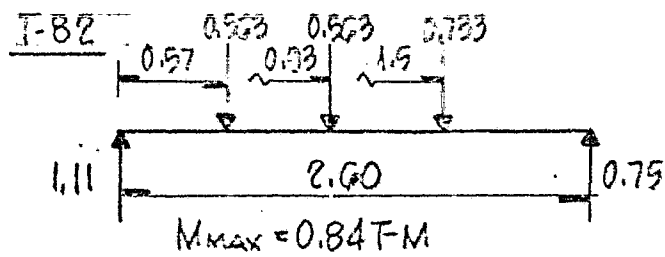
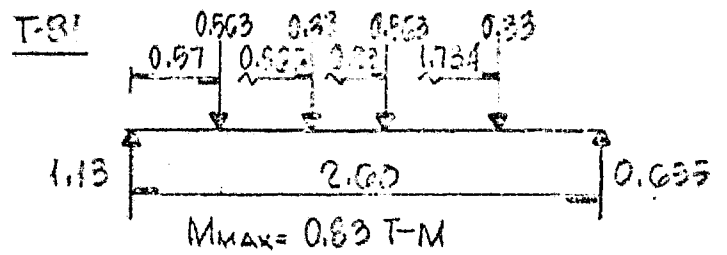
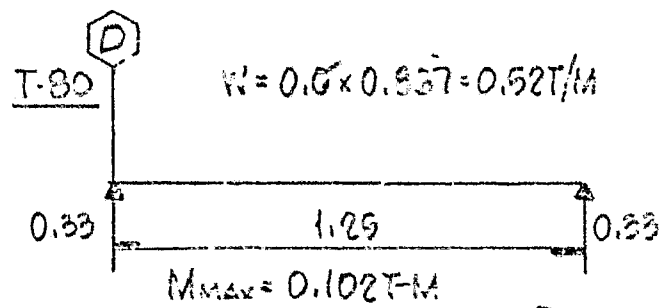
$W = 0.6 \times 0.75 = 0.45 \text{ T/M}$



T-79

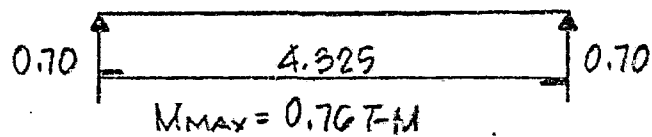


# NIVEL T/S + 14.915



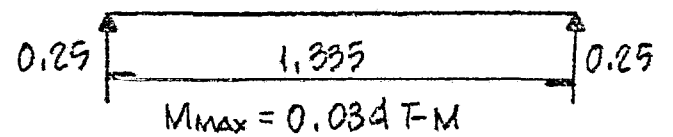
NIVEL T/S + 15.194

T-86  $W = 0.6 \times 0.5875 = 0.3525 T/M$

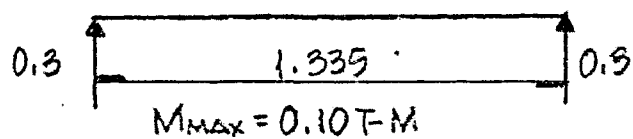


NIVEL T/S + 13.694

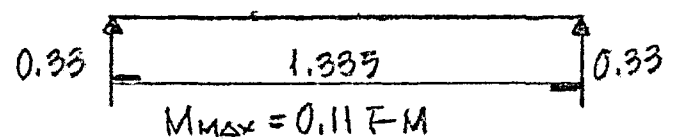
T-87  $W = 0.6 \times 0.625 = 0.375 T/M$



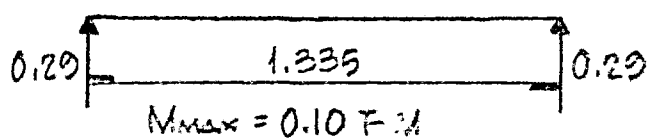
T-88  $W = 0.6 \times 0.75 = 0.45 T/M$



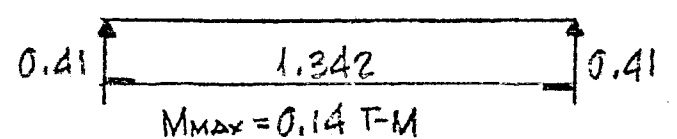
T-89  $W = 0.6 \times 0.813 = 0.4878 T/M$



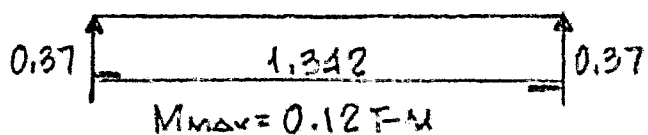
T-90  $W = 0.6 \times 0.725 = 0.435 T/M$



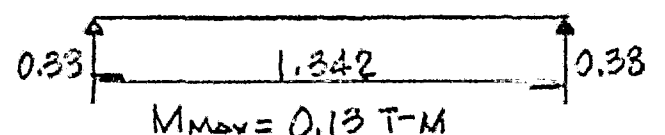
T-91  $W = 0.6 \times 1.013 = 0.6078 T/M$

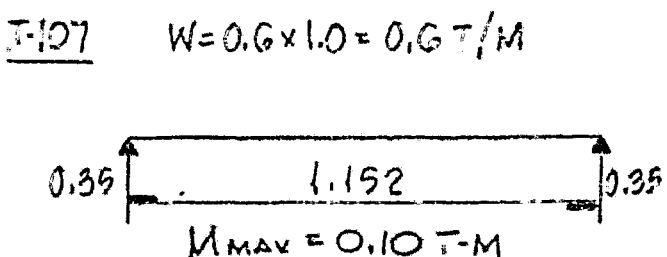
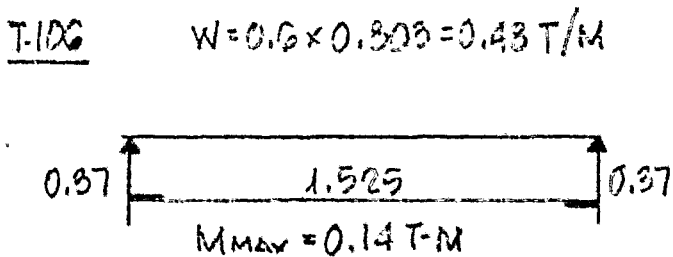
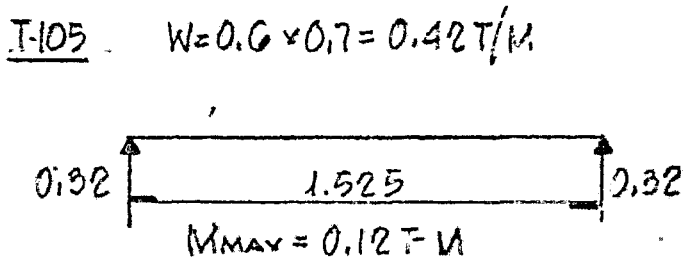
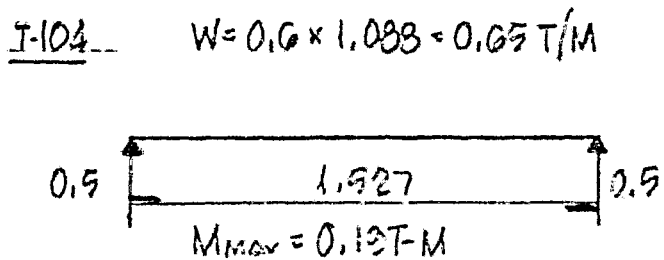
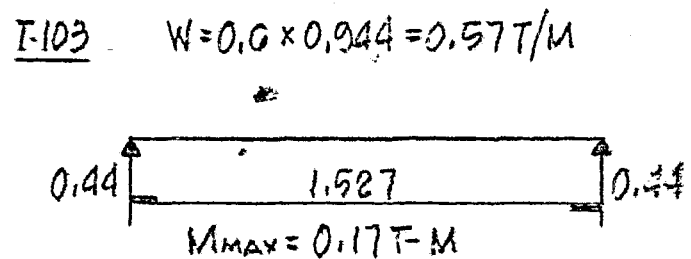
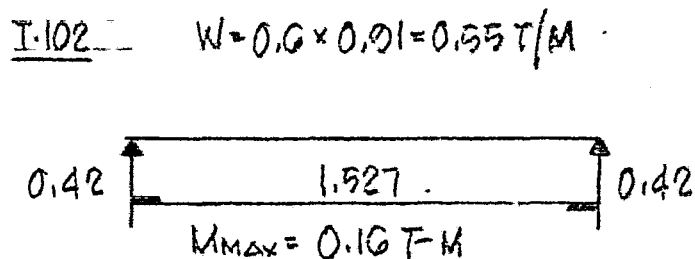
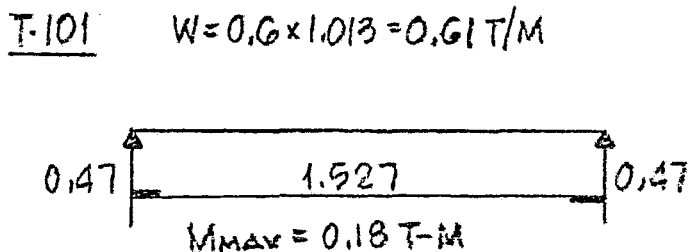
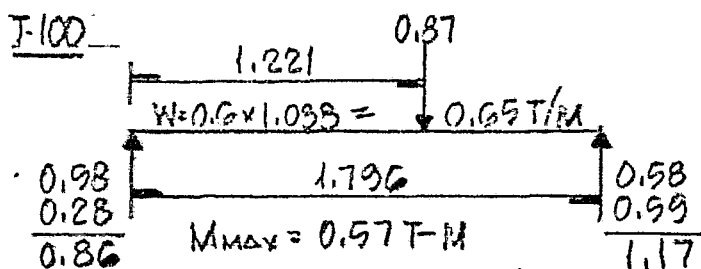
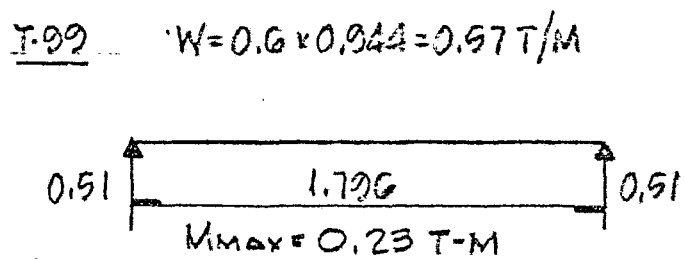
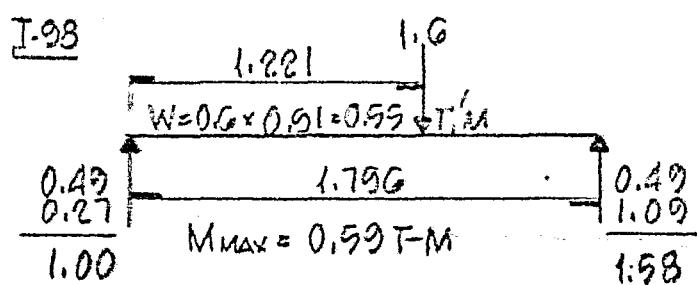
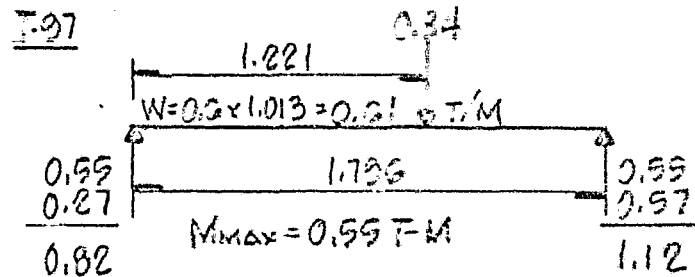
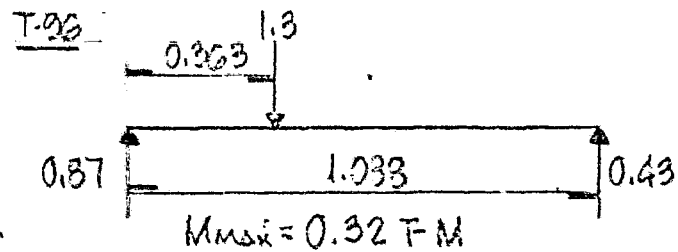
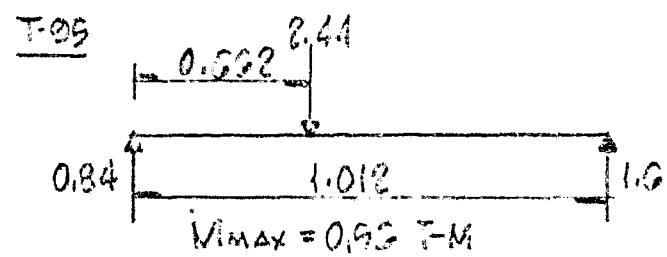
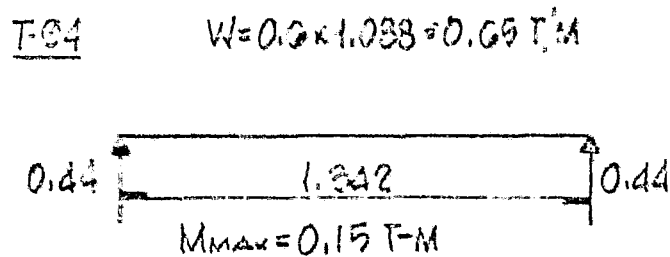


T-92  $W = 0.6 \times 0.2 = 0.12 T/M$

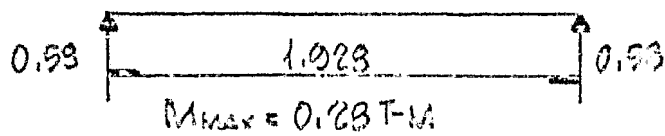


T-93  $W = 0.6 \times 0.844 = 0.5064 T/M$

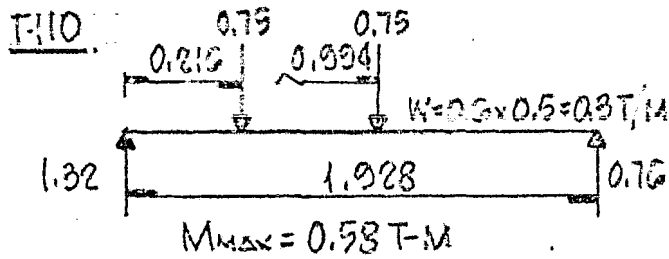
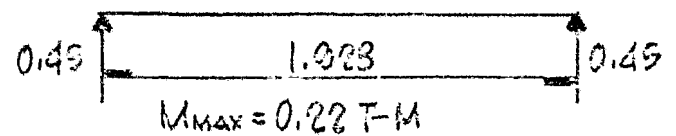




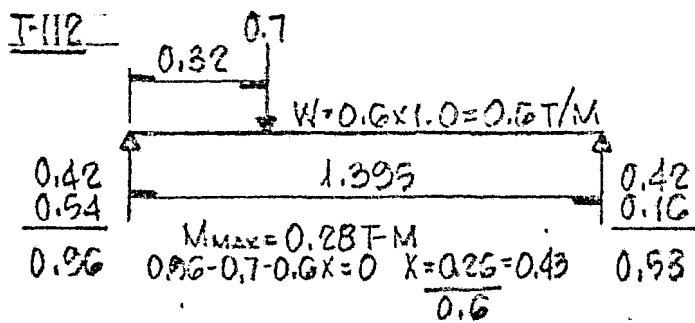
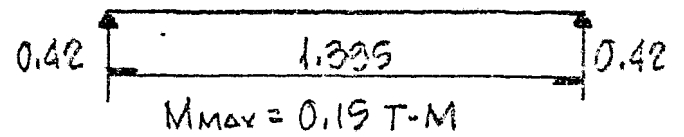
T-108  $W = 0.6 \times 1.0 = 0.6 T/M$



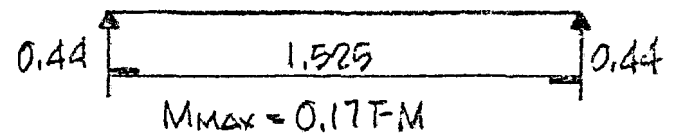
T-109  $W = 0.6 \times 0.79 = 0.47 T/M$



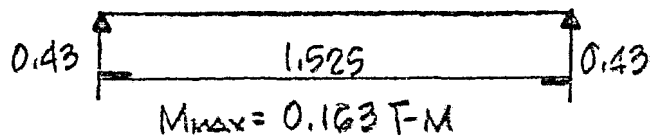
T-111  $W = 0.6 \times 1.0 = 0.6 T/M$



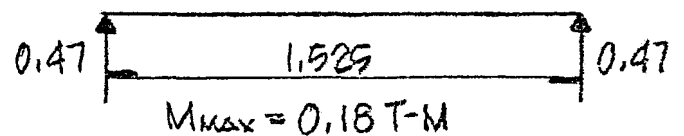
T-113  $W = 0.6 \times 0.96 = 0.58 T/M$



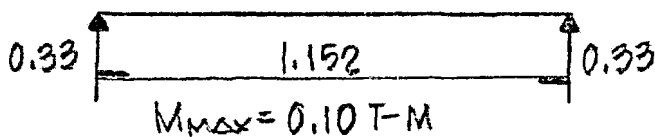
T-114  $W = 0.6 \times 0.94 = 0.56 T/M$



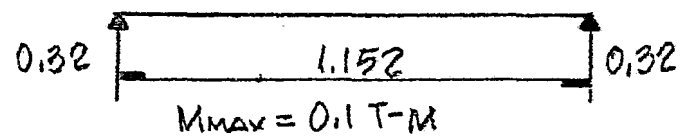
T-115  $W = 0.6 \times 1.025 = 0.62 T/M$



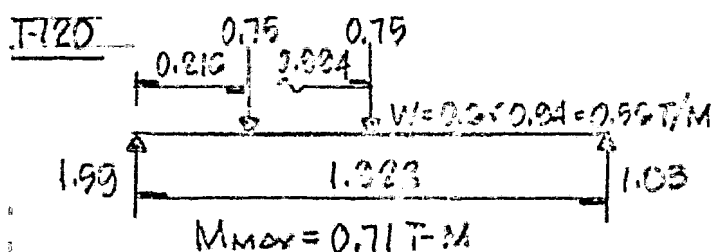
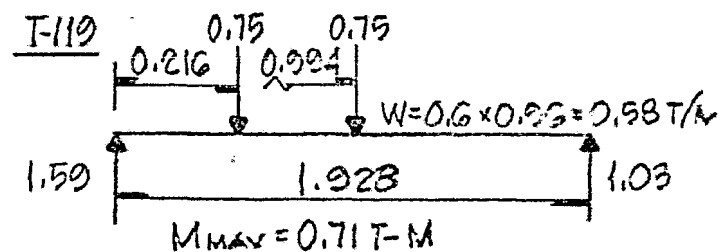
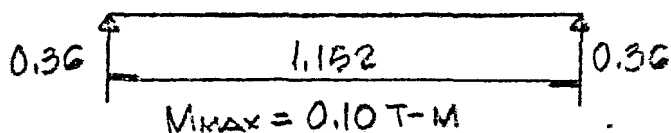
T-116  $W = 0.6 \times 0.96 = 0.58 T/M$



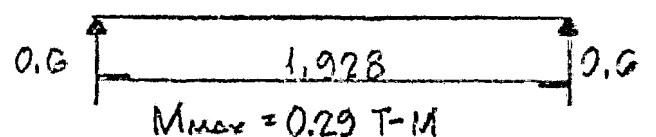
T-117  $W = 0.6 \times 0.94 = 0.56 T/M$



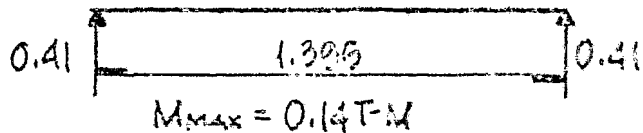
T-118  $W = 0.6 \times 1.025 = 0.62 T/M$



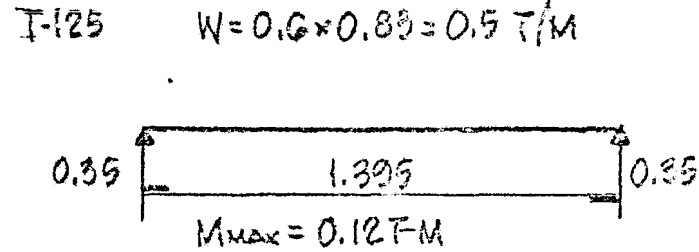
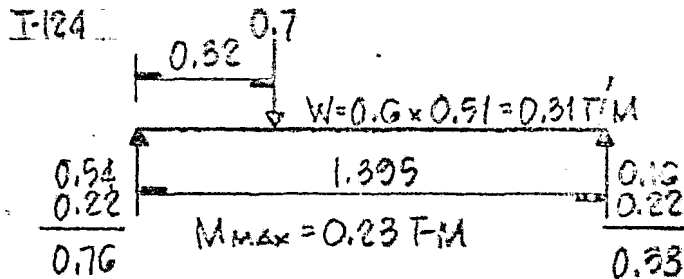
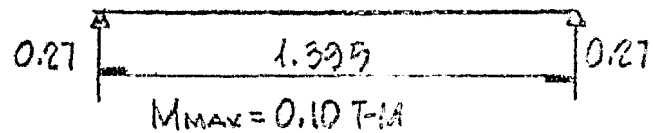
T-121  $W = 0.6 \times 1.025 = 0.62 T/M$



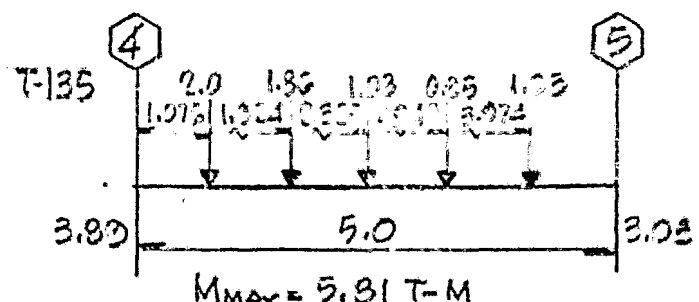
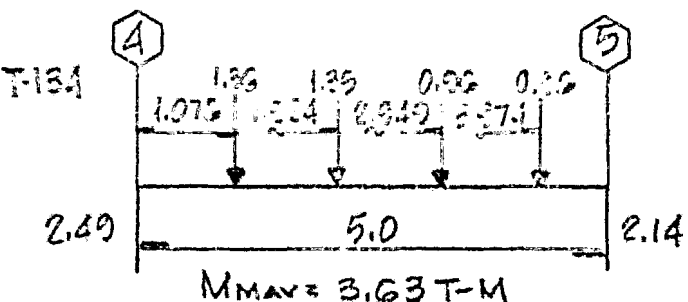
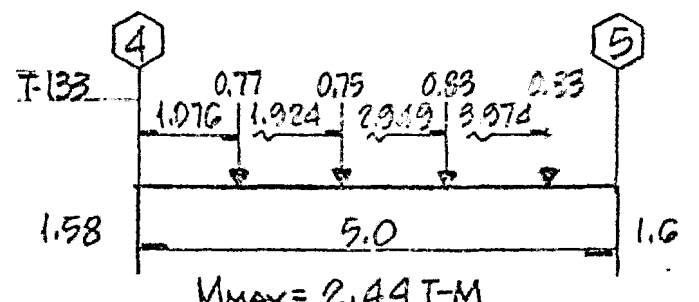
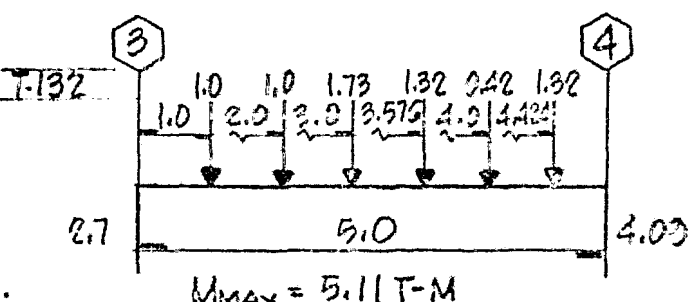
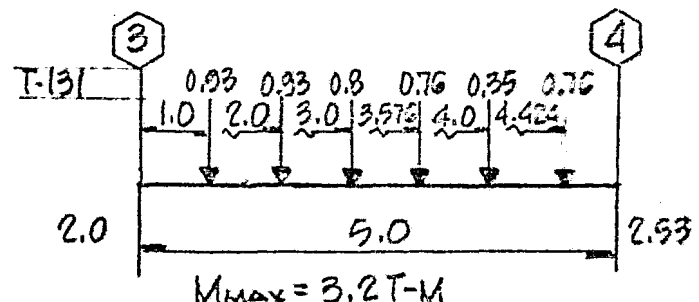
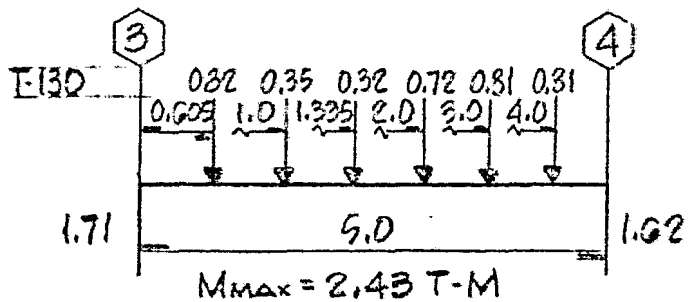
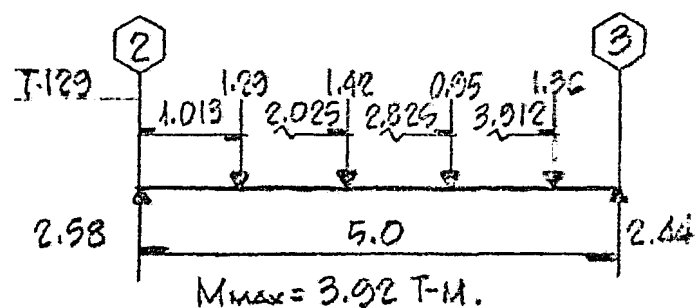
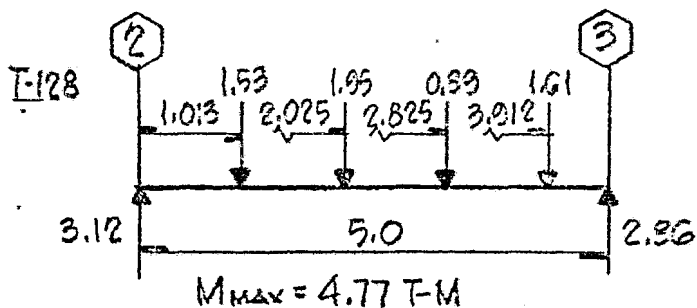
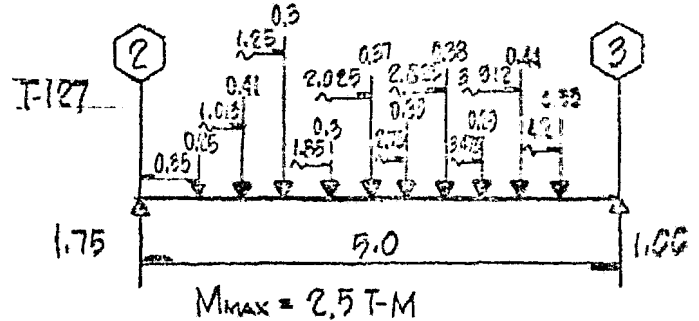
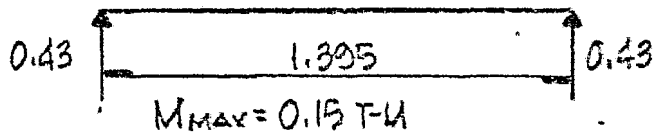
T-122  $W = 0.6 \times 0.95 = 0.58 T/M$



T-123  $W = 0.6 \times 0.63 = 0.38 T/M$



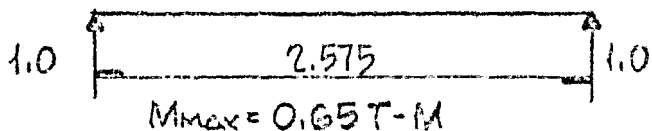
T-126  $W = 0.6 \times 1.025 = 0.62 T/M$



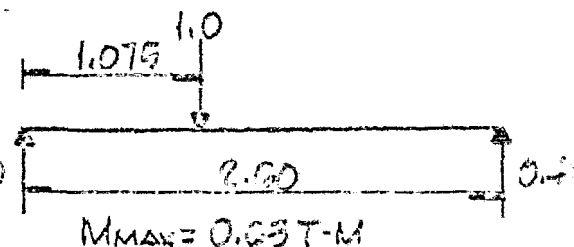
NIVEL T/S + 18.804

T-136

$W = 0.6 \times 1.3 = 0.78 T/M$

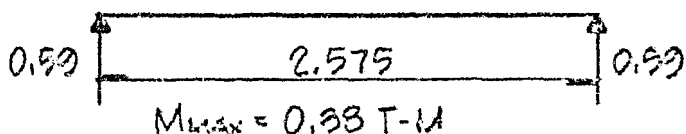


T-137



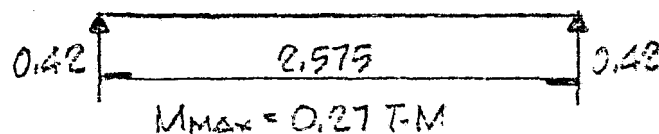
T-138

$W = 0.6 \times 0.76 = 0.46 T/M$



T-138A

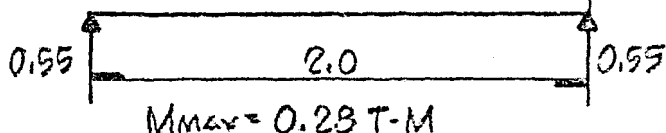
$W = 0.6 \times 0.94 = 0.56 T/M$



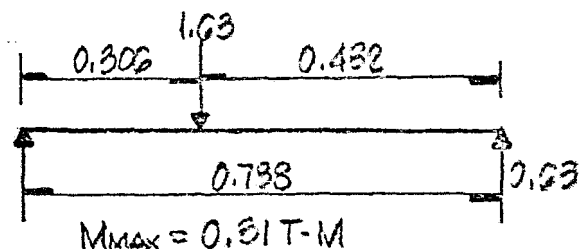
NIVEL T/S + 17.604

T-139

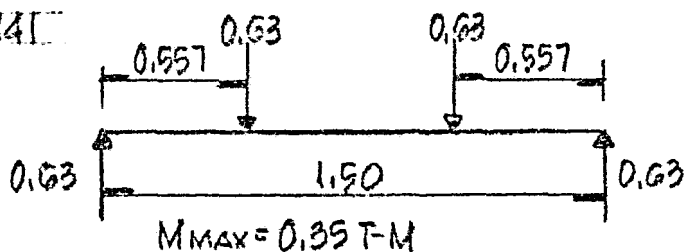
$W = 0.6 \times 0.917 = 0.55 T/M$



T-140

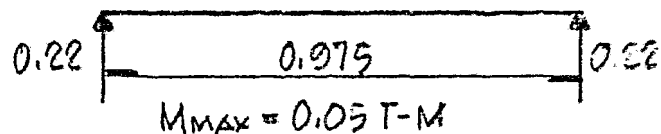


T-141

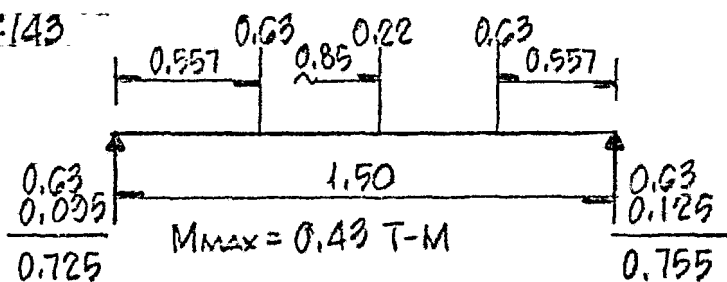


T-142

$W = 0.6 \times 0.75 = 0.45 T/M$

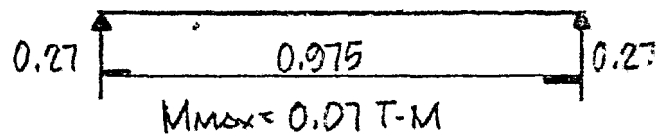


T-143

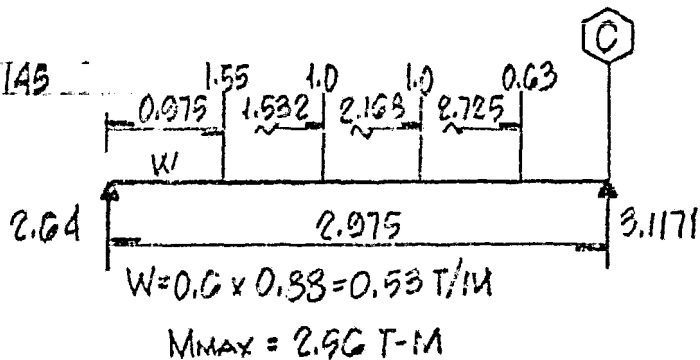


T-144

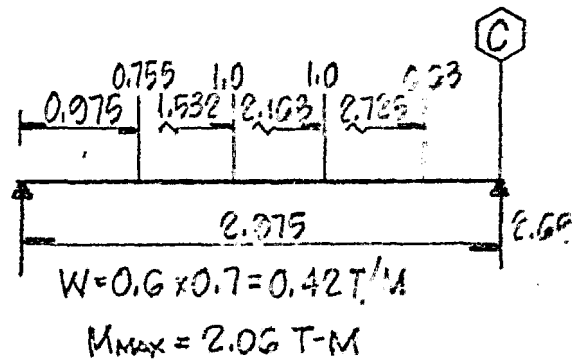
$W = 0.6 \times 0.917 = 0.55$



T-145

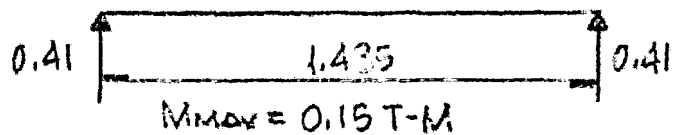


T-146

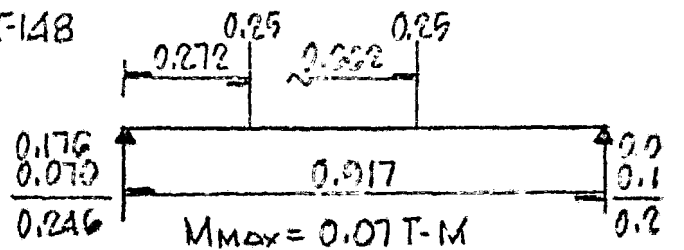


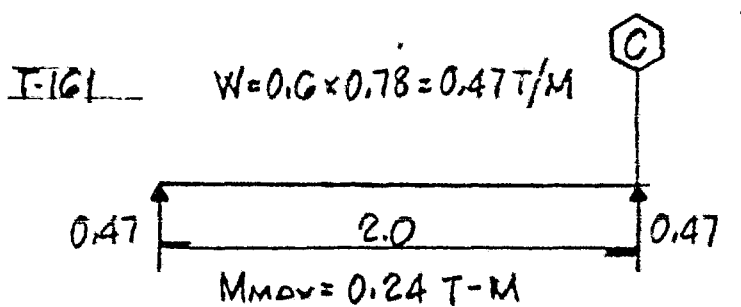
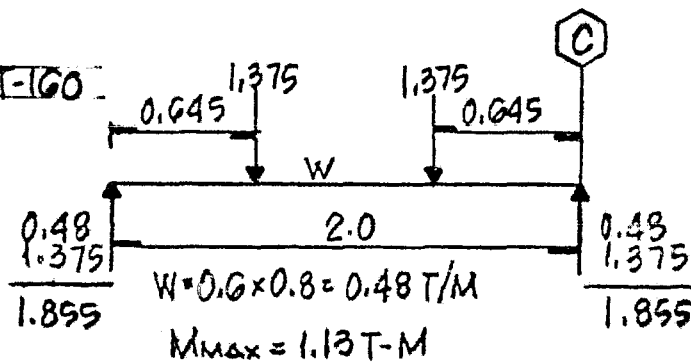
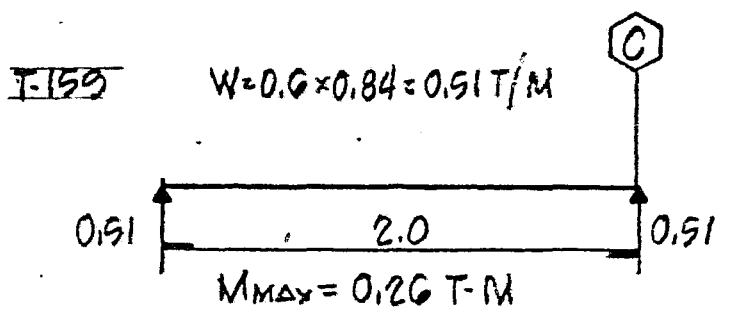
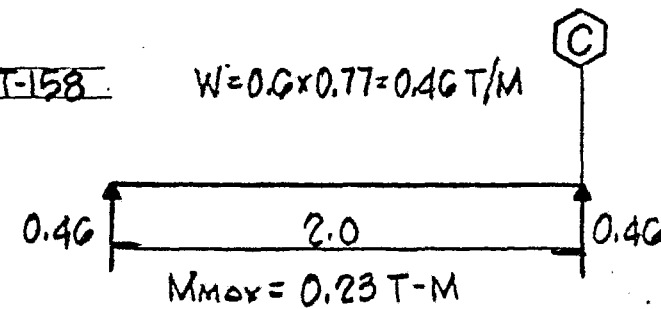
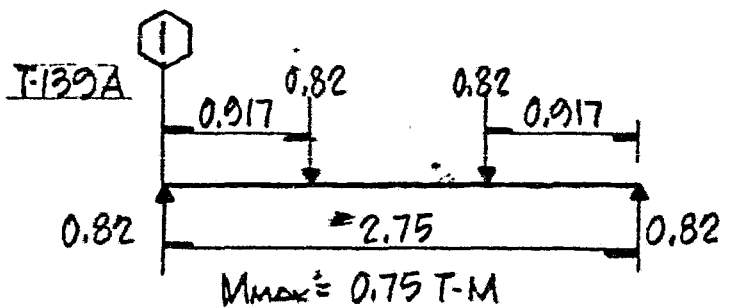
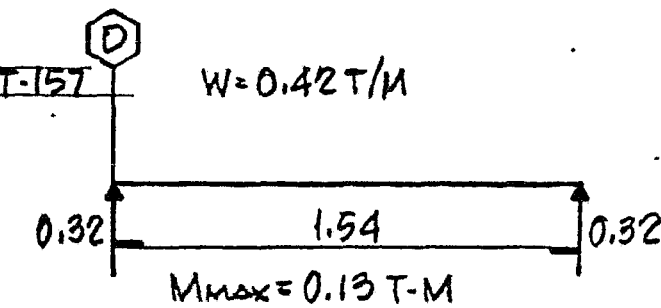
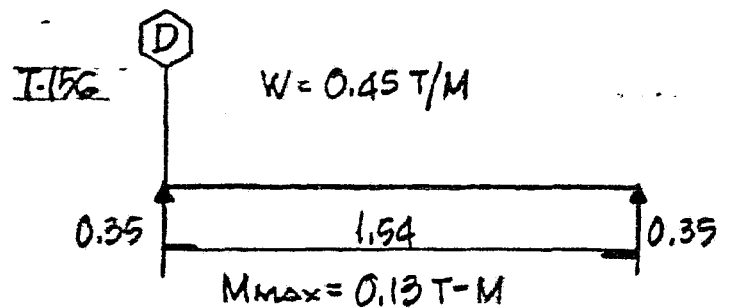
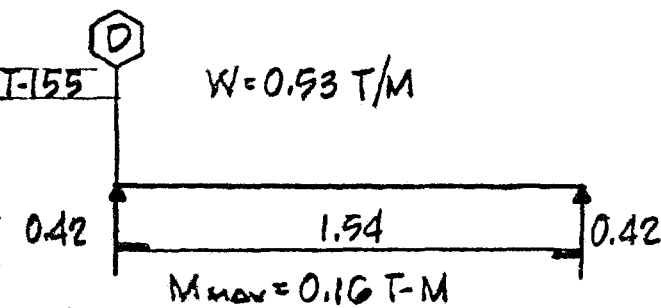
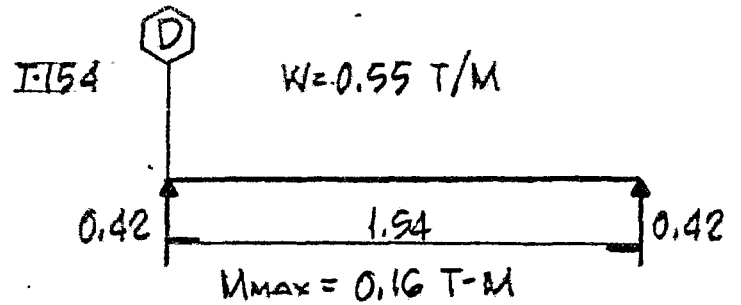
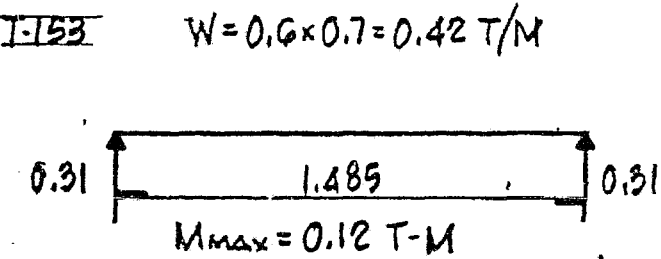
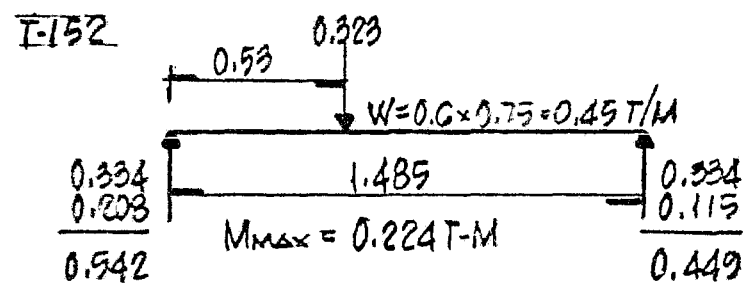
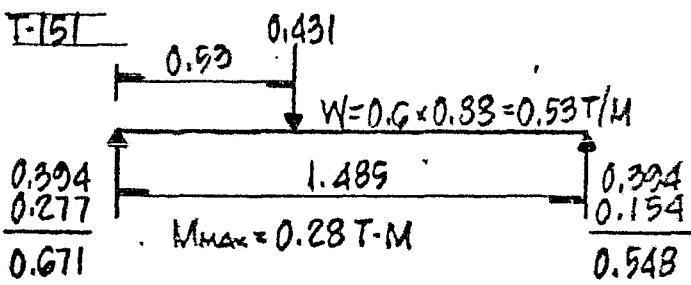
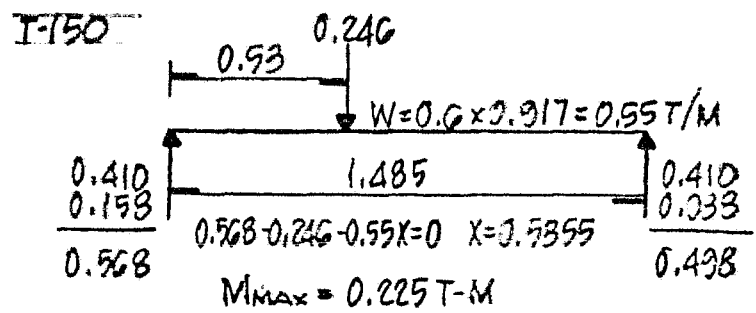
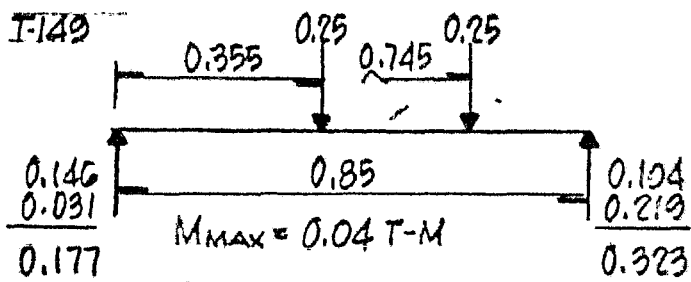
T-147

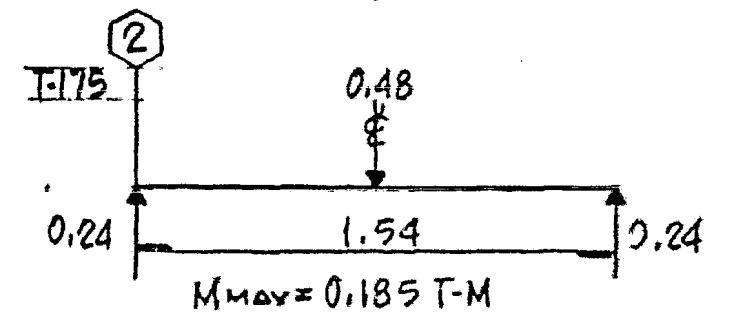
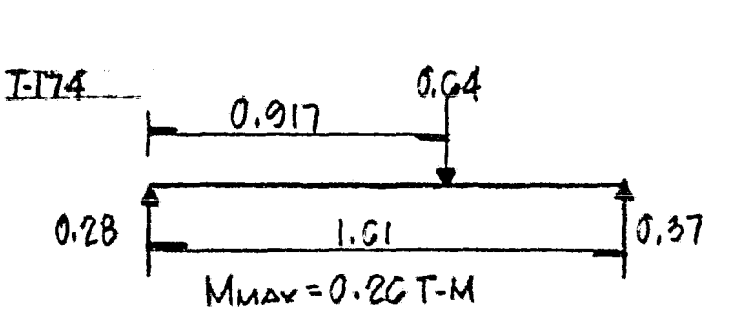
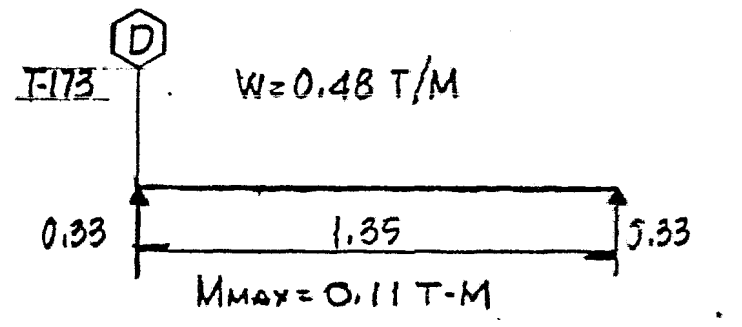
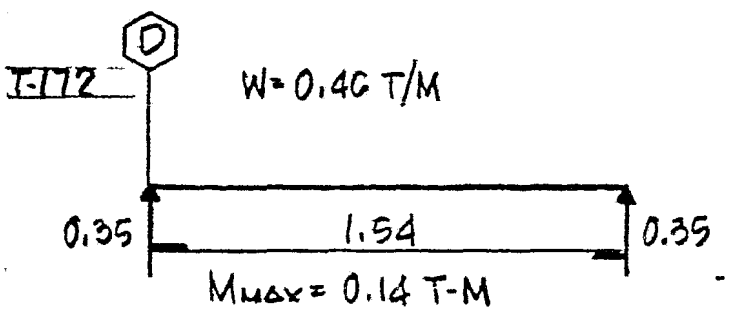
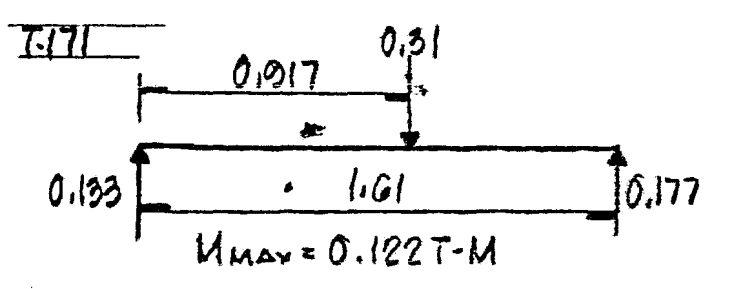
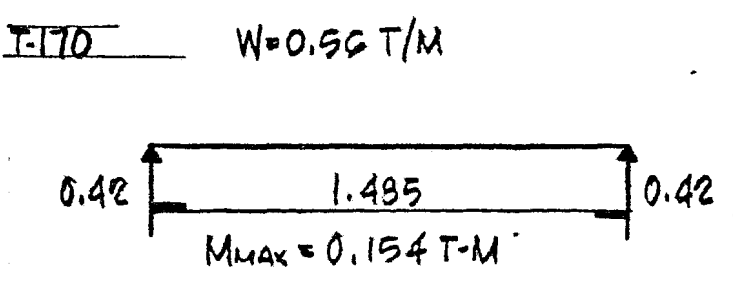
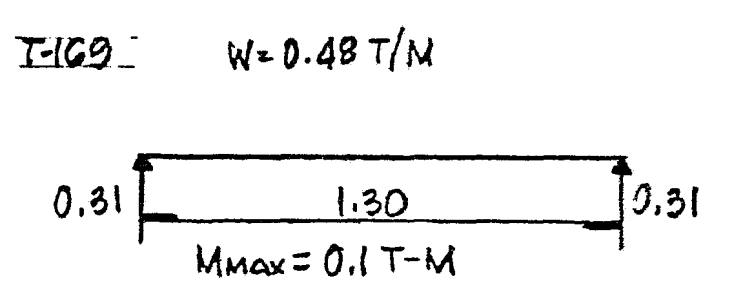
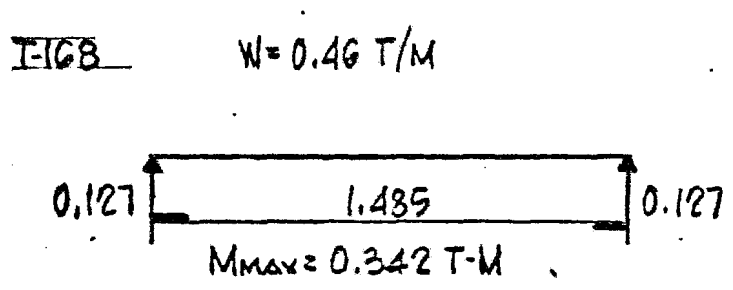
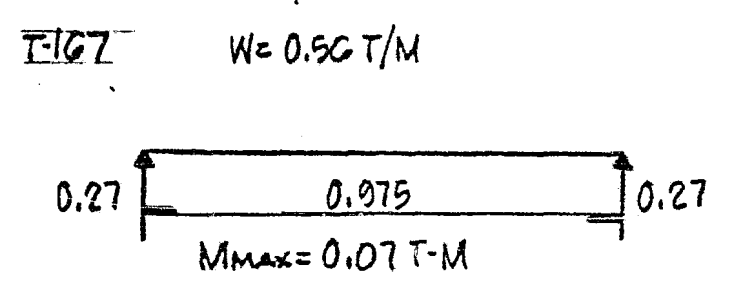
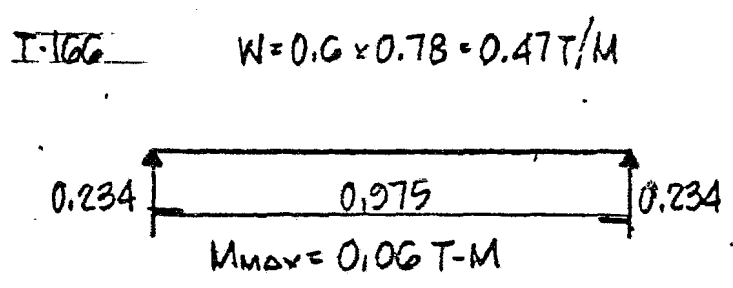
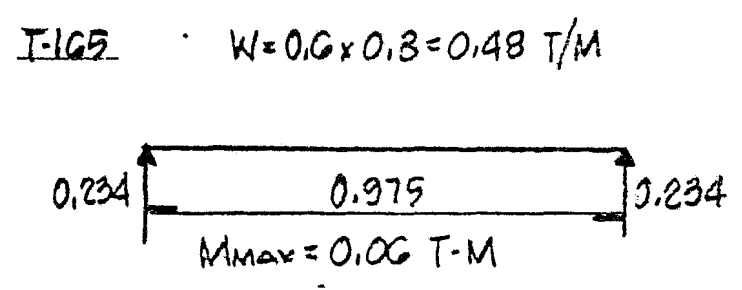
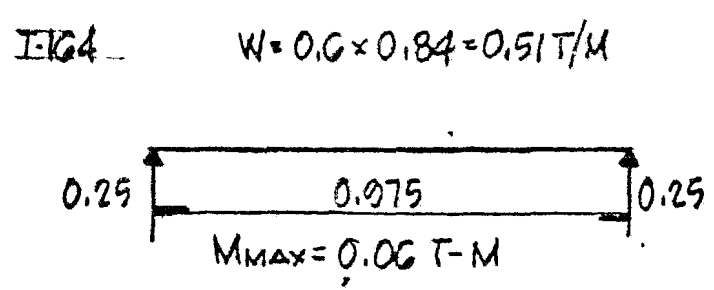
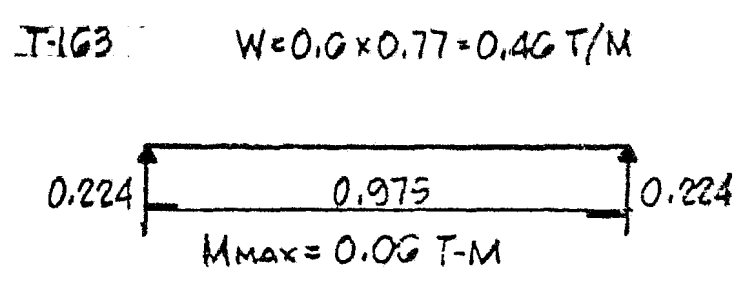
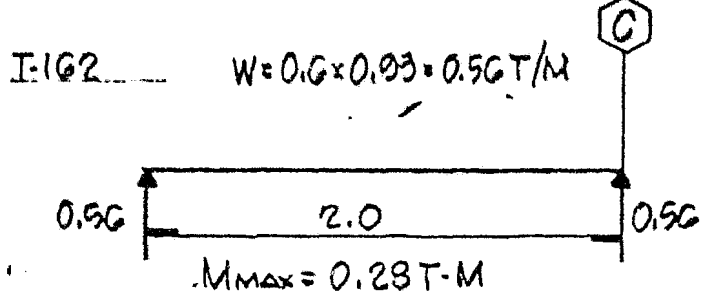
$W = 0.6 \times 0.917 = 0.55 T/M$



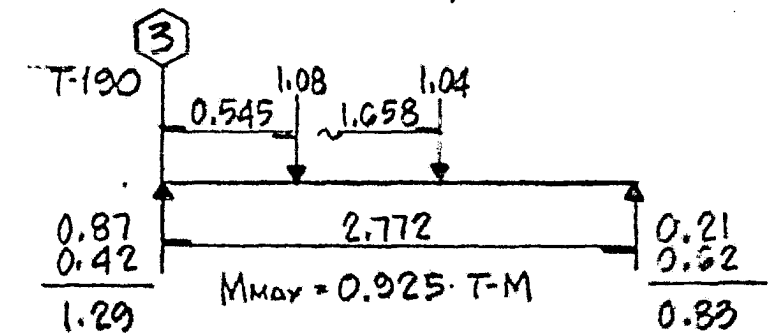
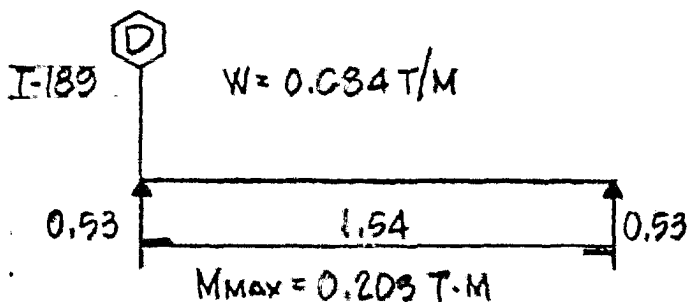
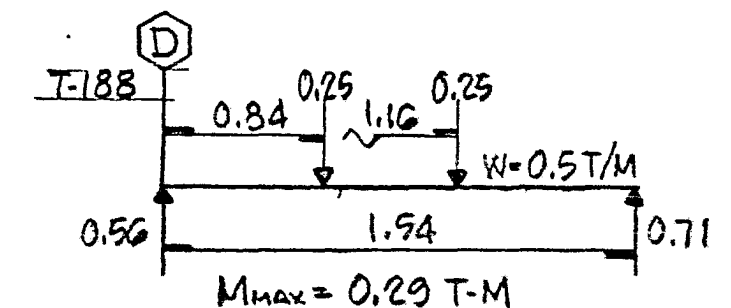
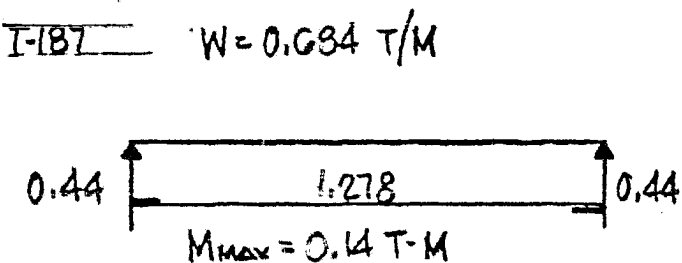
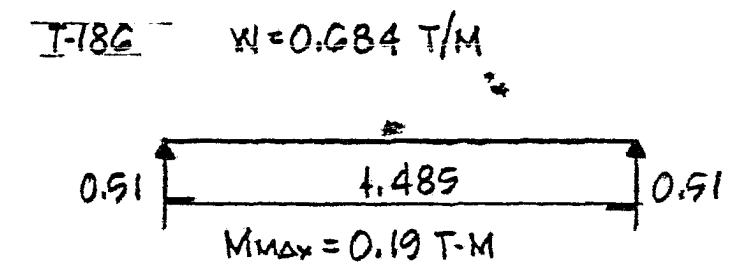
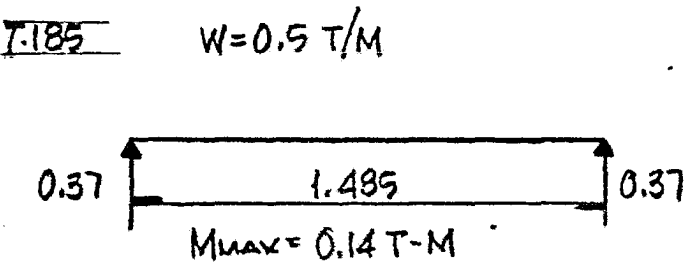
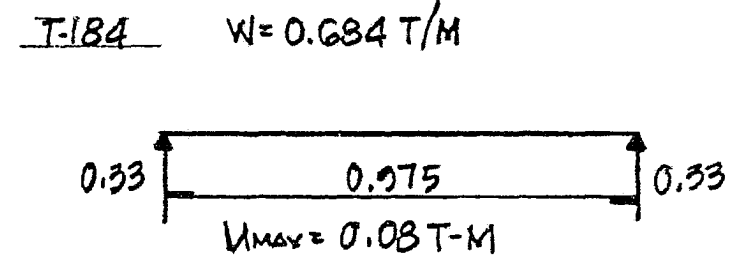
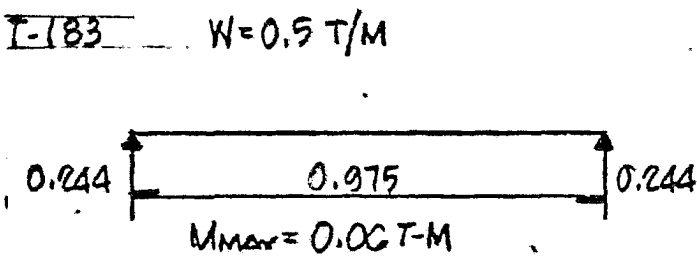
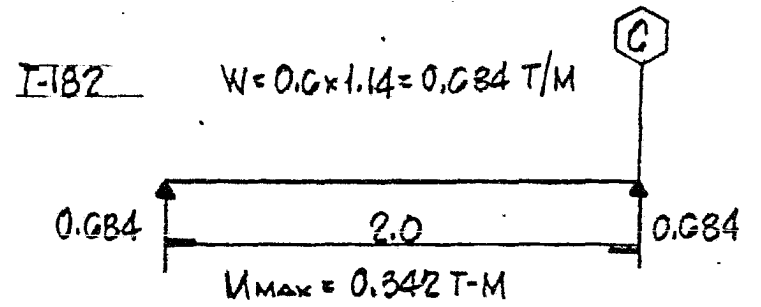
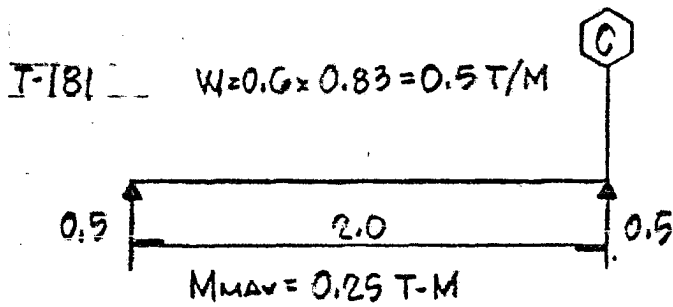
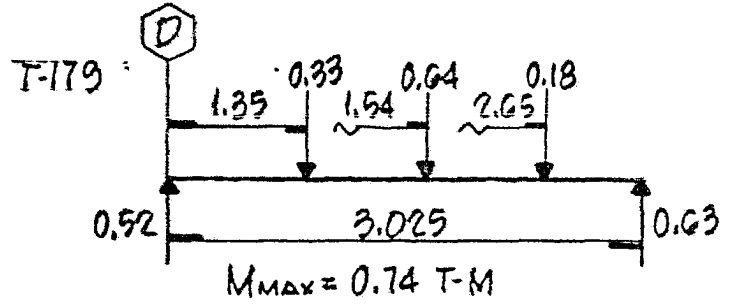
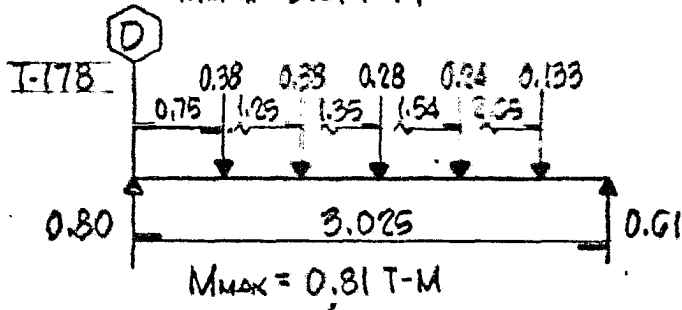
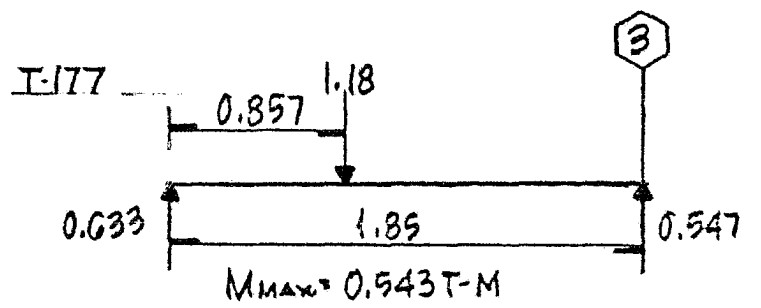
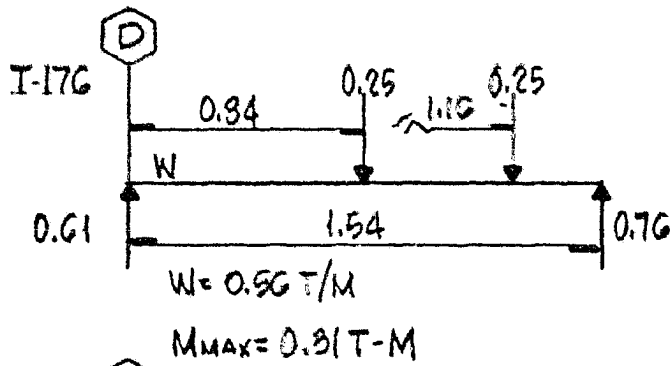
T-148

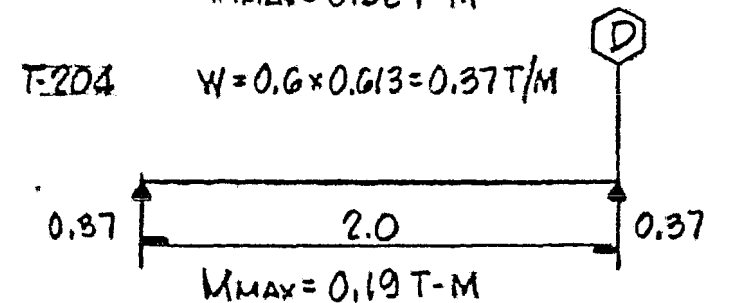
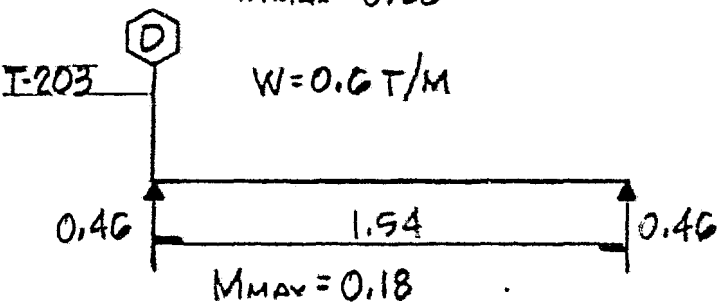
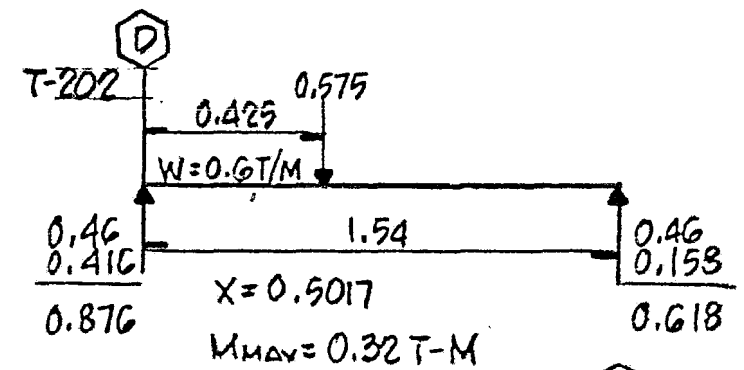
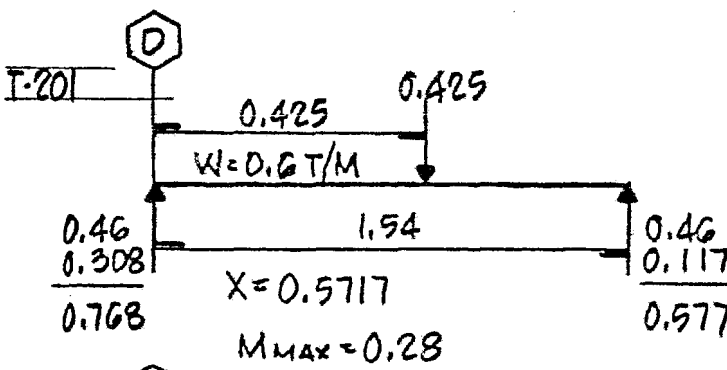
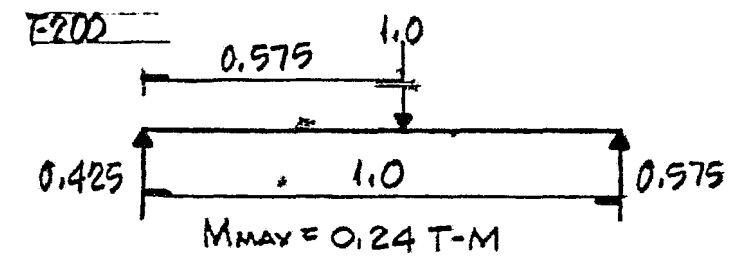
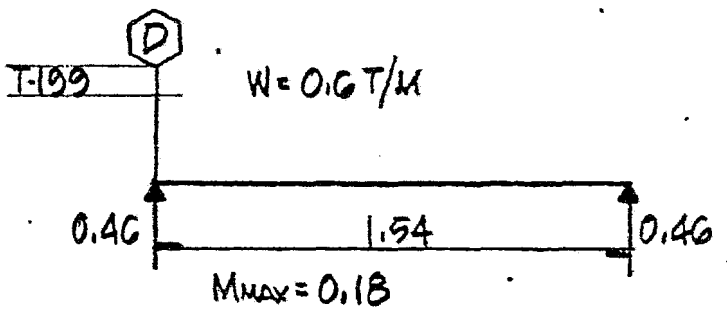
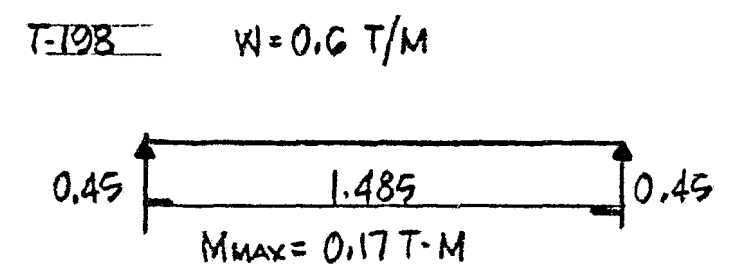
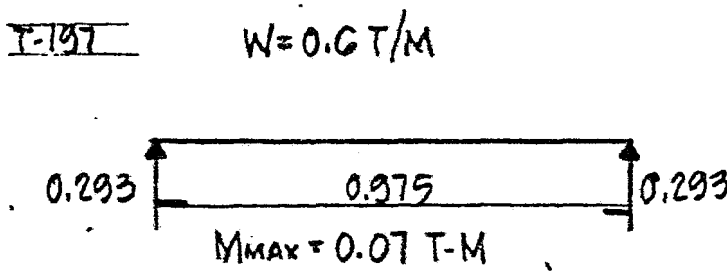
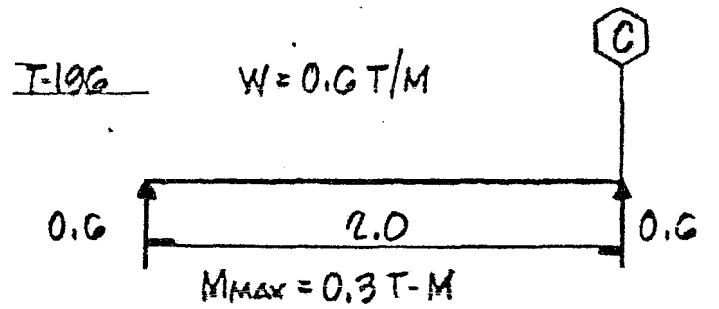
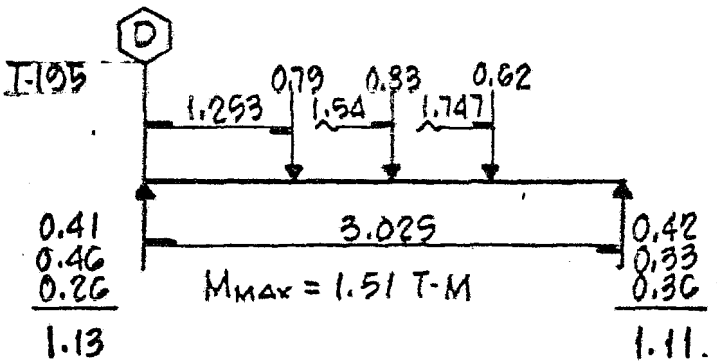
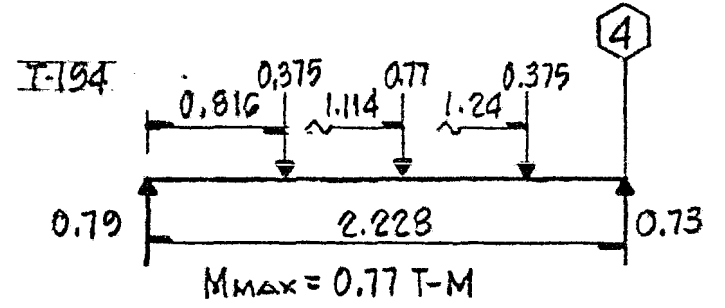
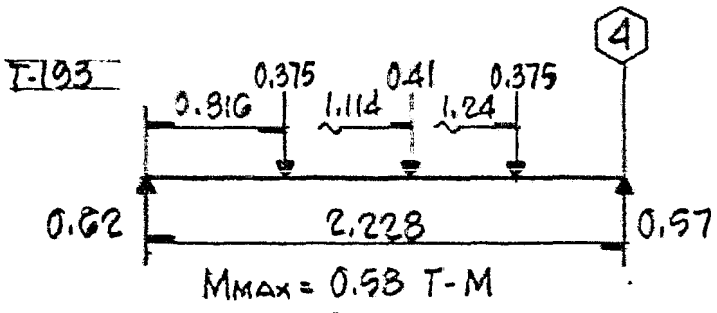
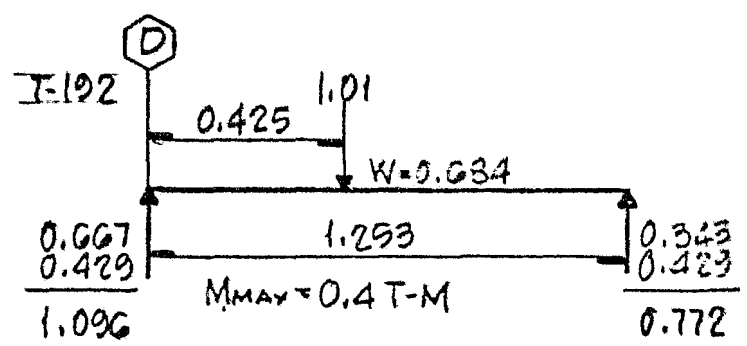
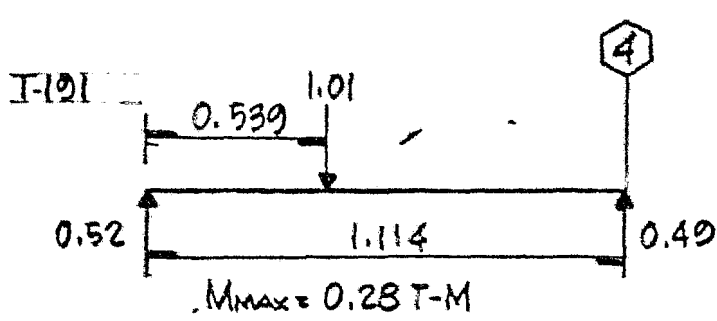


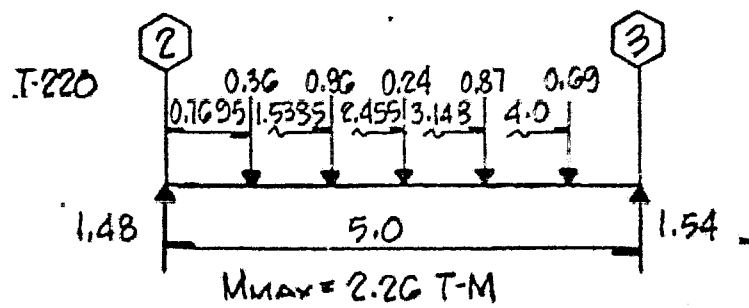
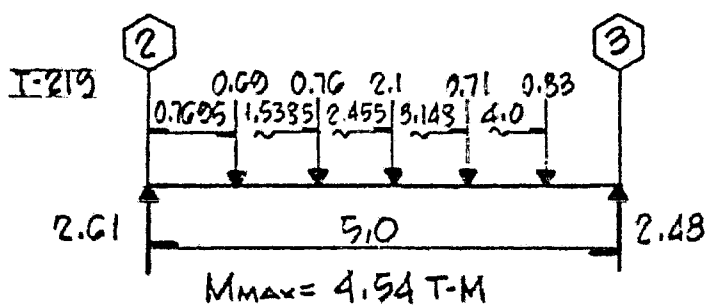
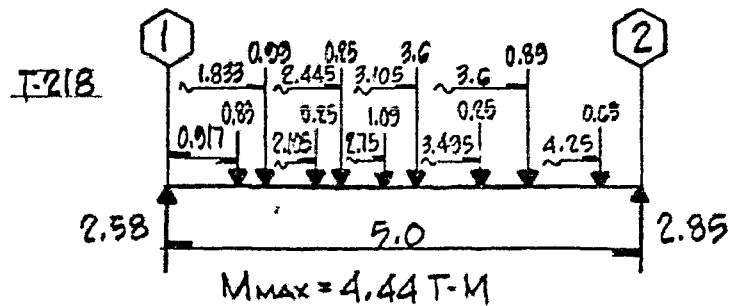
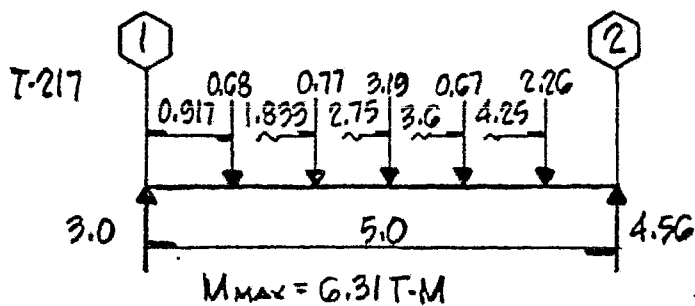
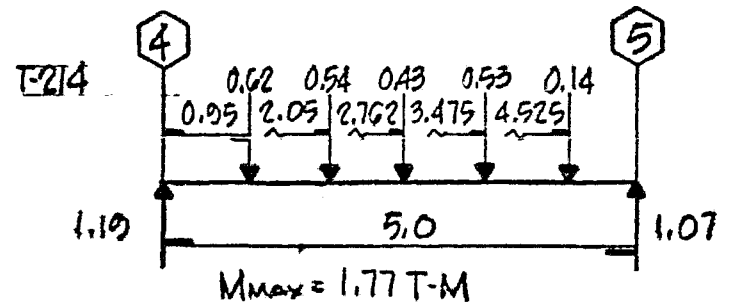
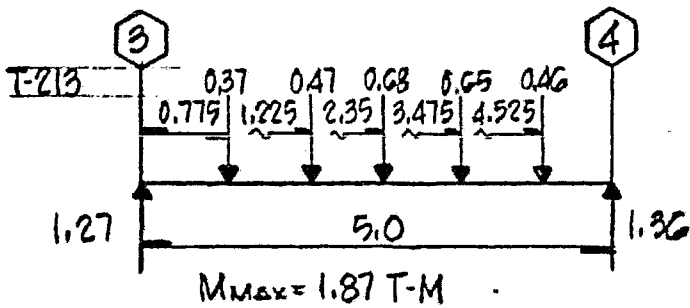
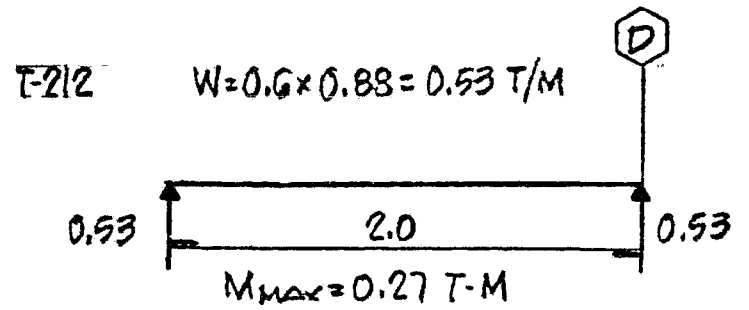
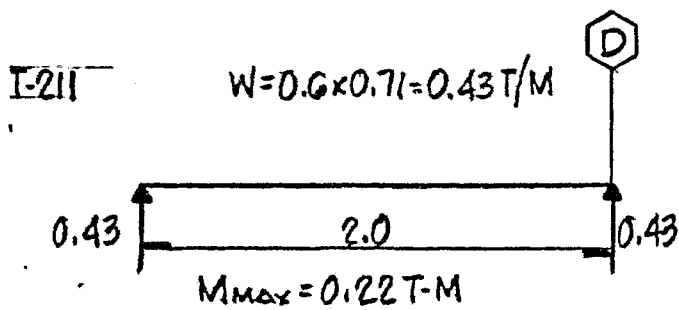
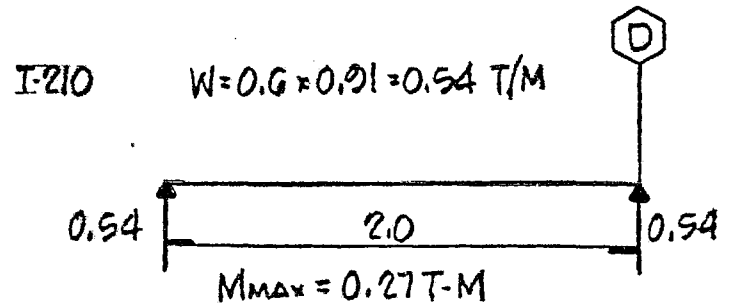
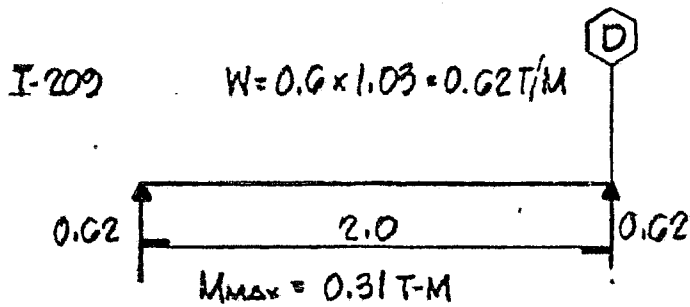
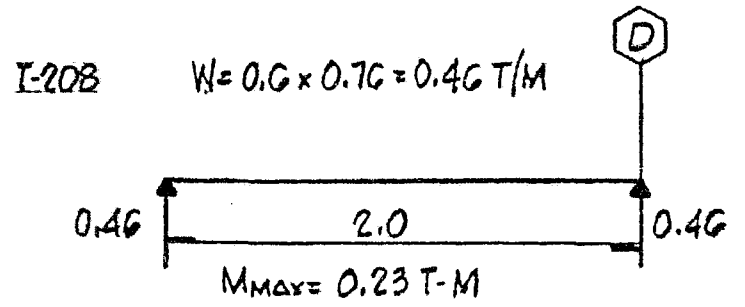
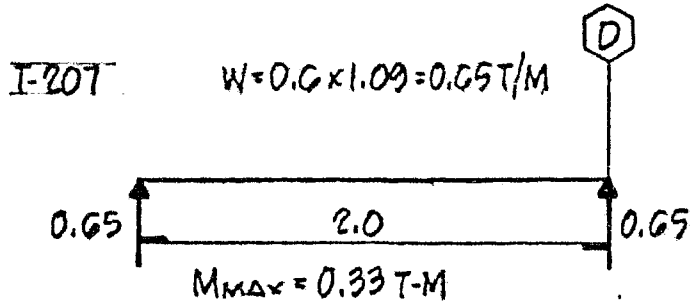
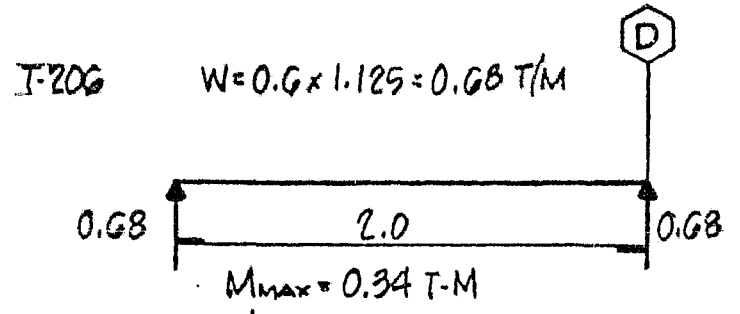
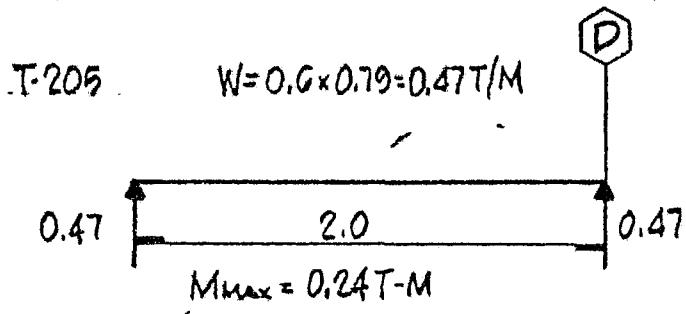


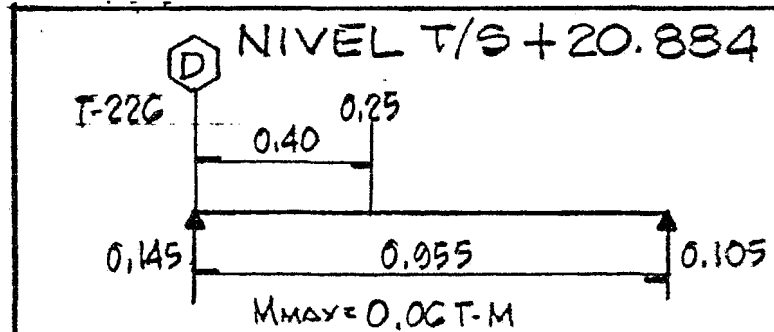
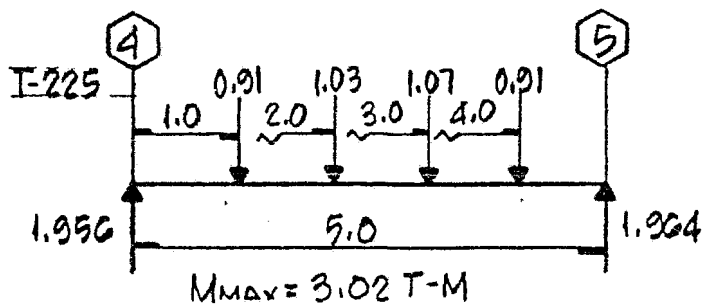
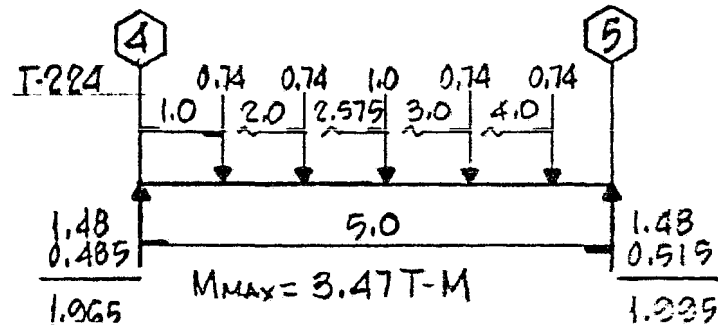
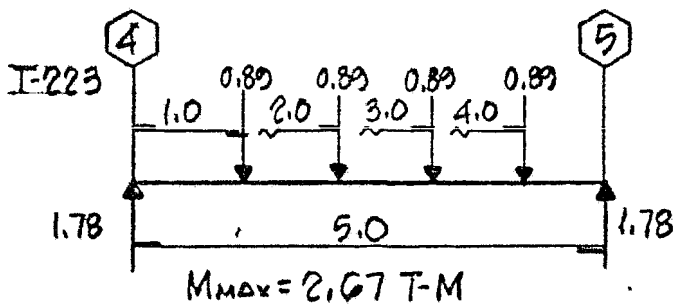
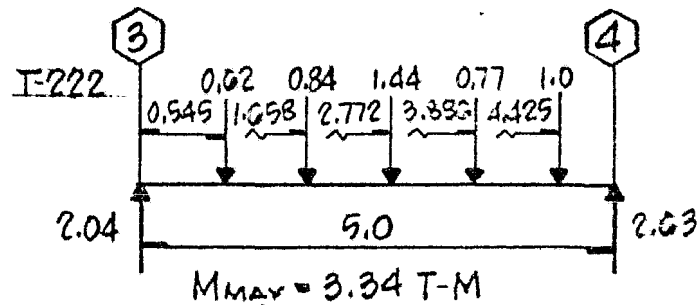
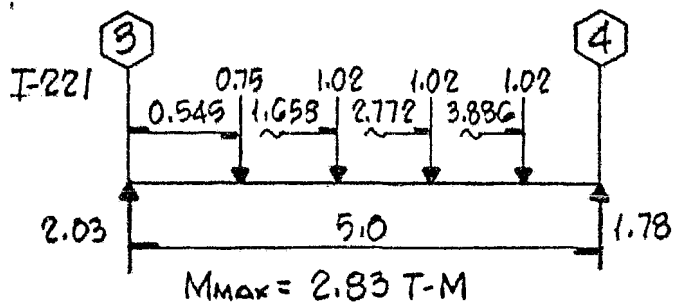




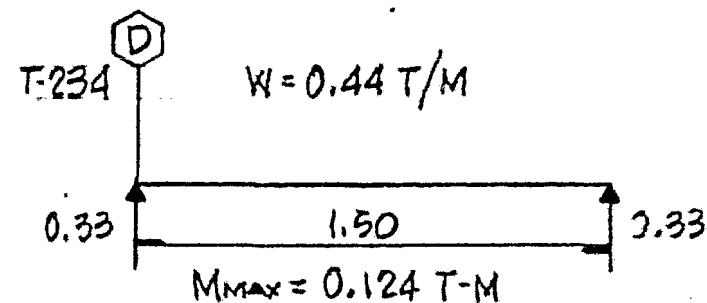
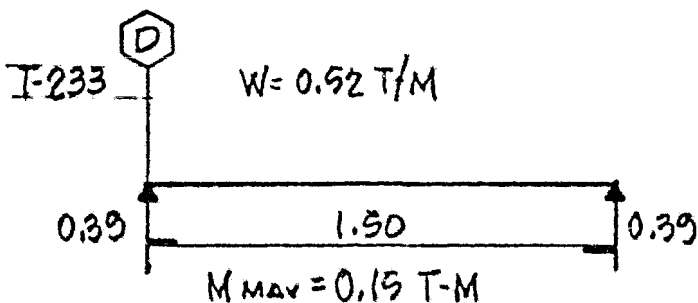
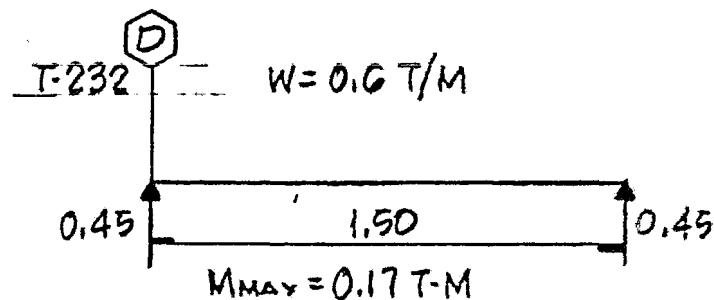
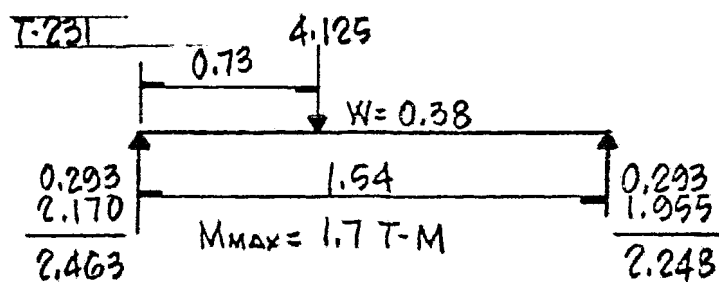
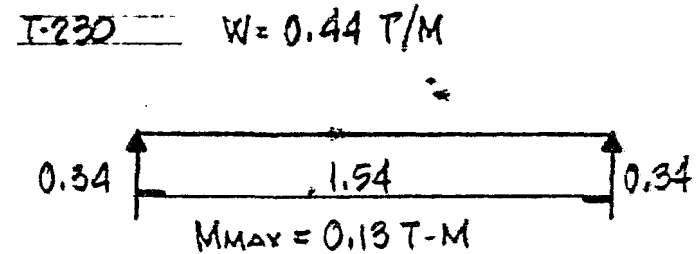
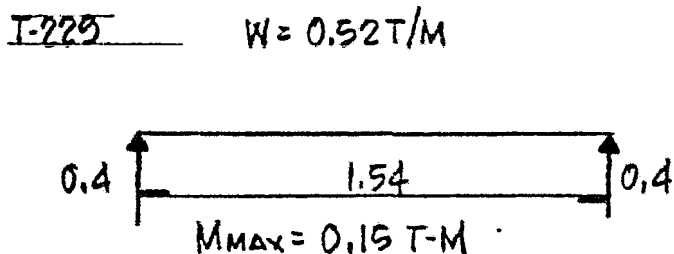
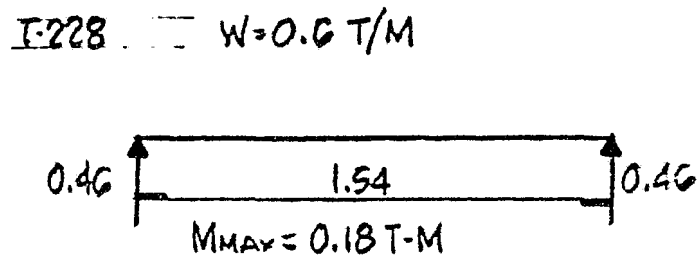
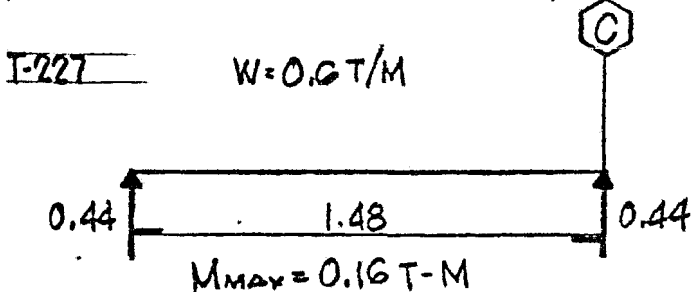


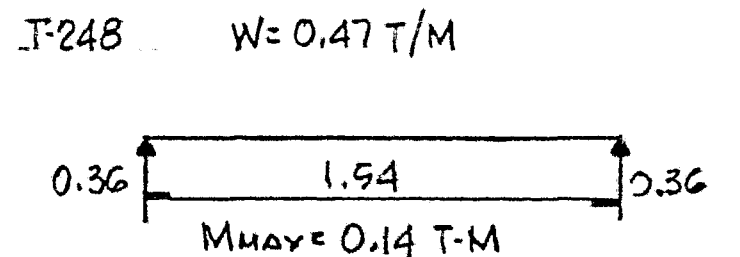
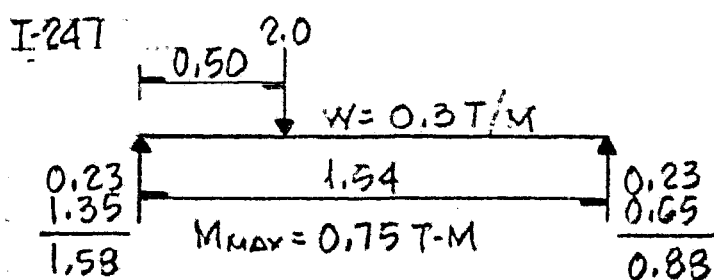
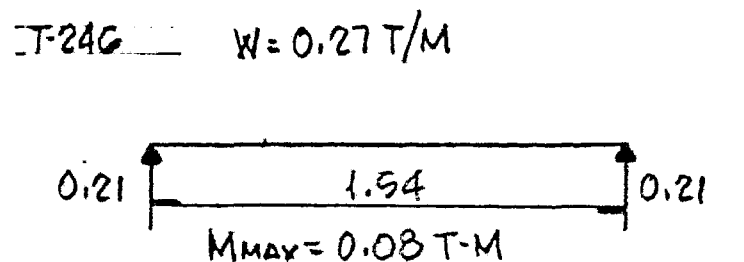
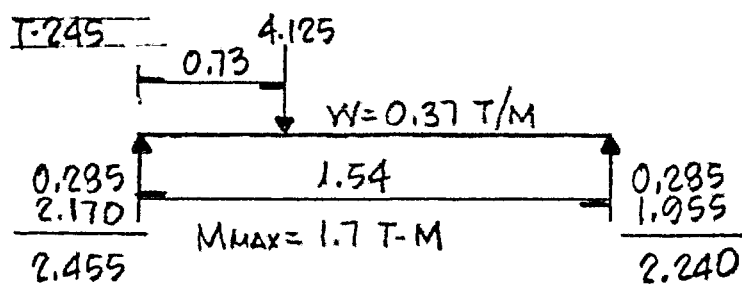
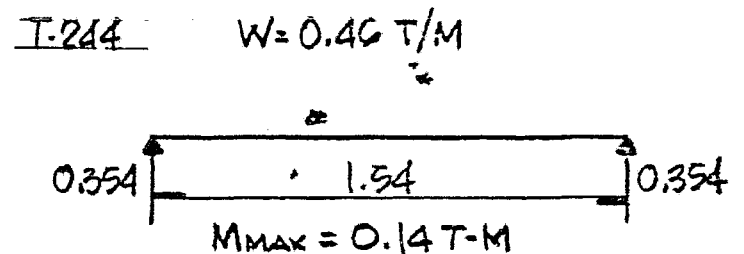
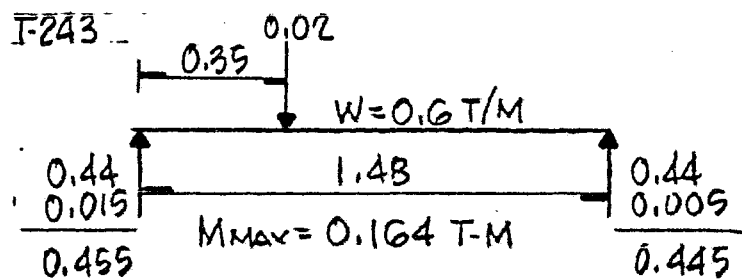
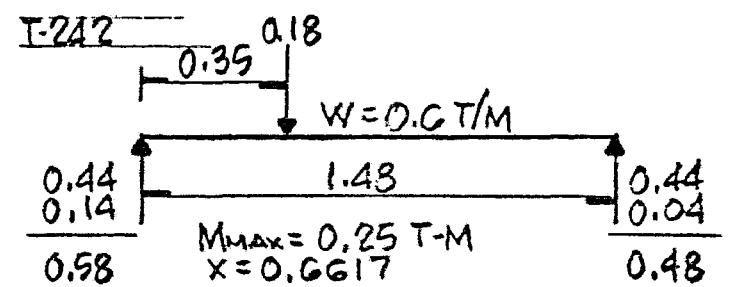
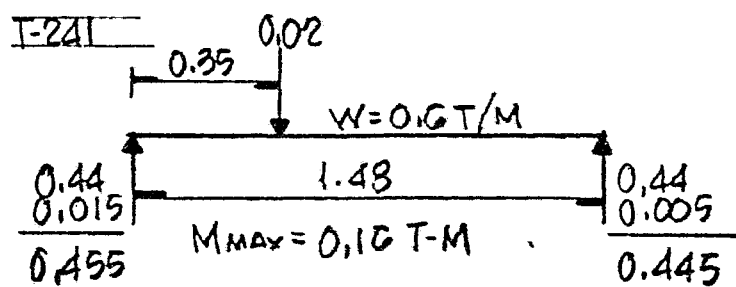
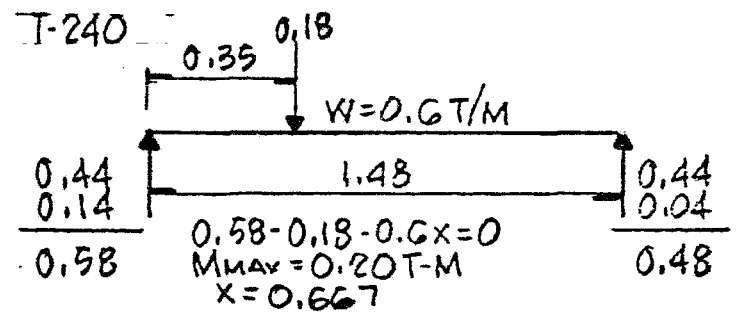
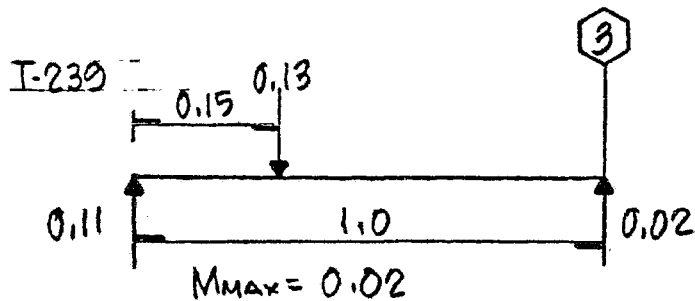
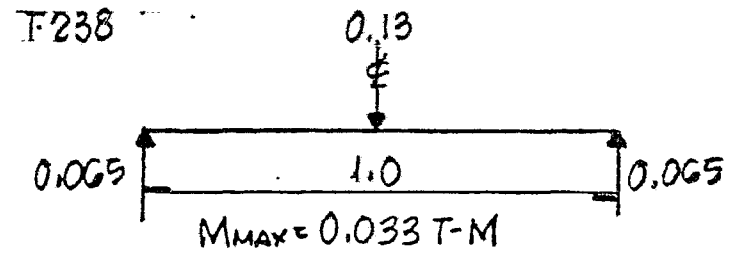
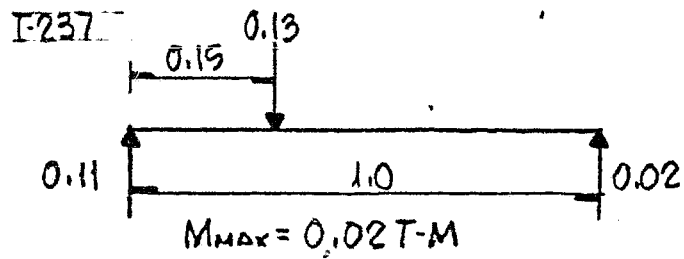
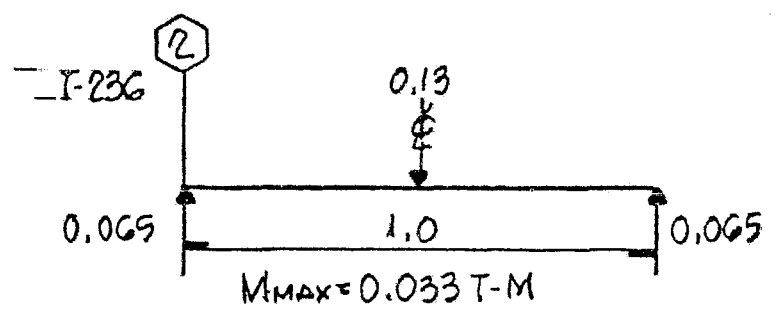
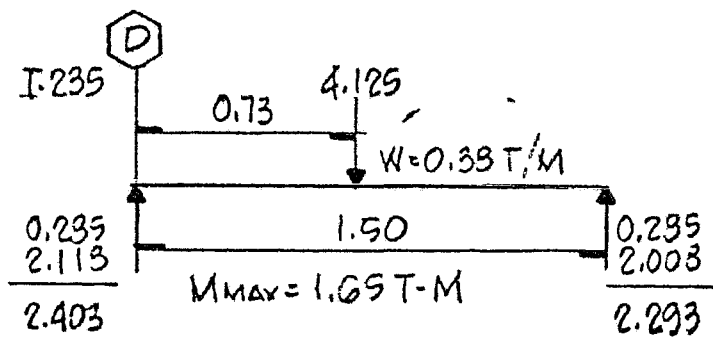


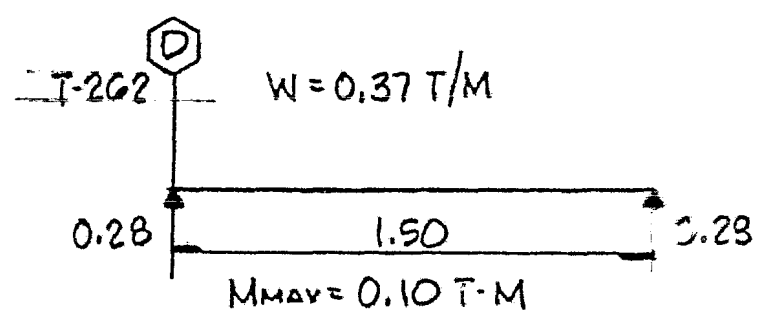
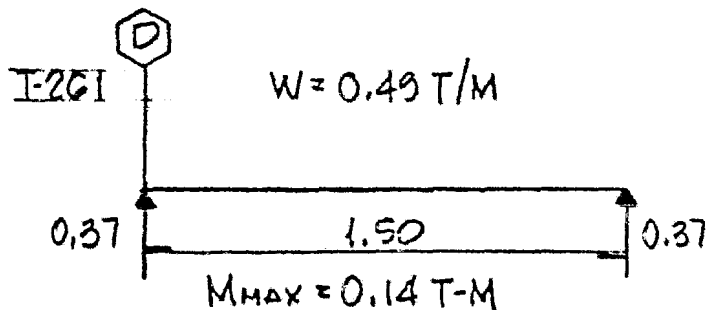
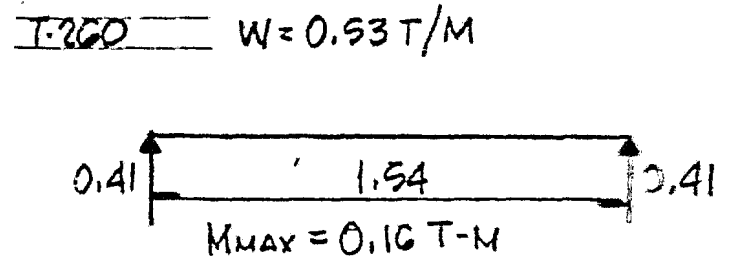
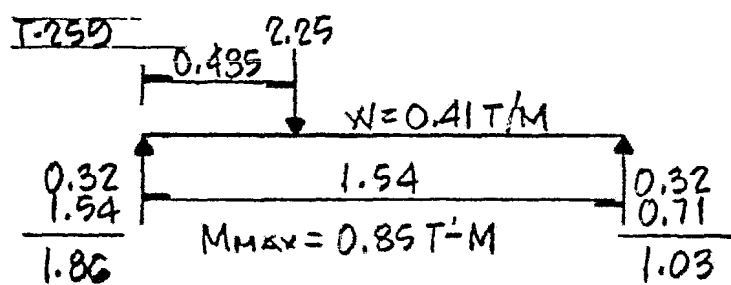
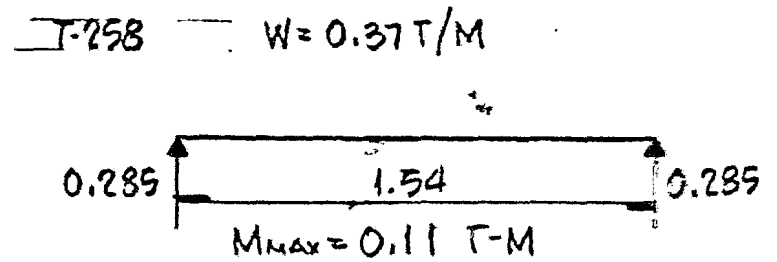
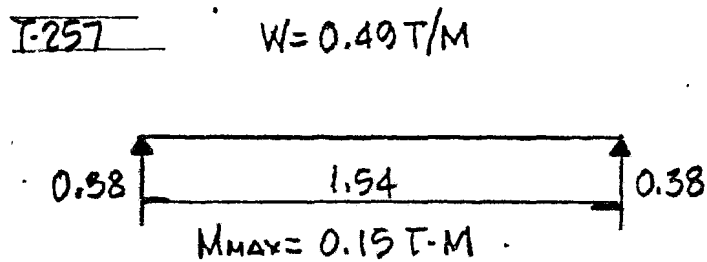
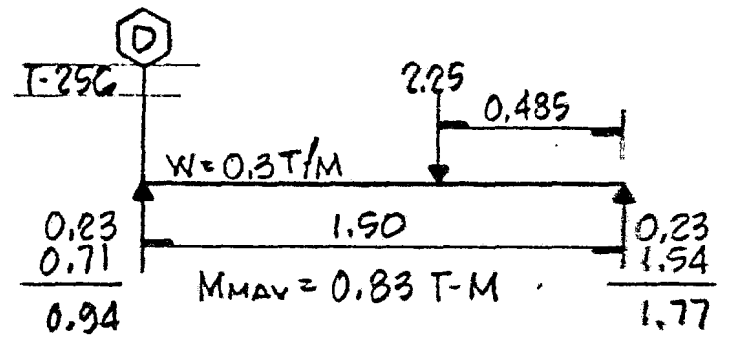
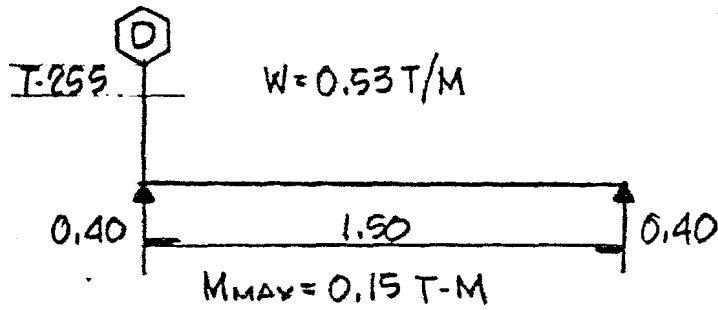
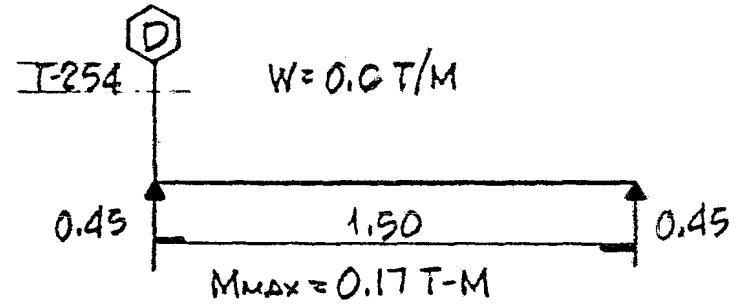
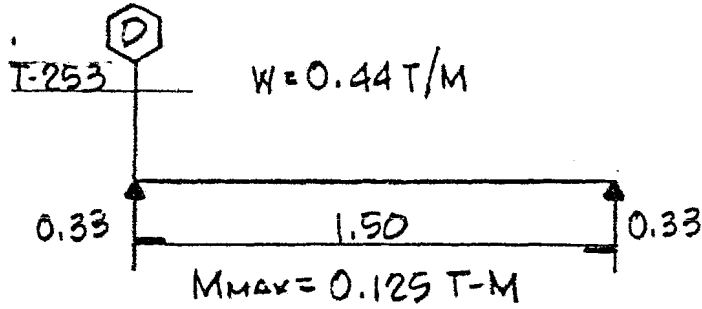
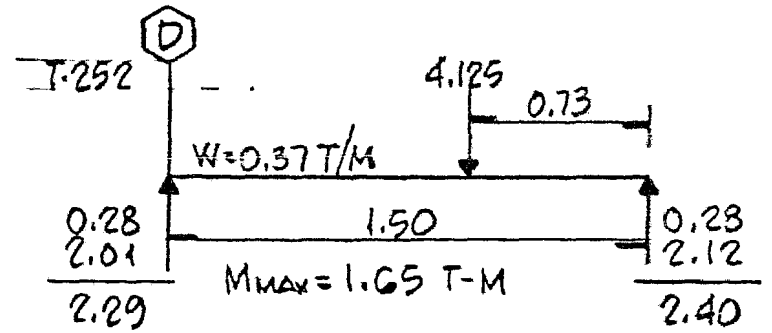
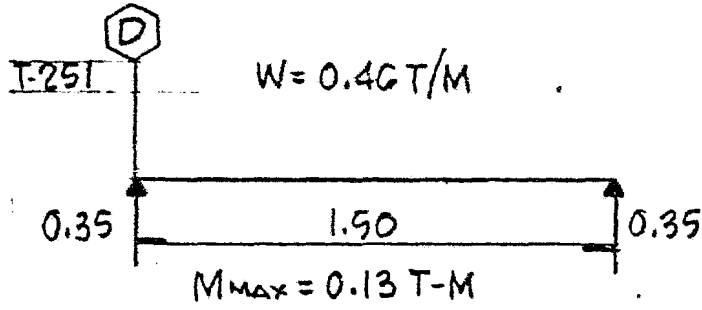
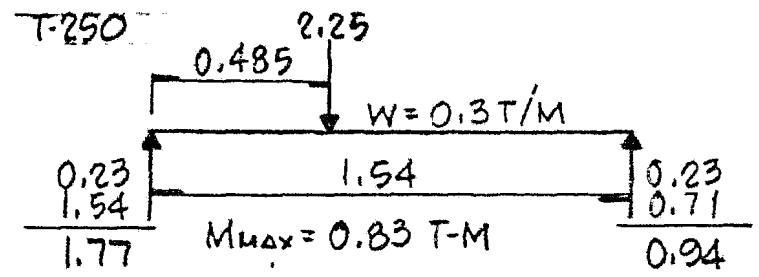
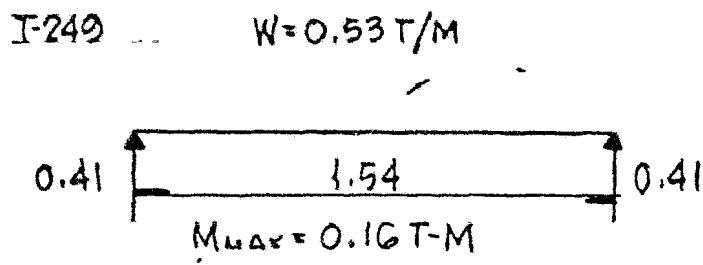


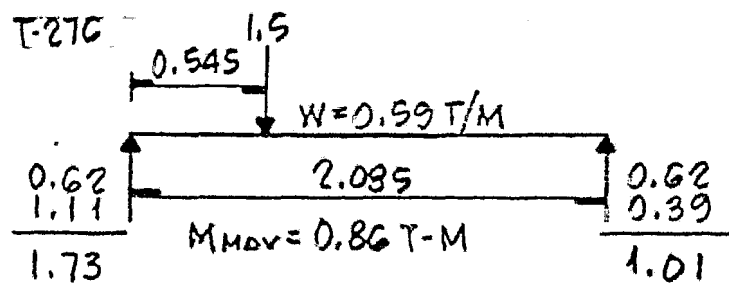
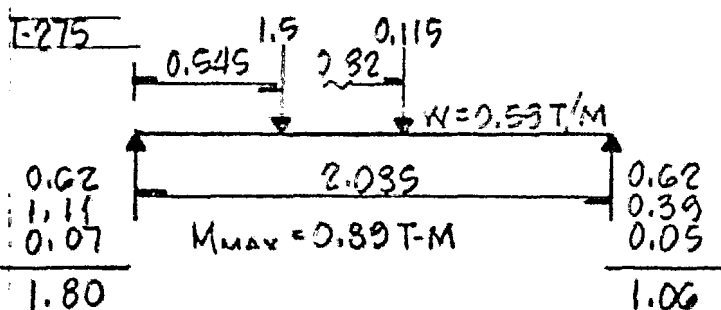
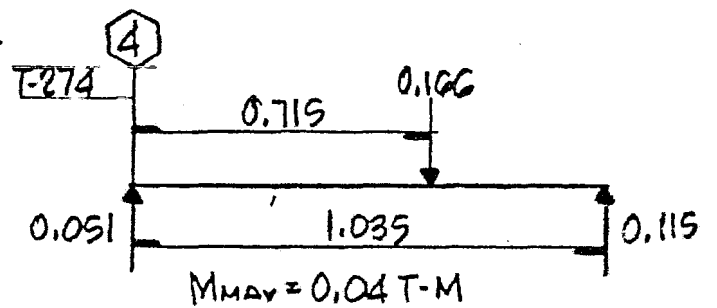
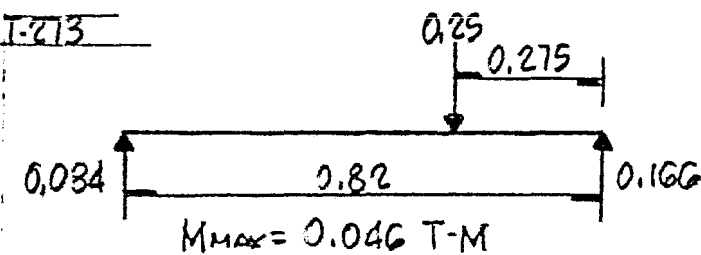
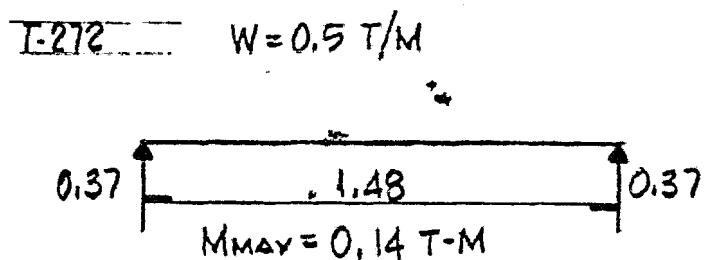
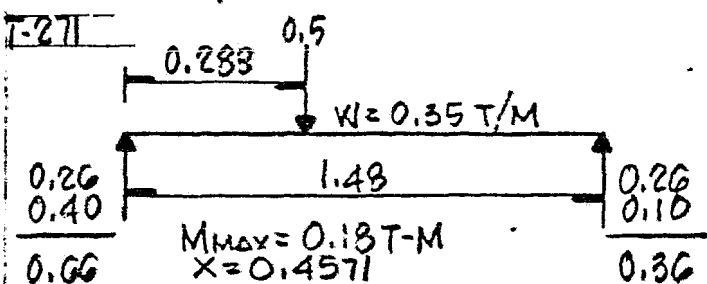
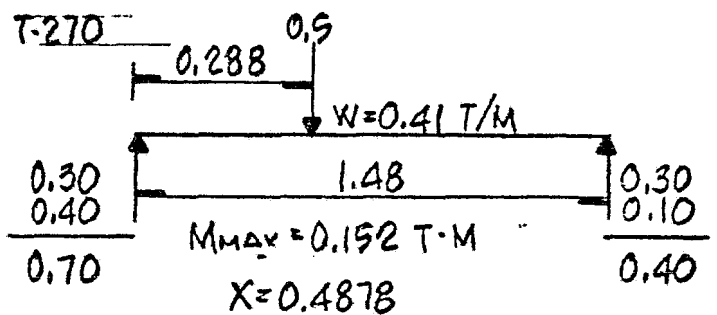
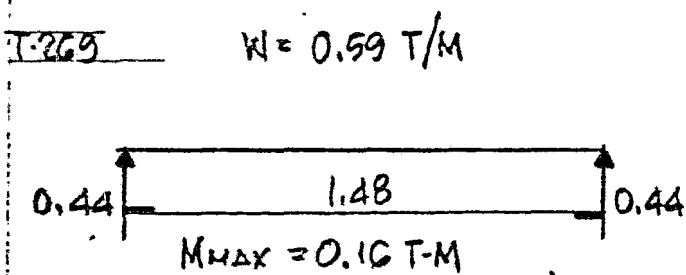
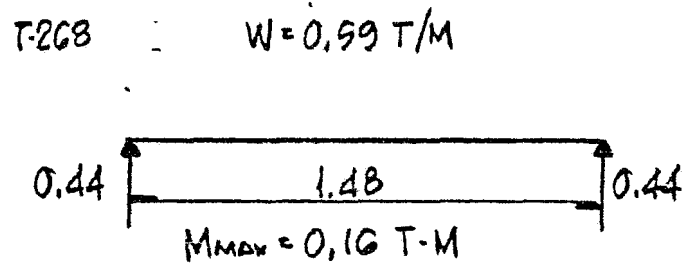
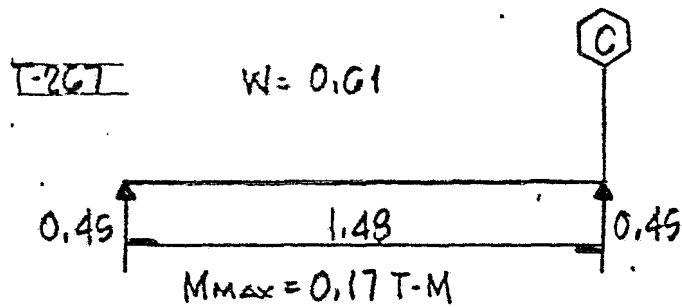
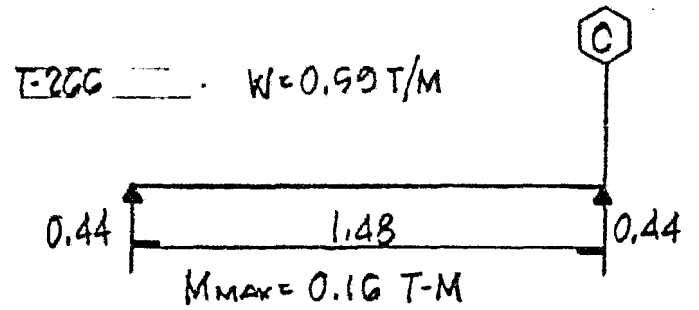
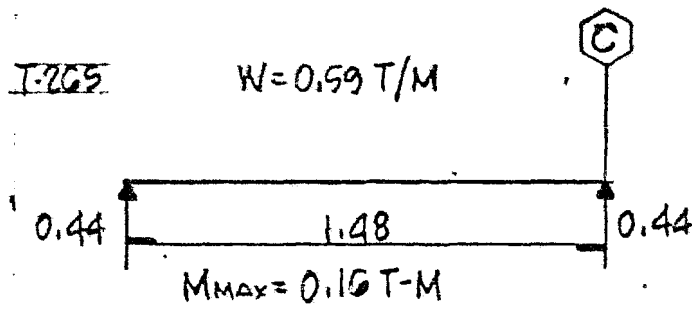
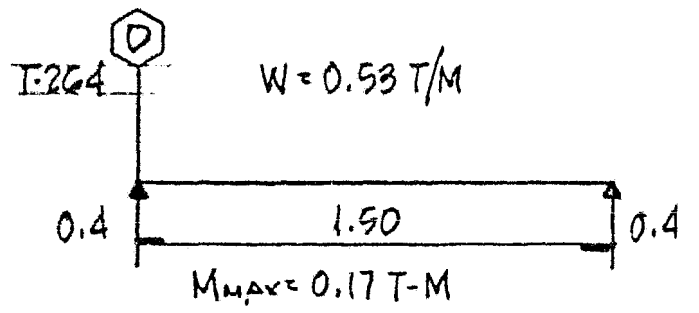
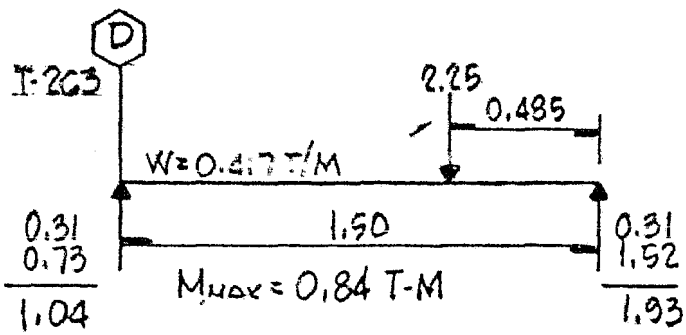


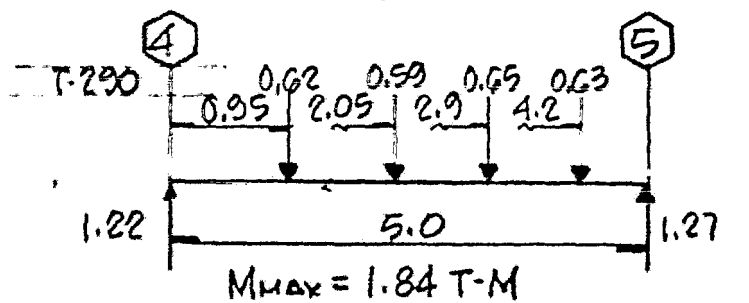
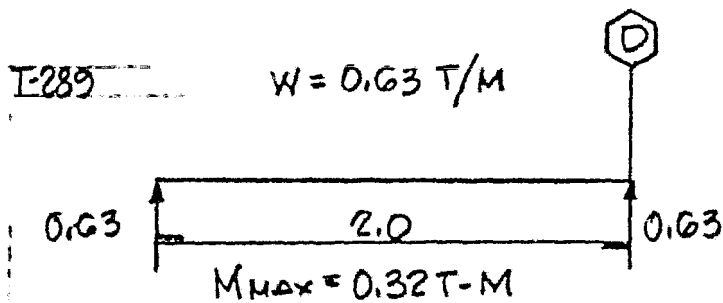
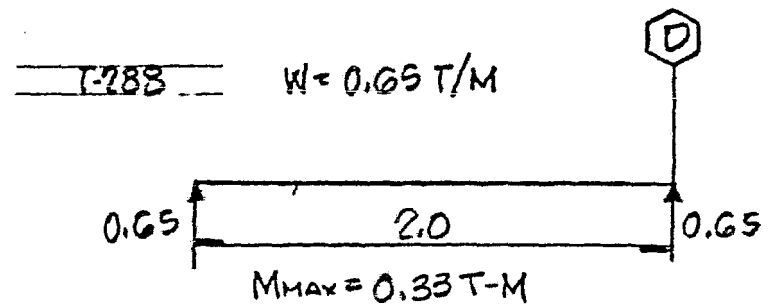
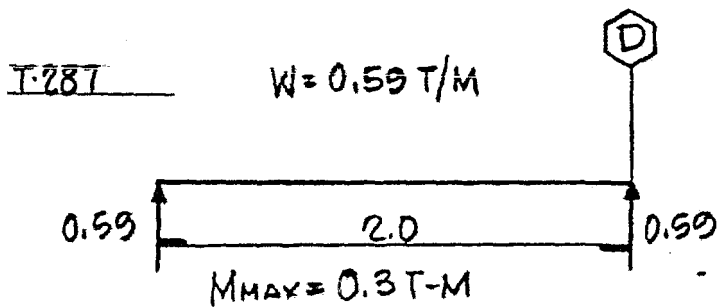
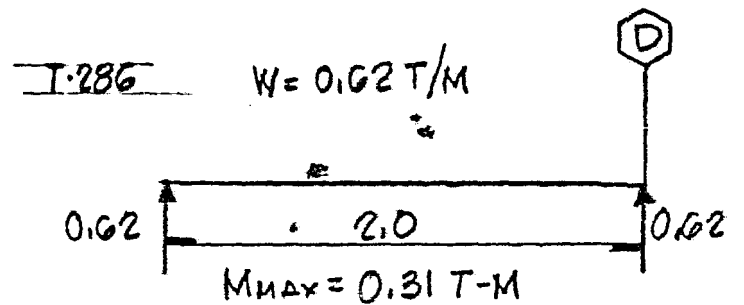
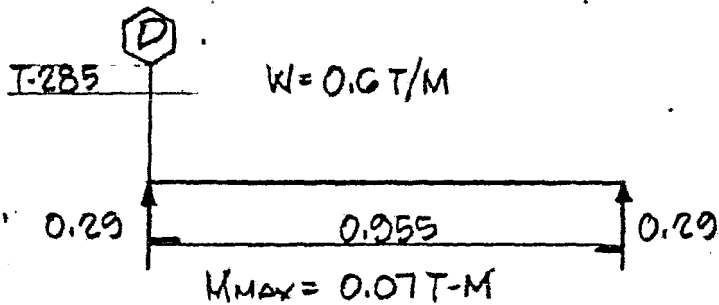
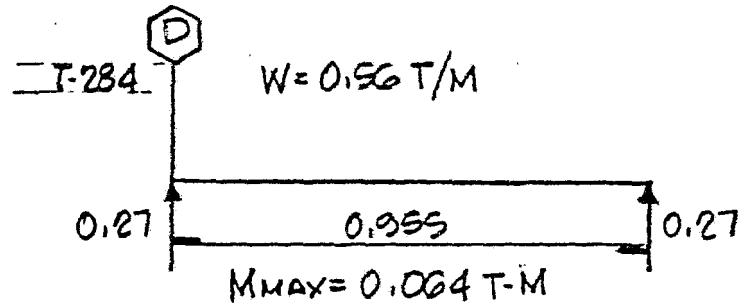
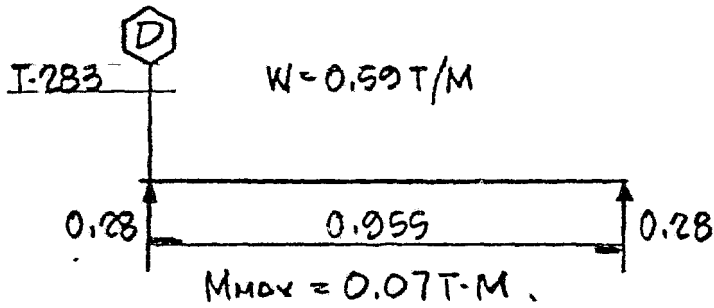
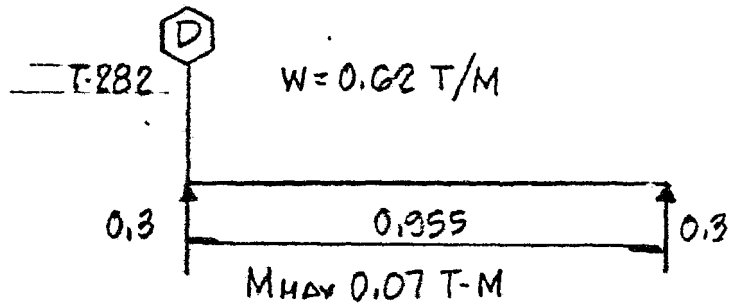
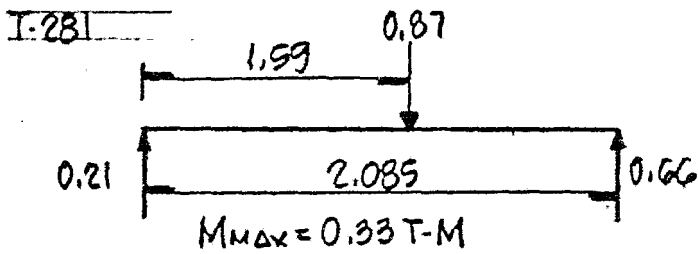
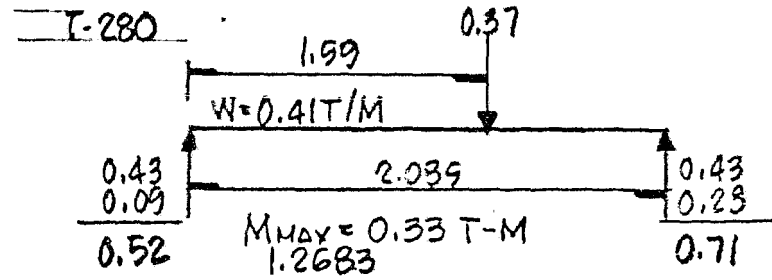
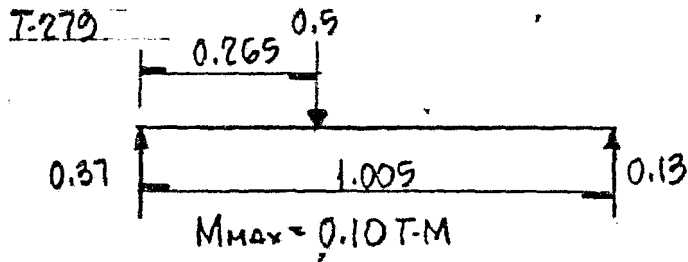
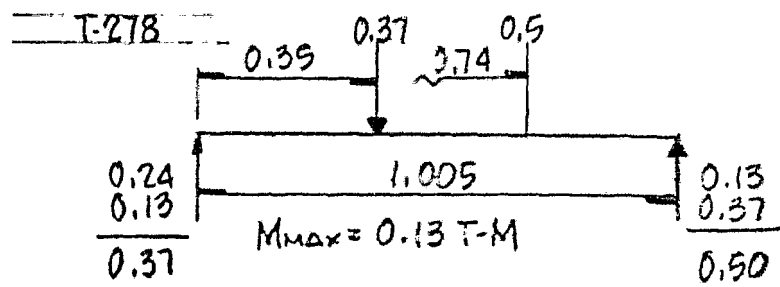
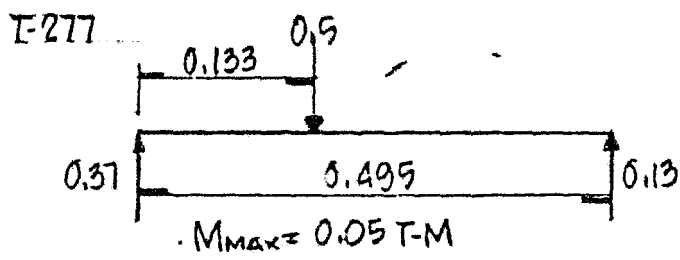
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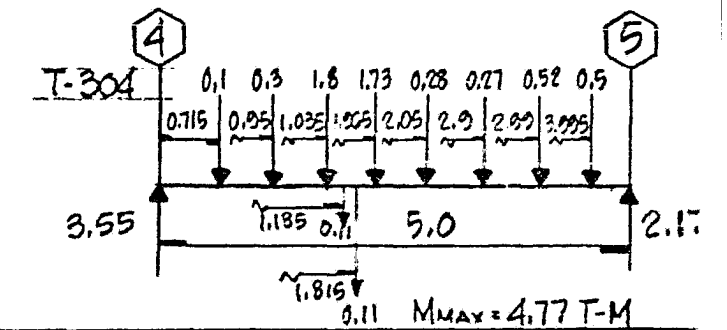
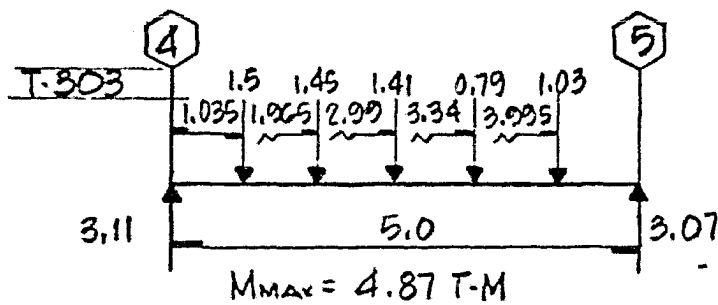
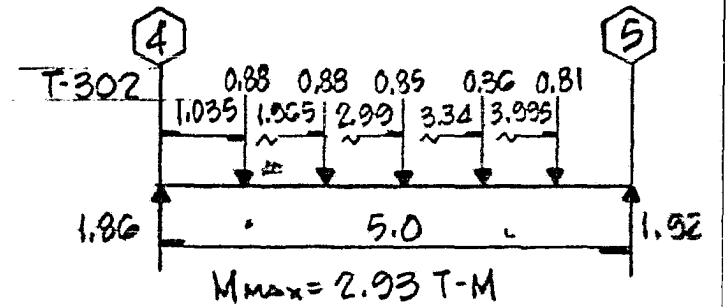
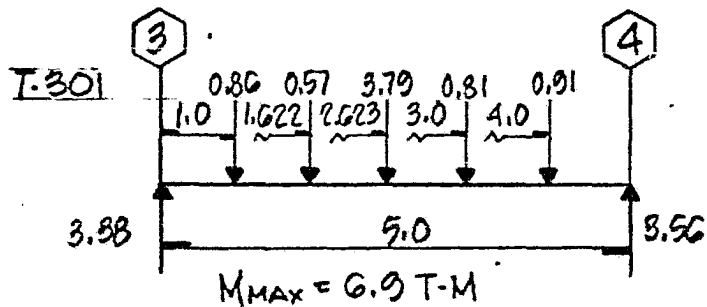
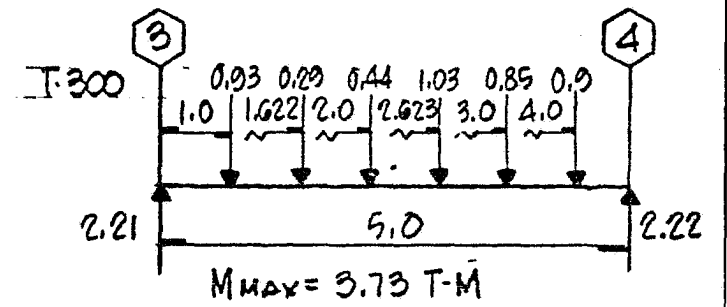
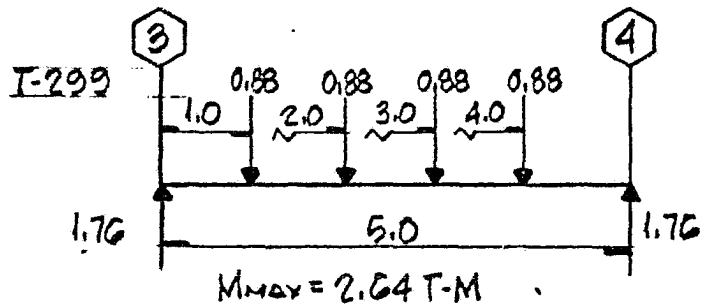
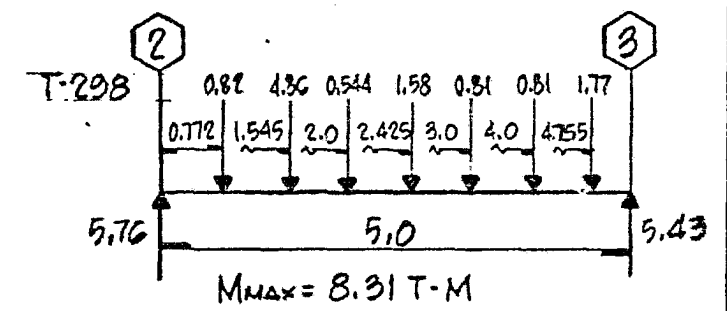
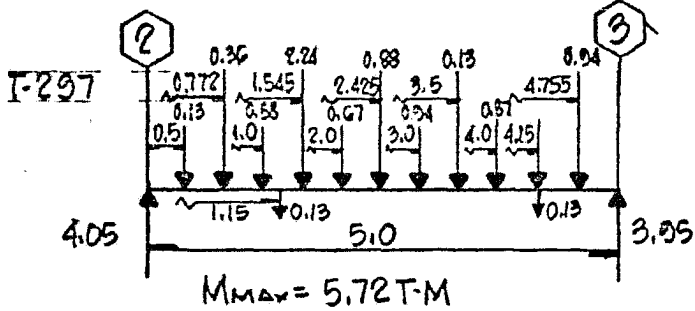
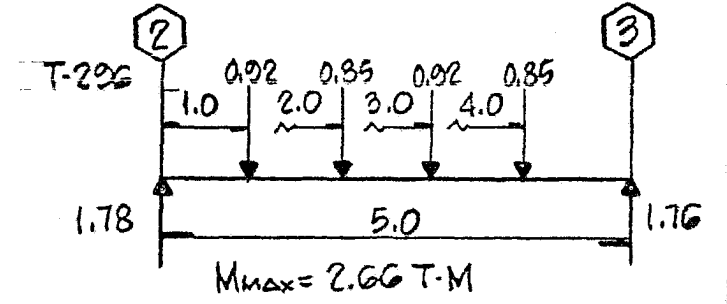
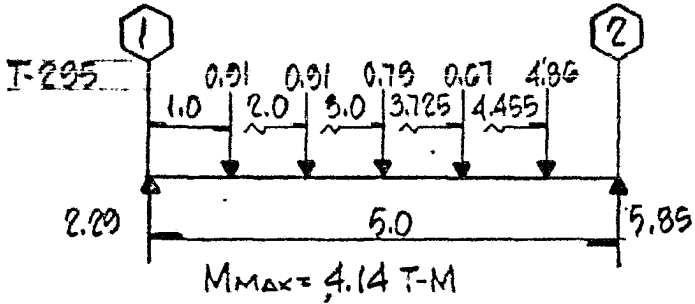
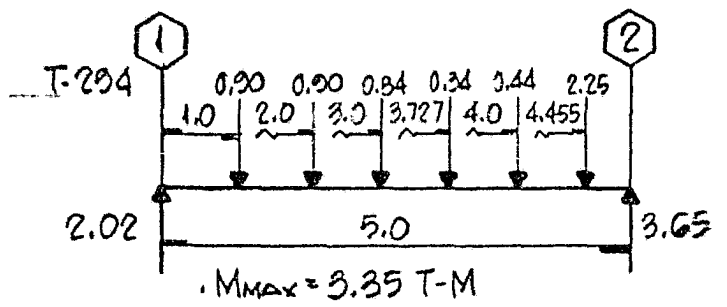
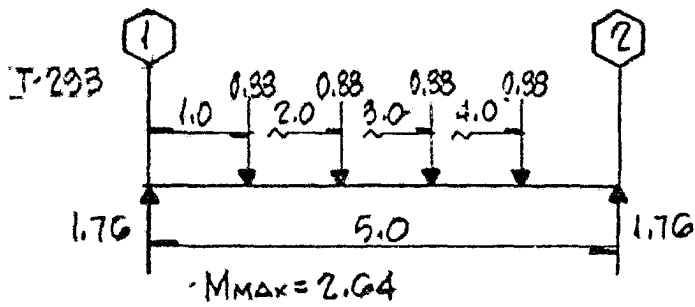




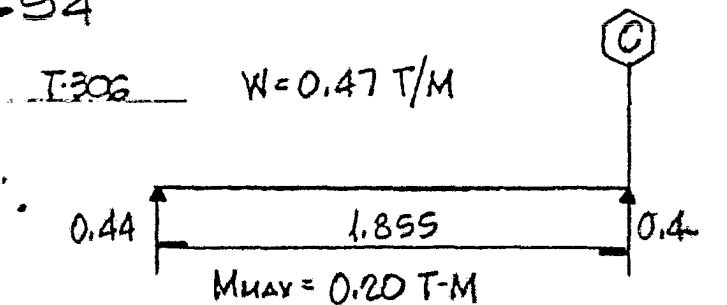
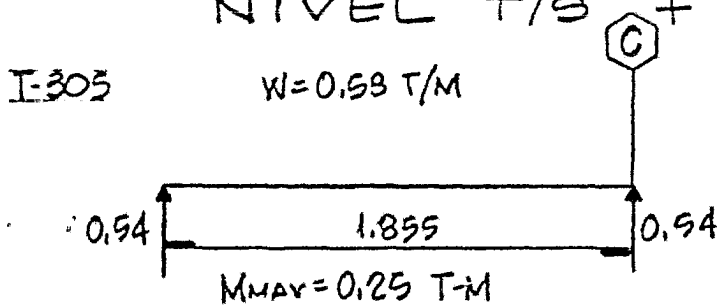


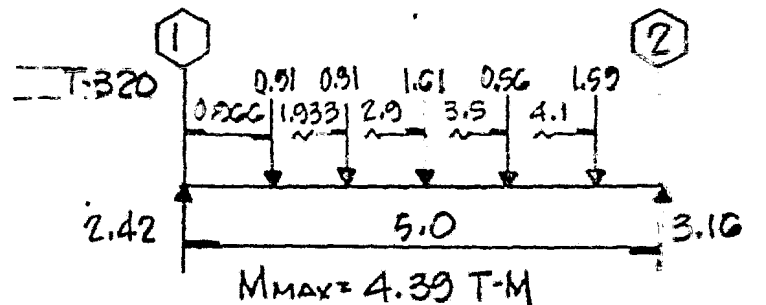
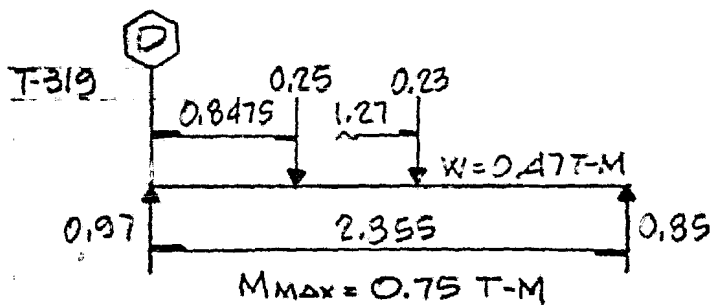
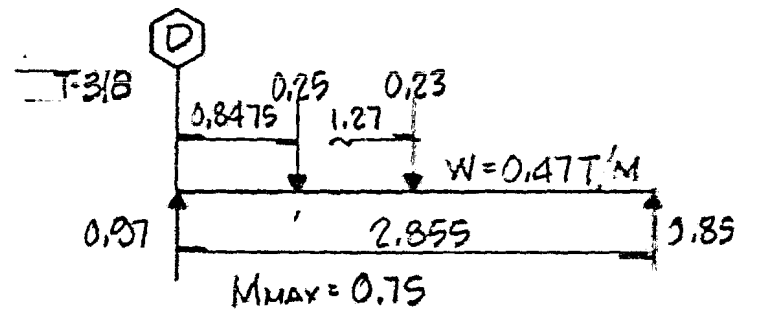
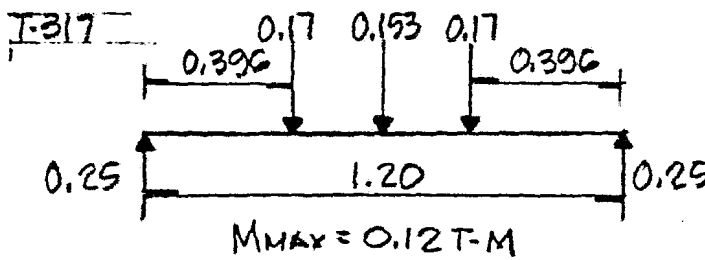
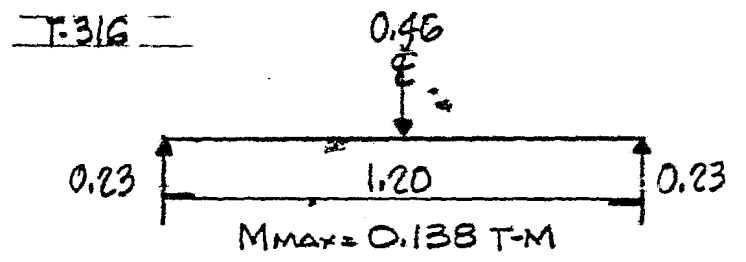
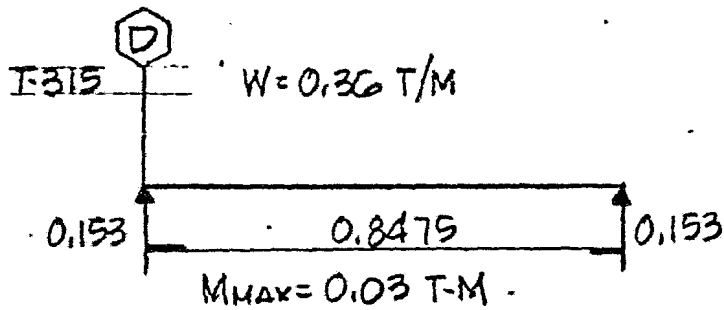
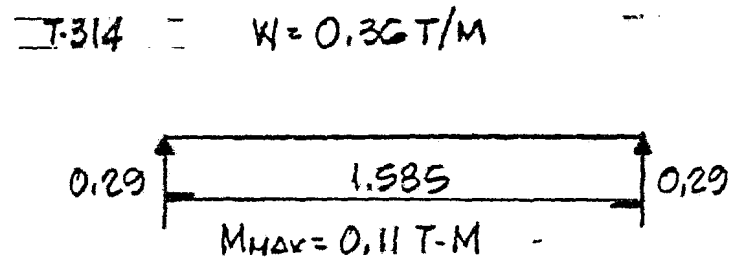
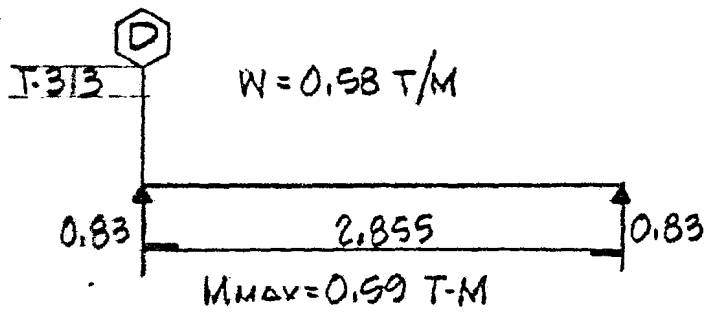
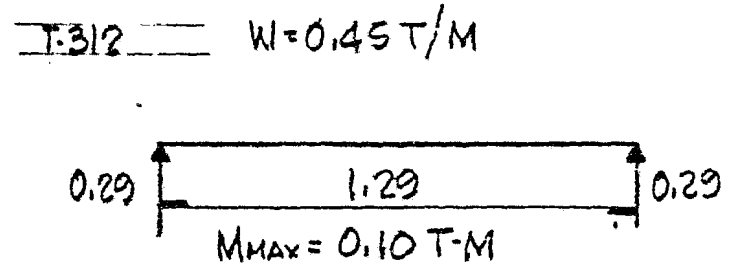
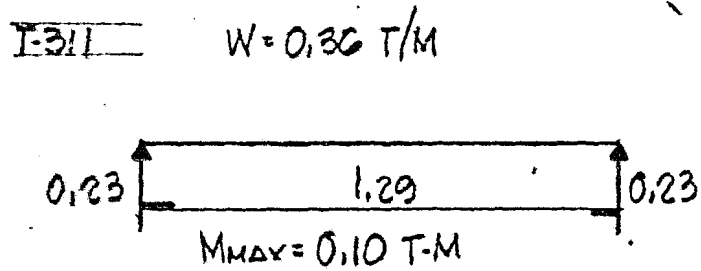
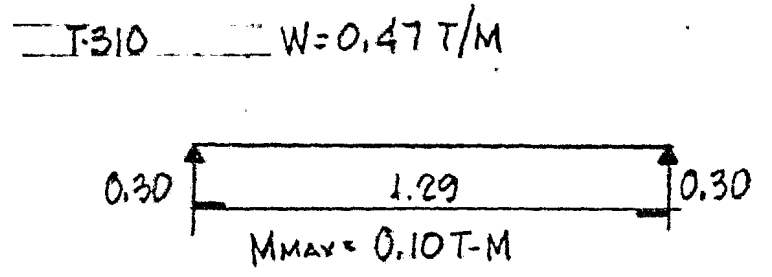
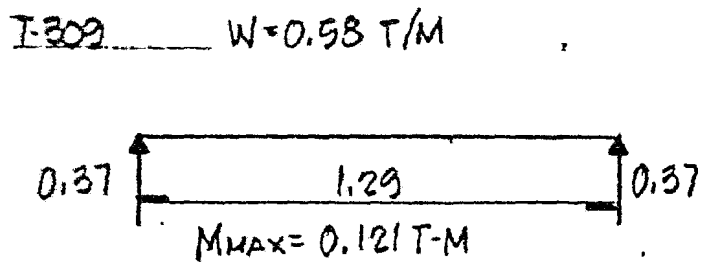
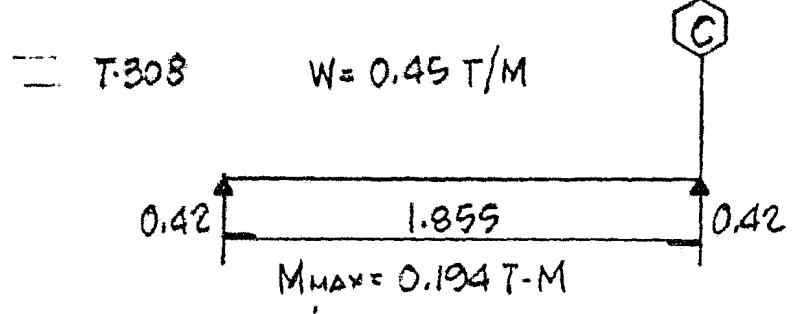
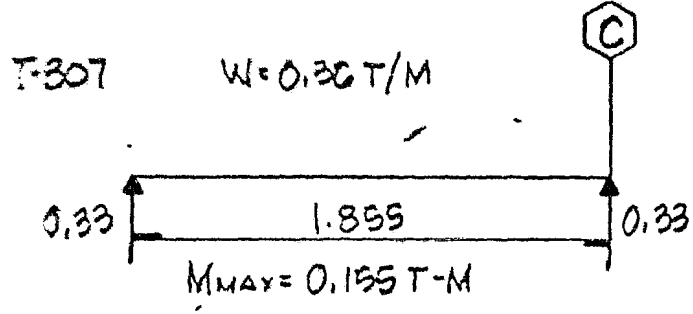


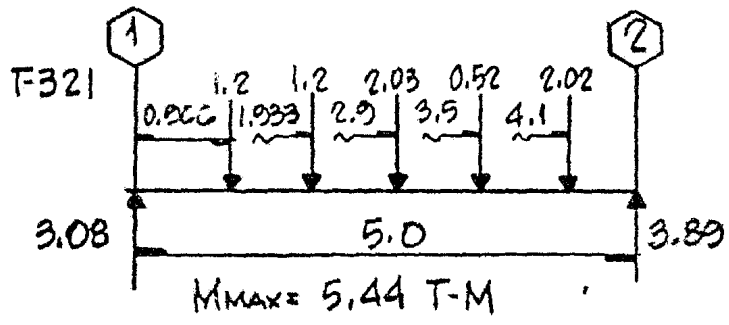


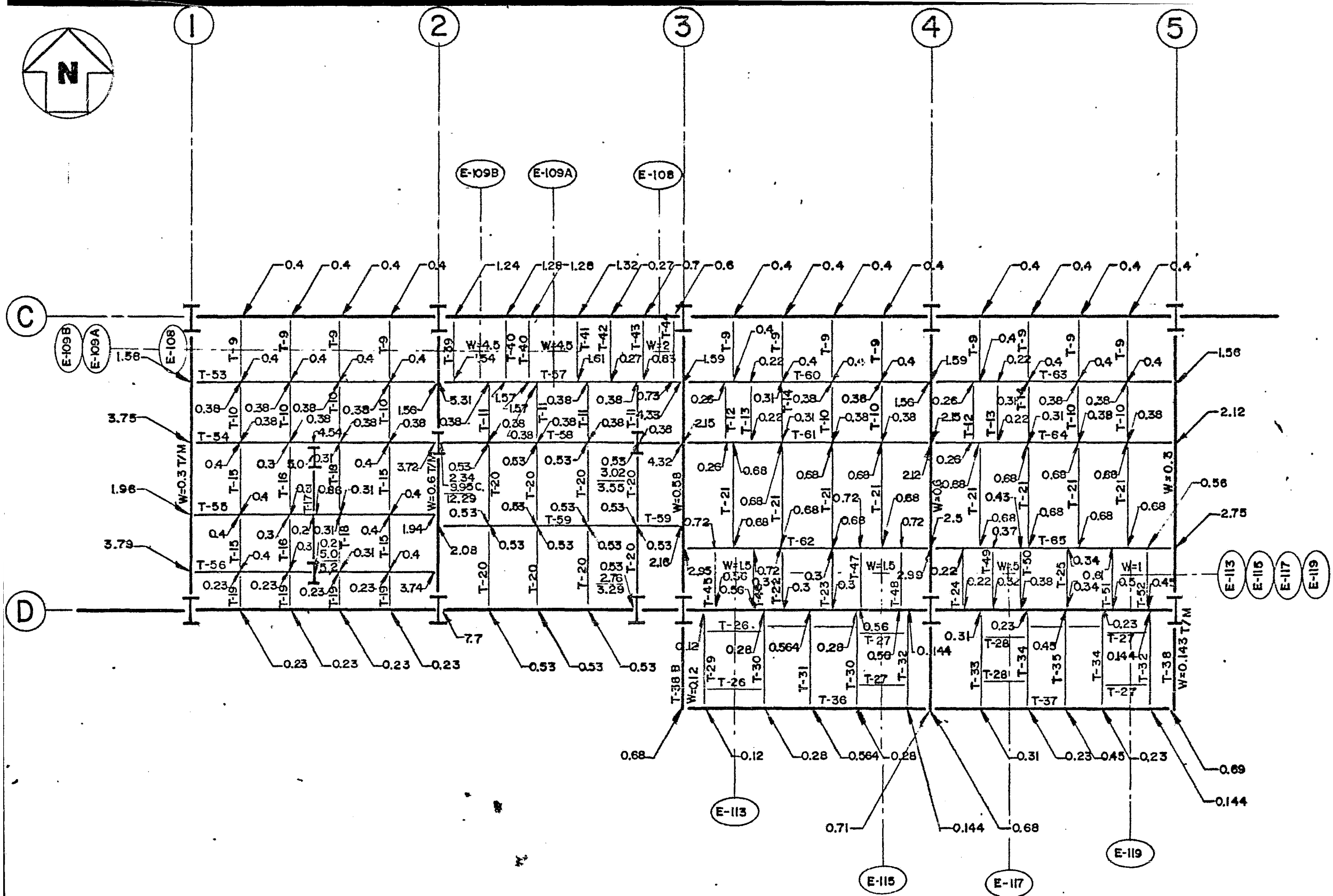
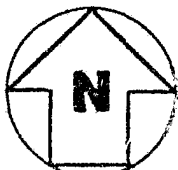


NIVEL T/S + 25.694

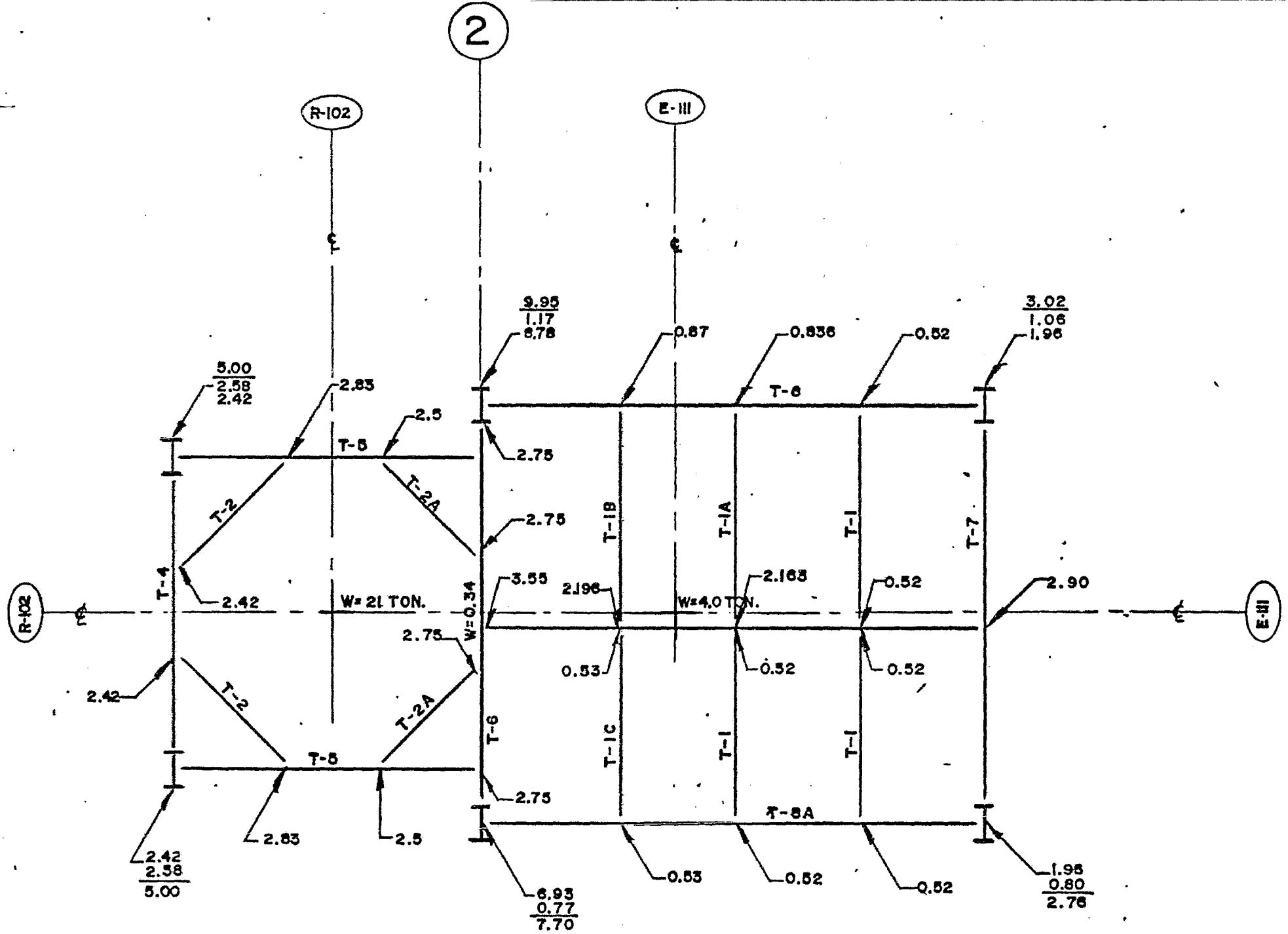
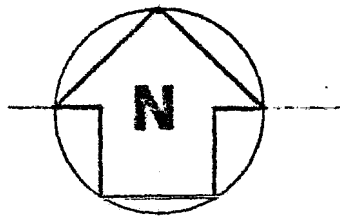




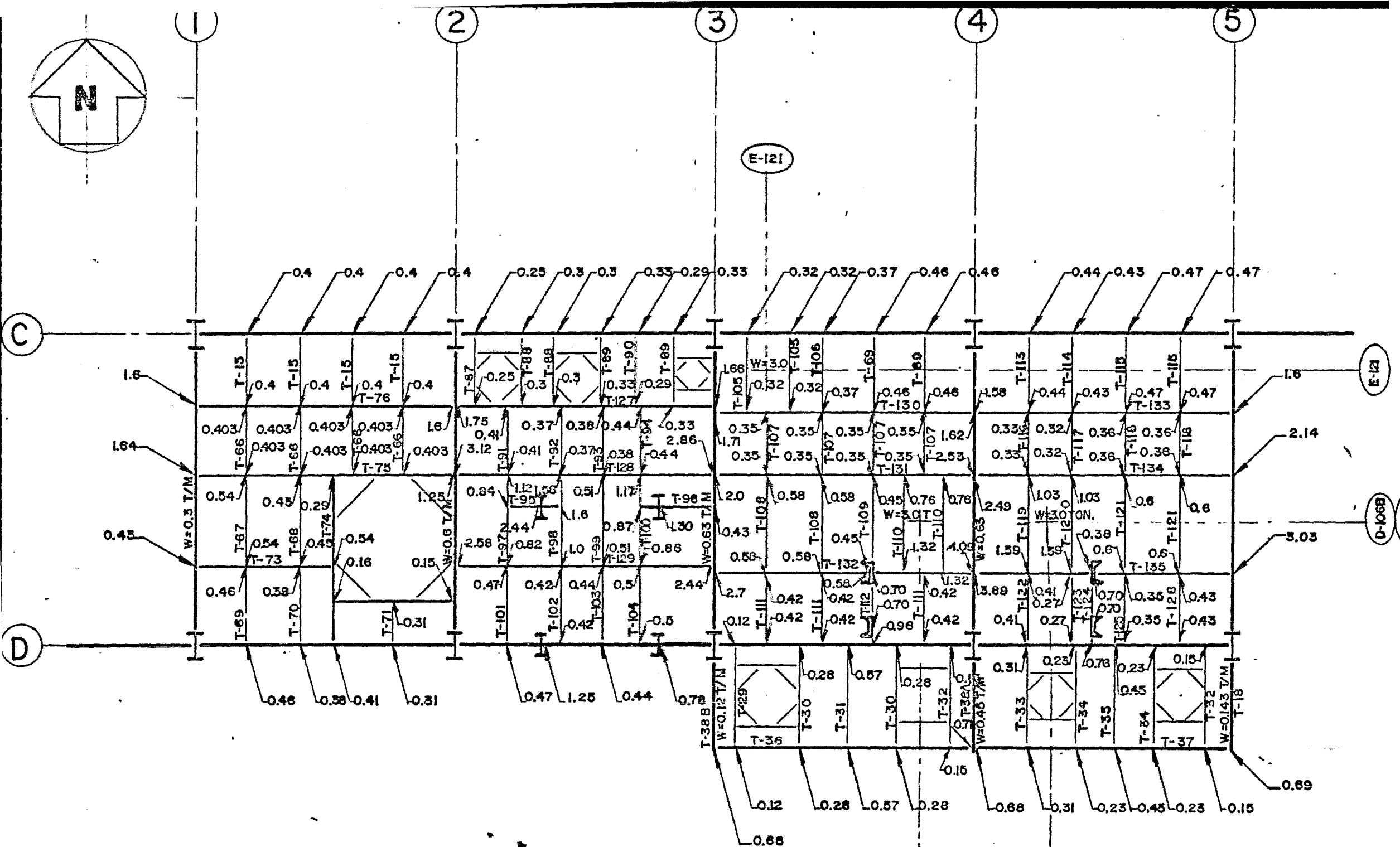
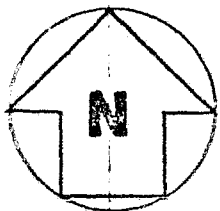




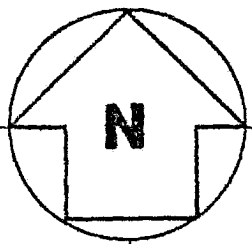
PLANTA NIVEL 2 T/S+9.694 Y T/S+9.194  
BAJADA DE CARGAS



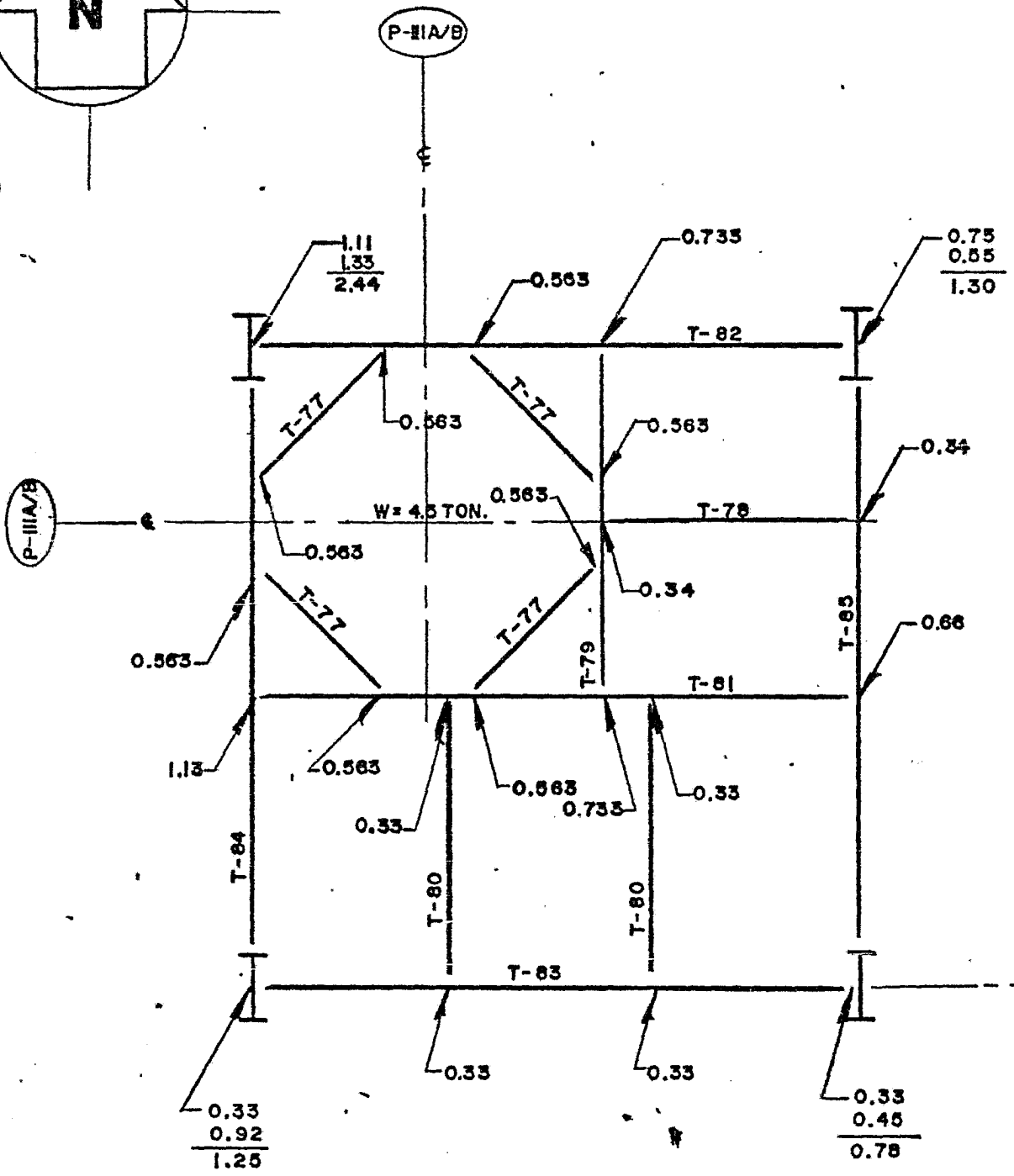
PLANTAS T/S + 11.864 Y T/S + 11.444  
BAJADA DE CARGAS



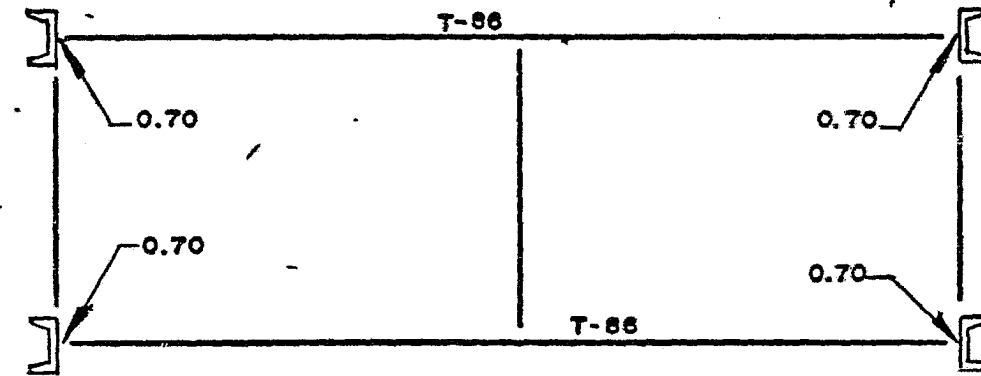
PLANTA NIVEL 3 T/S +13.694  
 BAJADA DE CARGAS



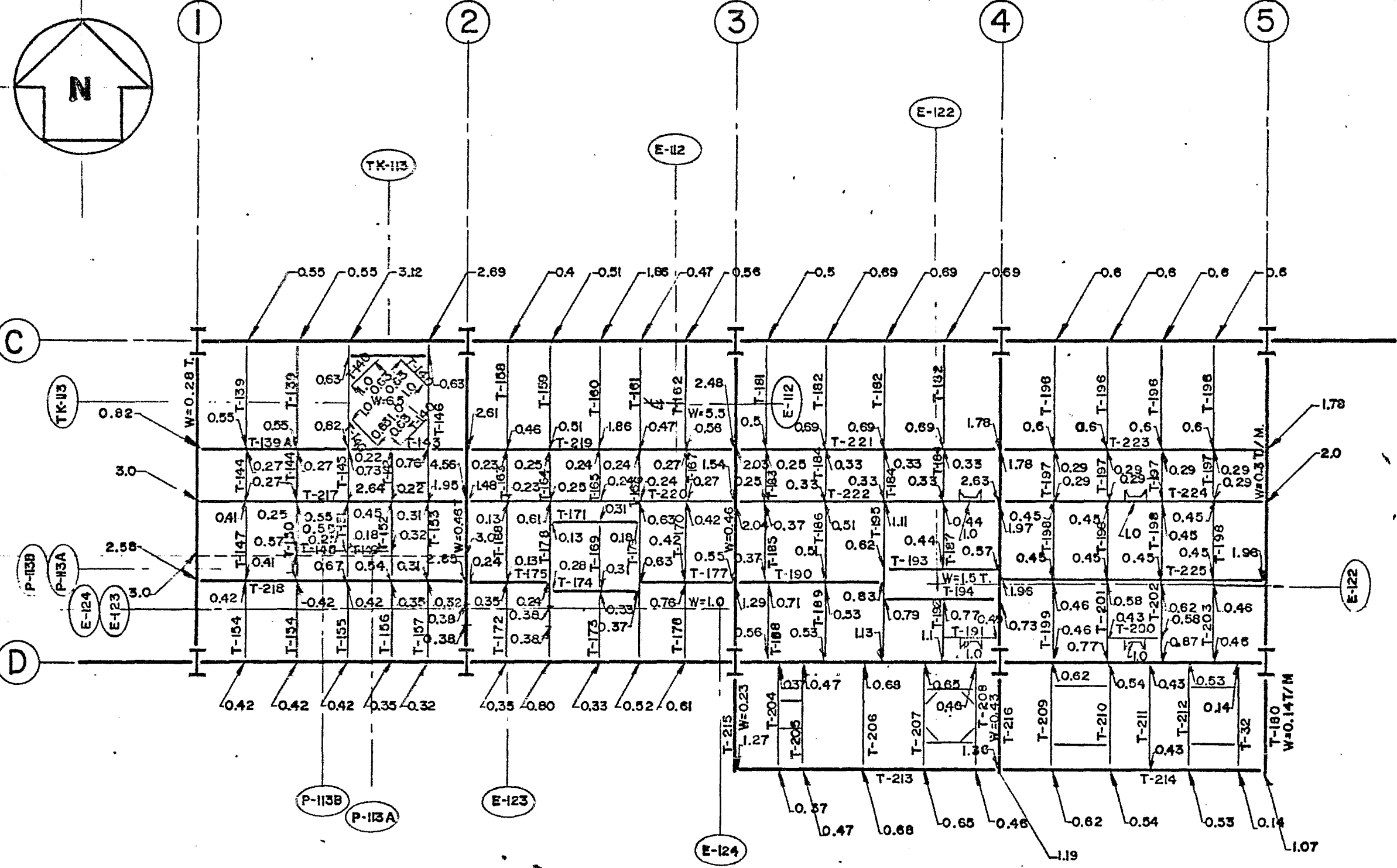
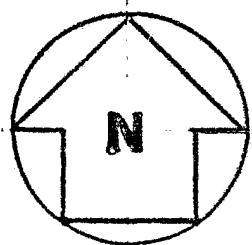
2



PLANTA T/S + 14.914  
BAJADA DE CARGAS

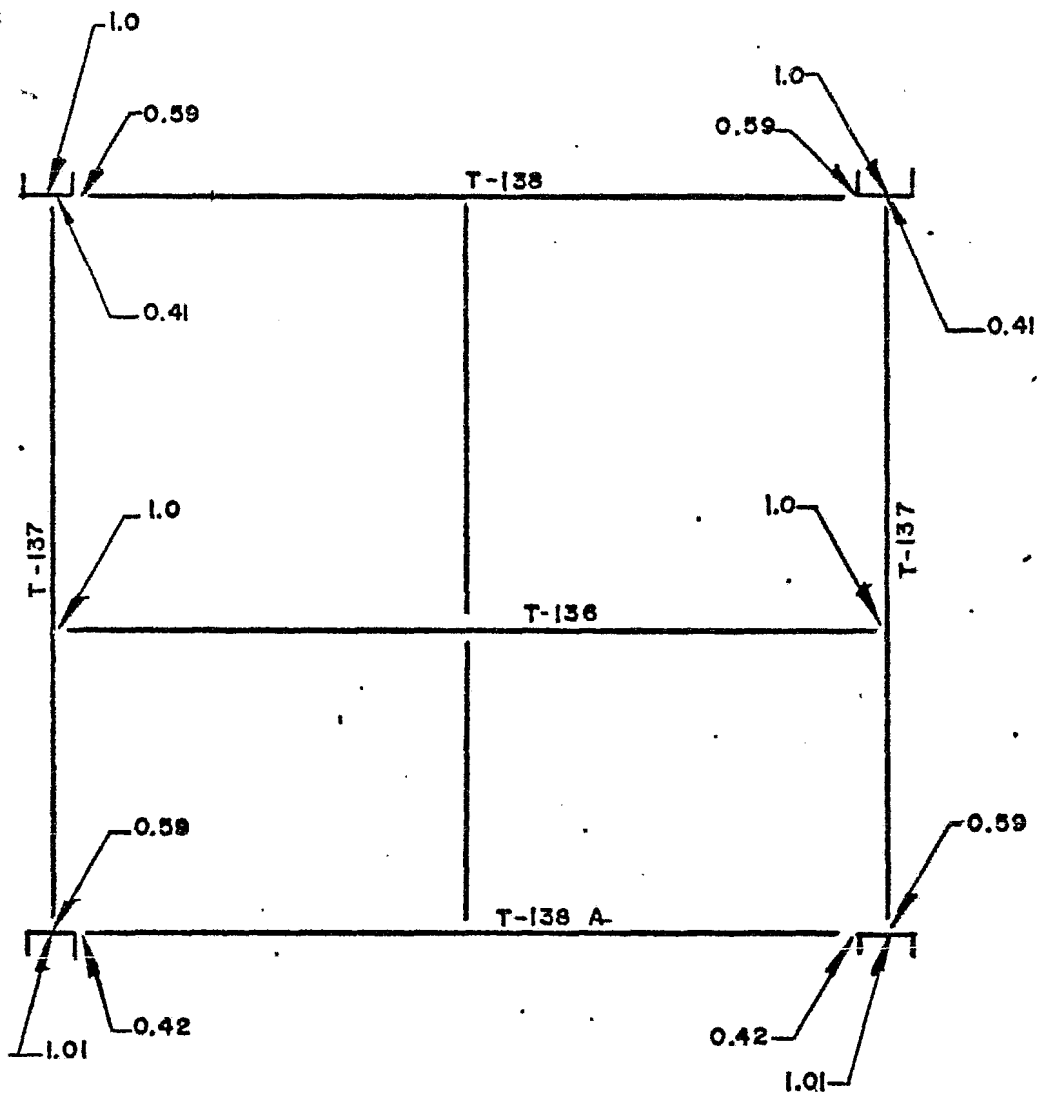
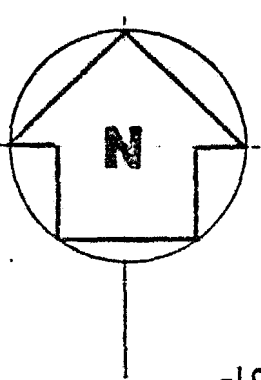


PLANTA T/S + 15.194  
BAJADA DE CARGAS

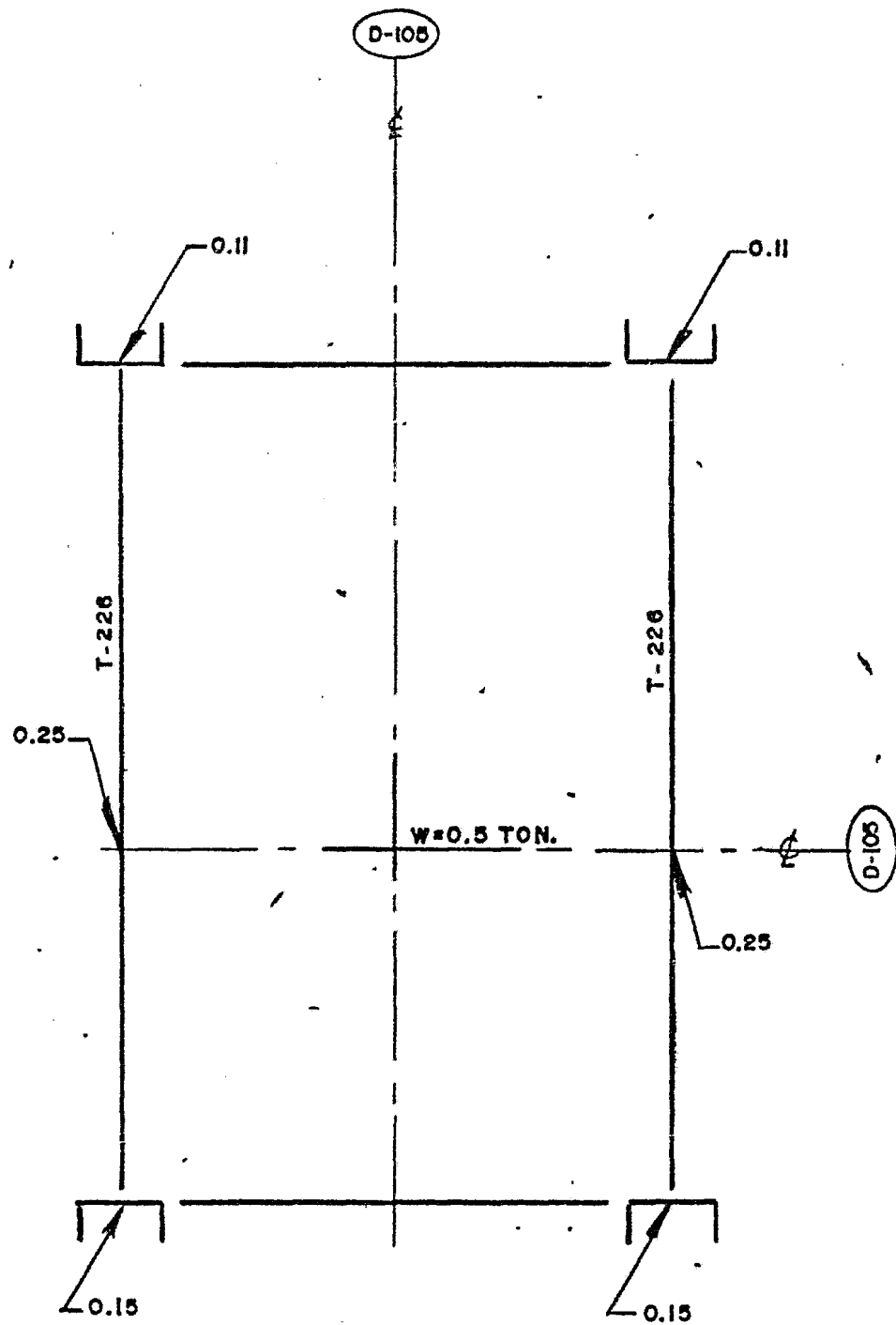


PLANTA NIVEL +4 T/S+17.694  
BAJADA DE CARGAS

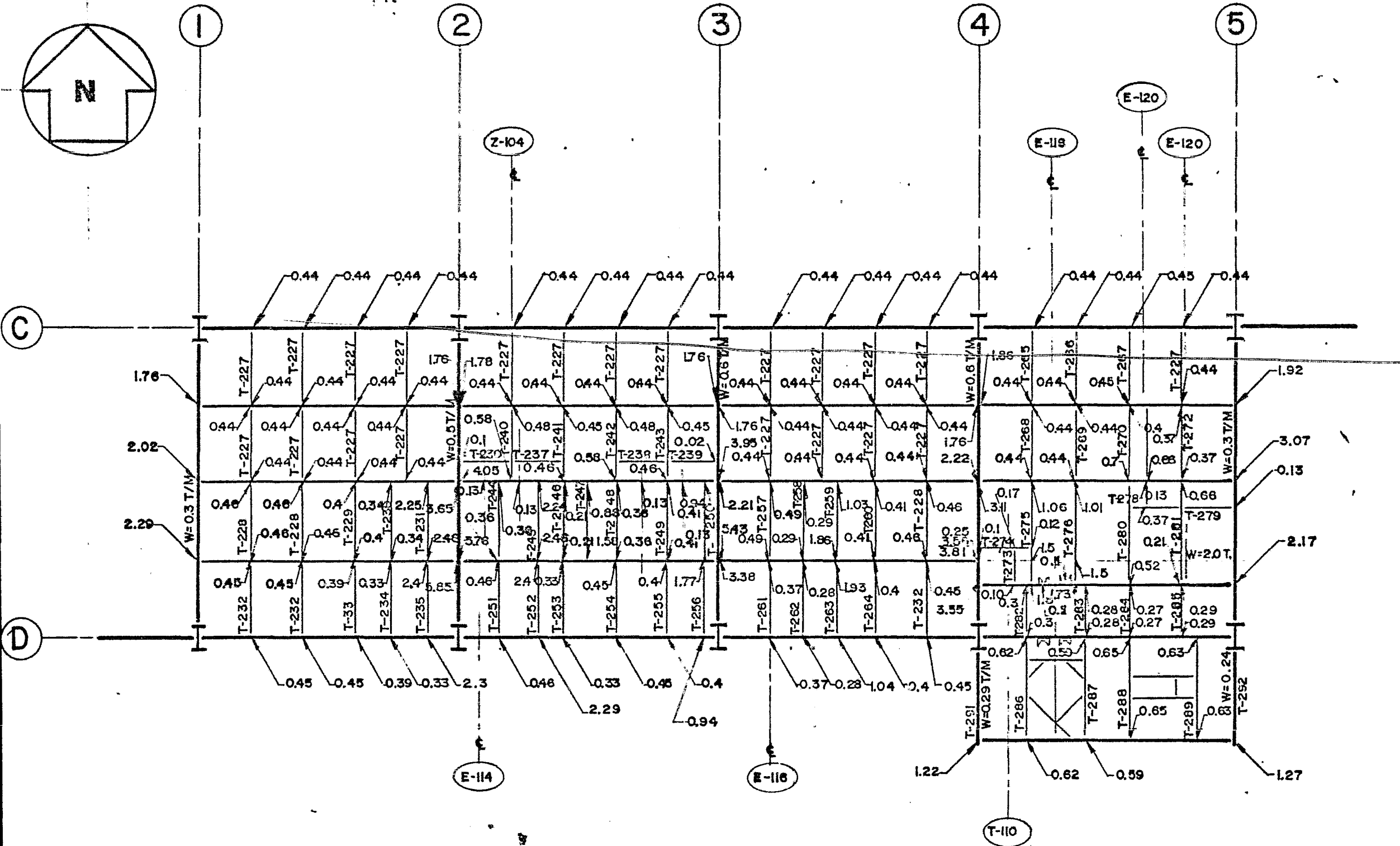
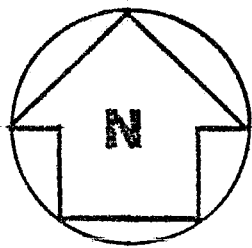




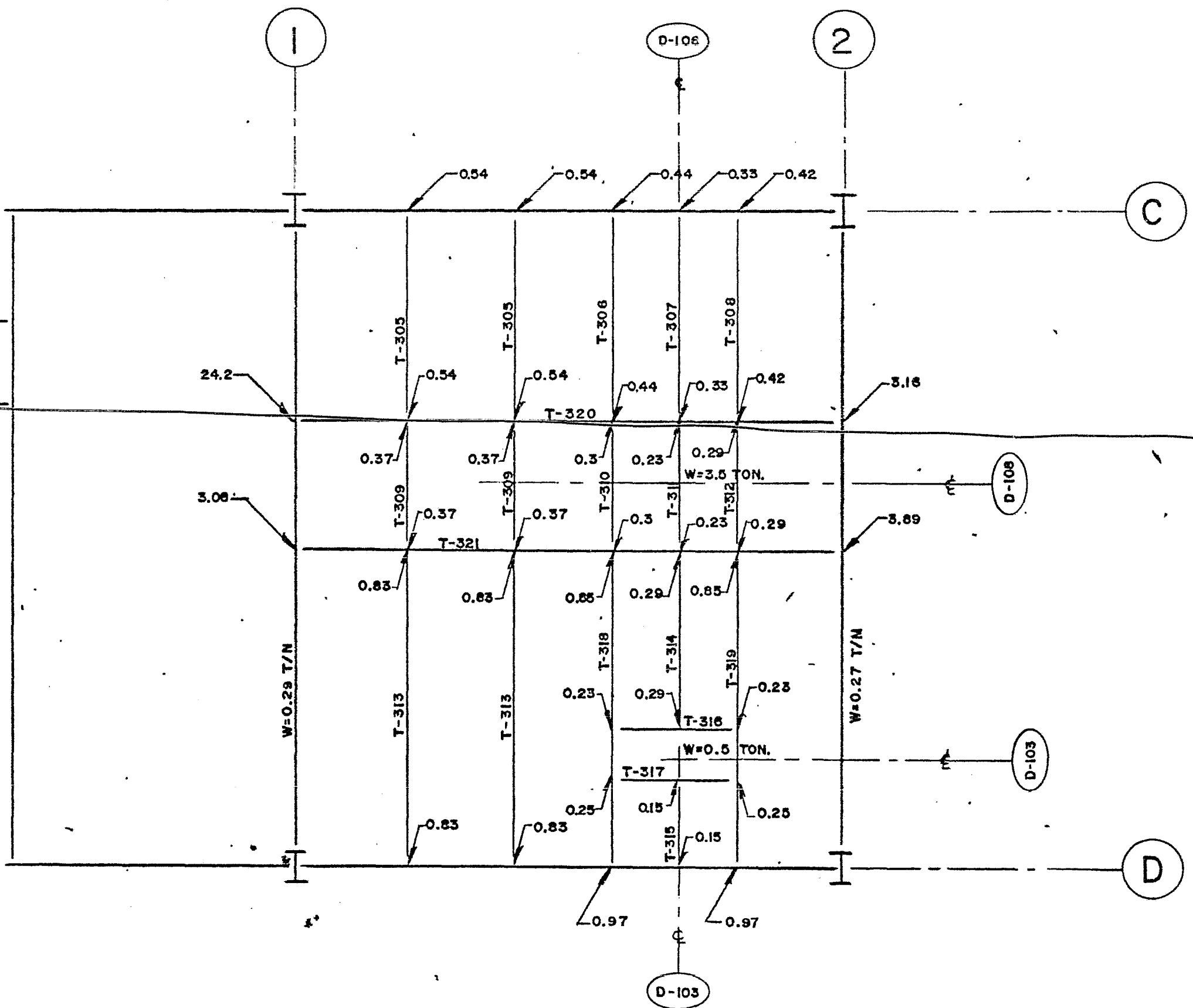
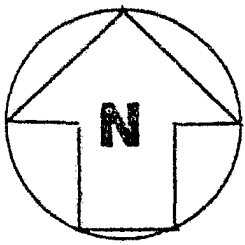
PLANTA T/S + 18.894  
BAJADA DE CARGAS



PLANTA T/S + 20.884  
BAJADA DE CARGAS



PLANTA NIVEL +5 T/S +21.694  
BAJADA DE CARGAS

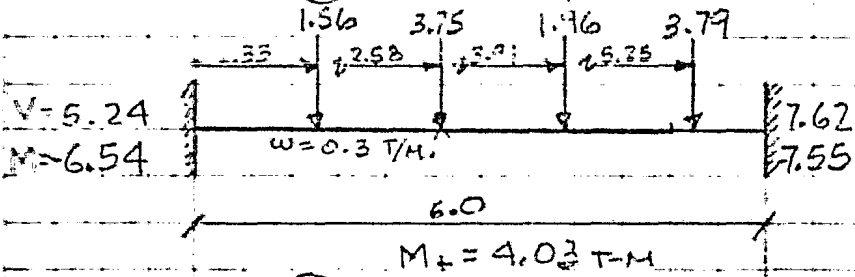


PLANTA + 6-T/S 25.694

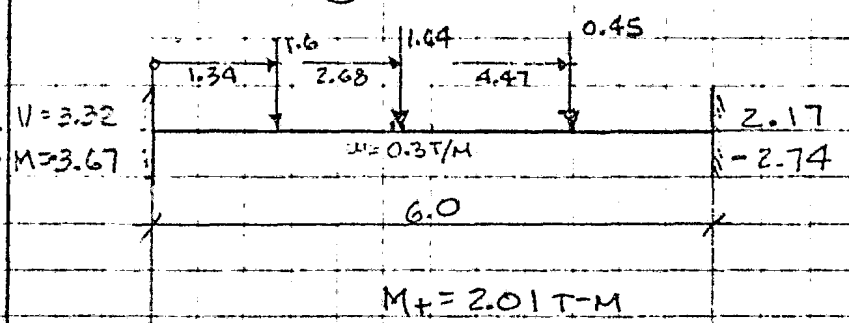
BAJADA DE CARGAS

ANALISIS PRELIMINAR DE TRABES PRINCIPALES TRANSVERSALES.

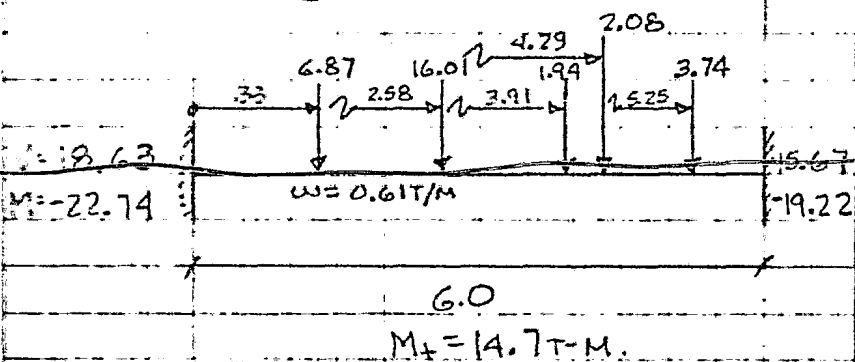
TRABE EJE ① NIVEL T/S+2



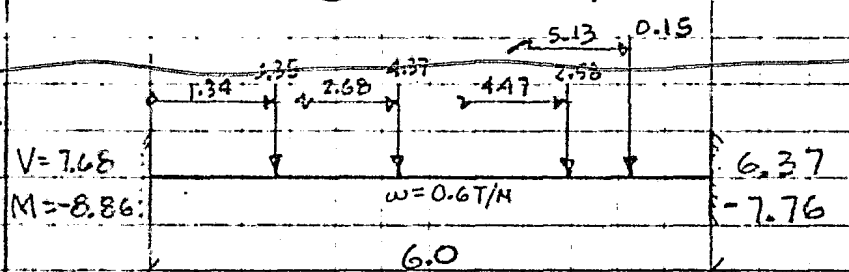
TRABE EJE ① NIVEL T/S+3



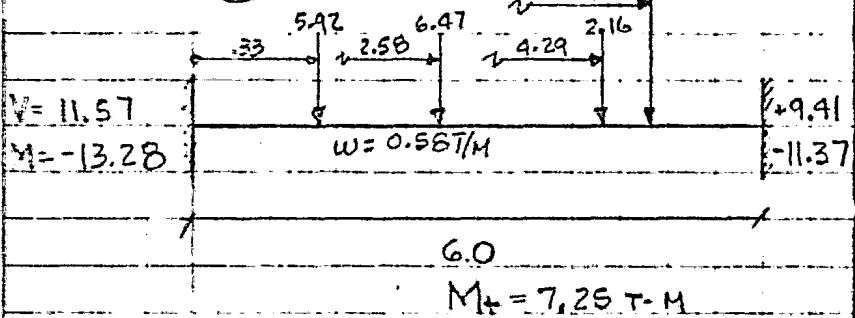
TRABE EJE ② NIVEL T/S+2



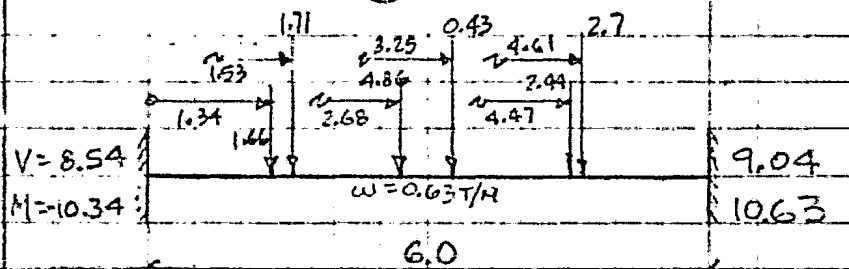
TRABE EJE ② NIVEL T/S+3



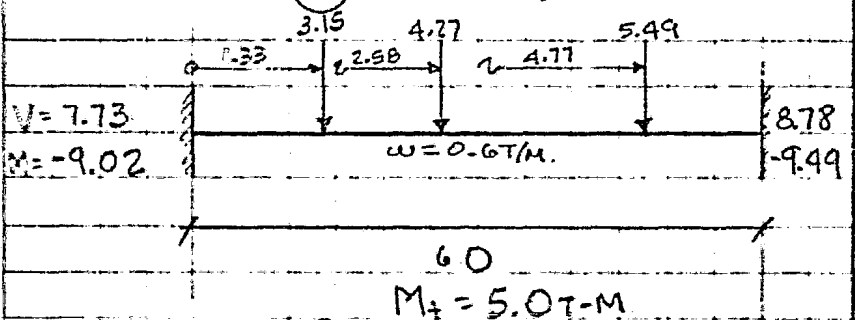
TRABE EJE ③ NIVEL T/S+2



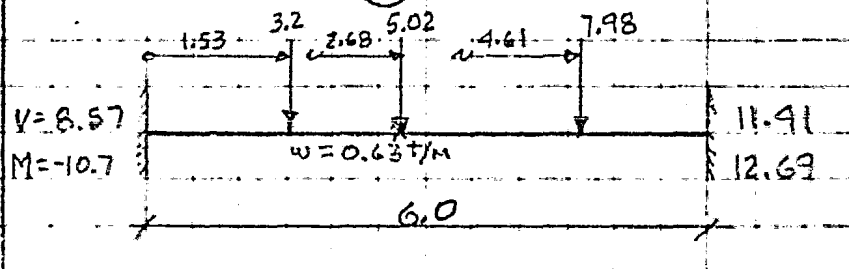
TRABE EJE ③ NIVEL T/S+3



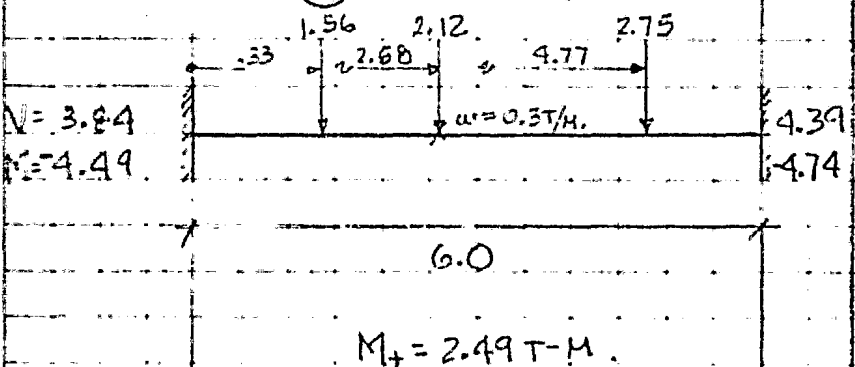
TRABE EJE ④ NIVEL T/S+2



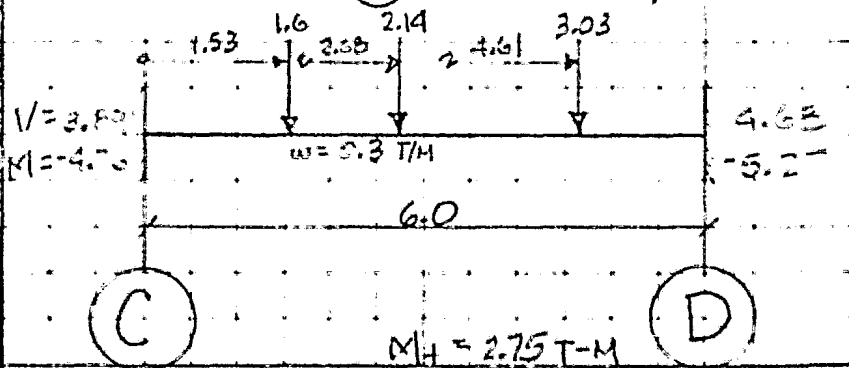
TRABE EJE ④ NIVEL T/S+3



TRABE EJE ⑤ NIVEL T/S+2



TRABE EJE ⑤ NIVEL T/S+3



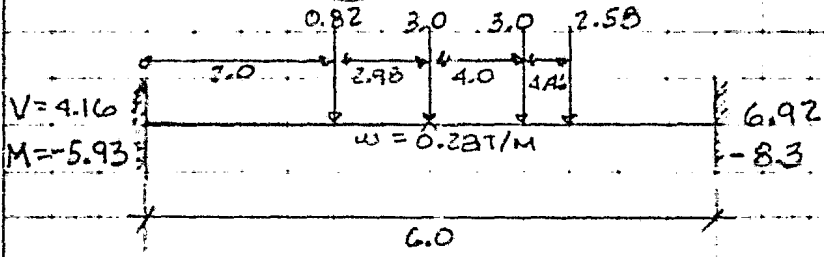
①

②

③

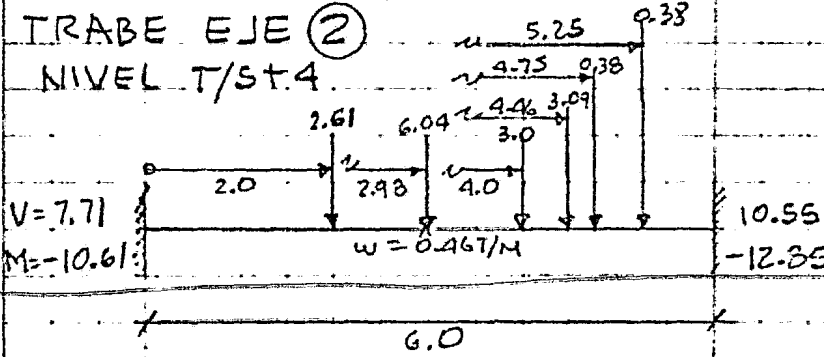
④

TRABE EJE ① NIVEL T/S+4



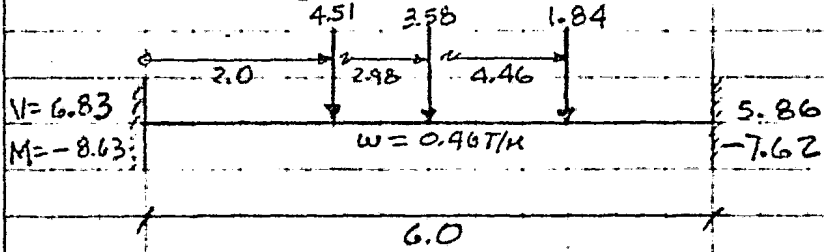
$M_+ = 4.43$

TRABE EJE ② NIVEL T/S+4



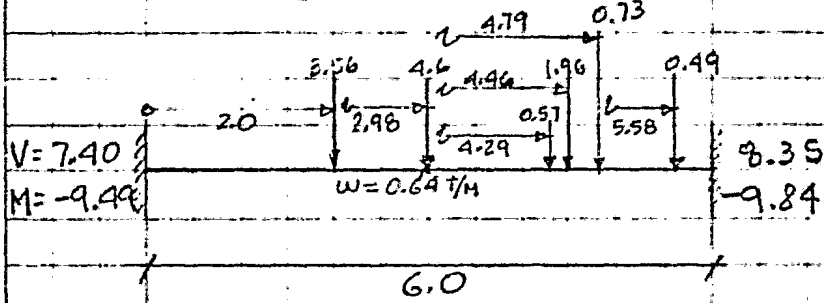
$M_+ = 7.75$

TRABE EJE ③ NIVEL T/S+4



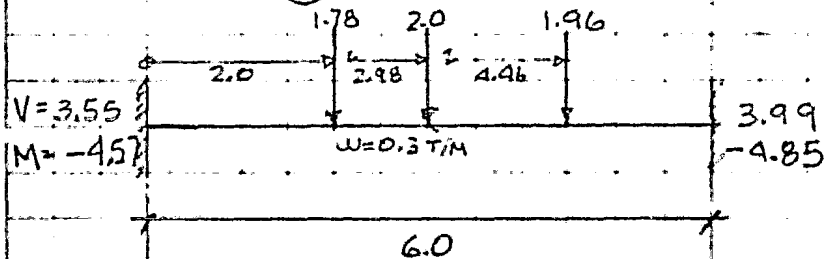
$M_+ = 5.26 T-M$

TRABE EJE ④ NIVEL T/S+4



$M_+ = 6.23 T-M$

TRABE EJE ⑤ NIVEL T/S+4

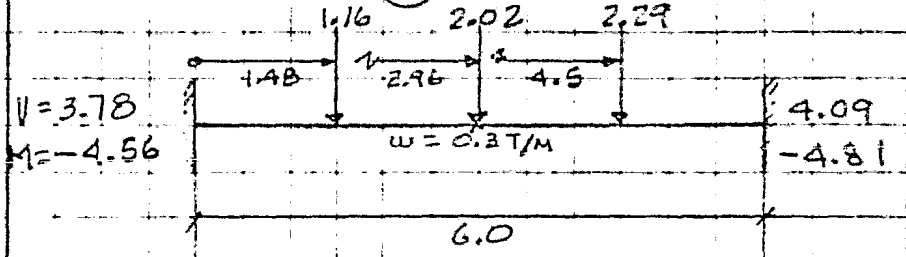


$M_+ = 2.93 T-M$

C

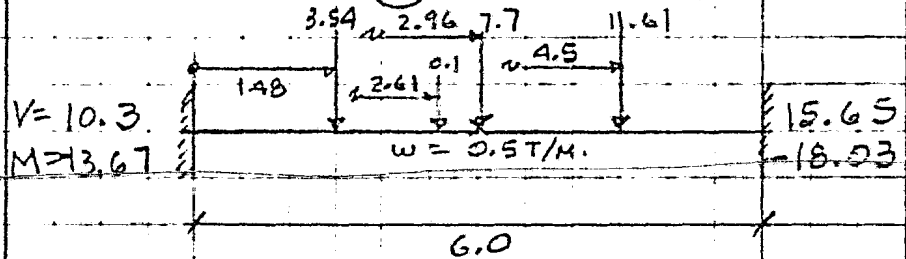
D

TRABE EJE ① NIVEL T/S+5



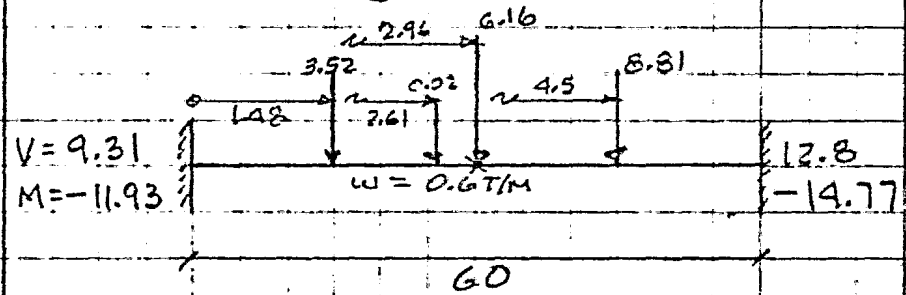
$M_+ = 2.71$

TRABE EJE ② NIVEL T/S+5



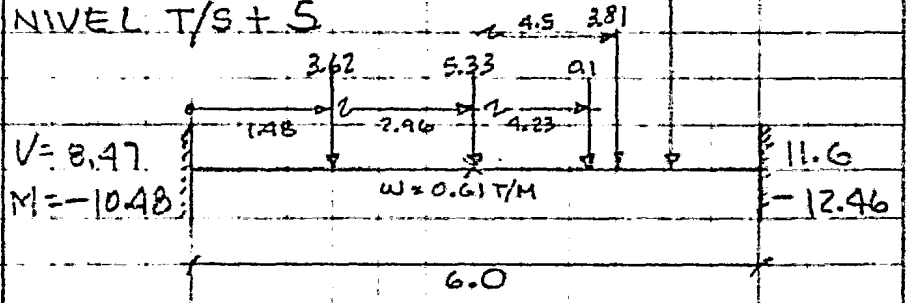
$M_+ = 9.35 T-M$

TRABE EJE ③ NIVEL T/S+5



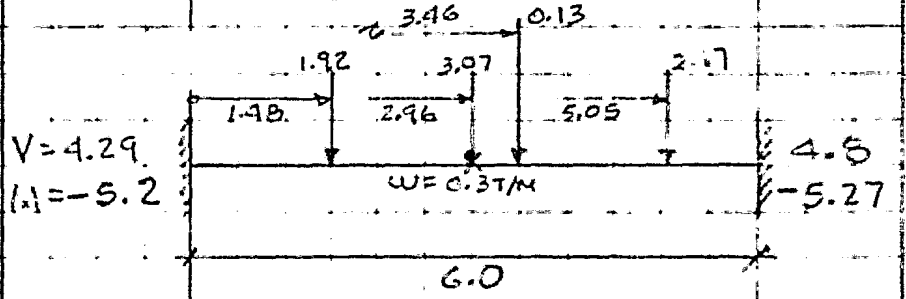
$M_+ = 7.79 T-M$

TRABE EJE ④ NIVEL T/S+5



$M_+ = 6.57 T-M$

TRABE EJE ⑤ NIVEL T/S+5

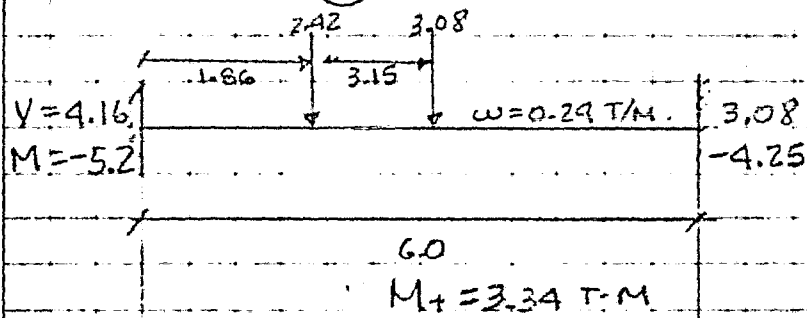


$M_+ = 3.34 T-M$

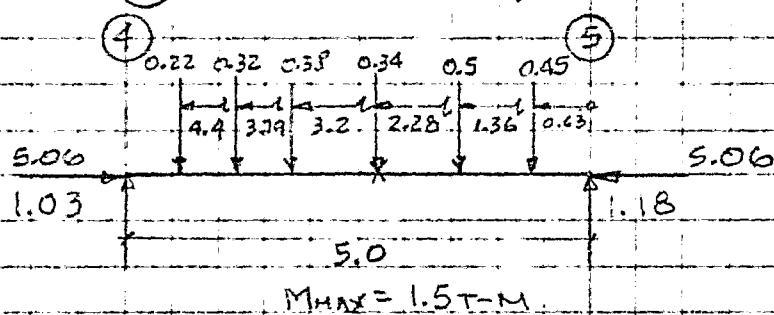
C

D

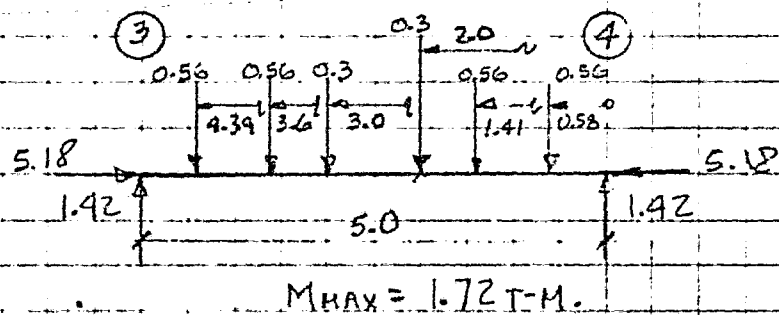
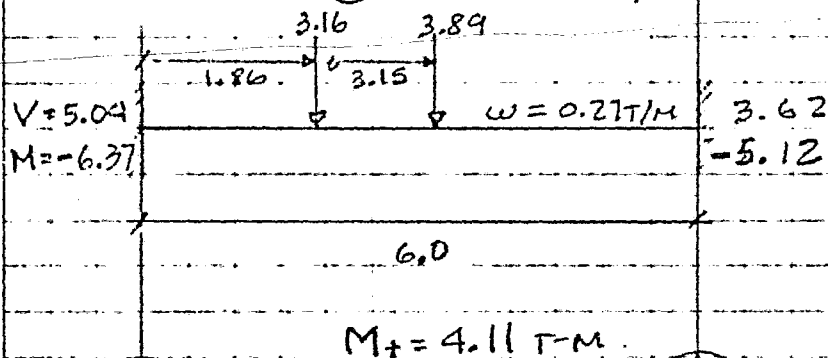
TRABE EJE ① NIVEL T/S+6



TRABES LONGITUDINALES EJE ① NIVEL T/S+2

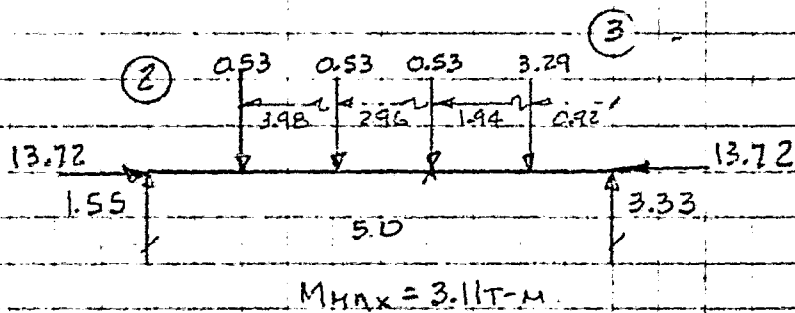
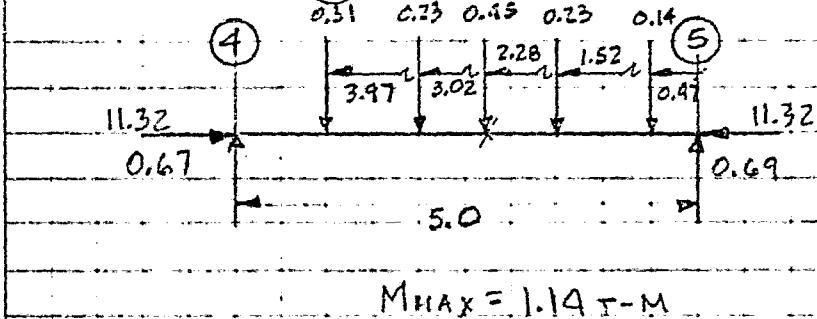


TRABE EJE ② NIVEL T/S+6

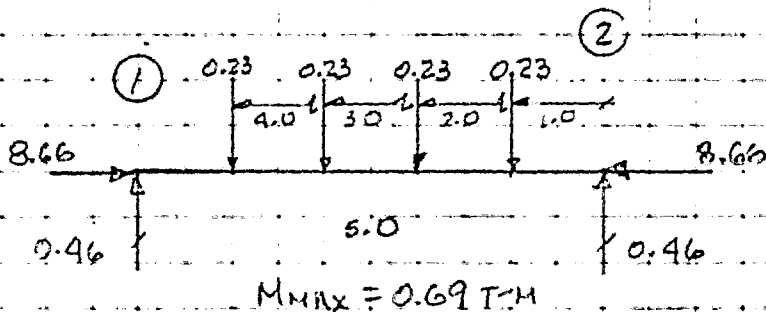
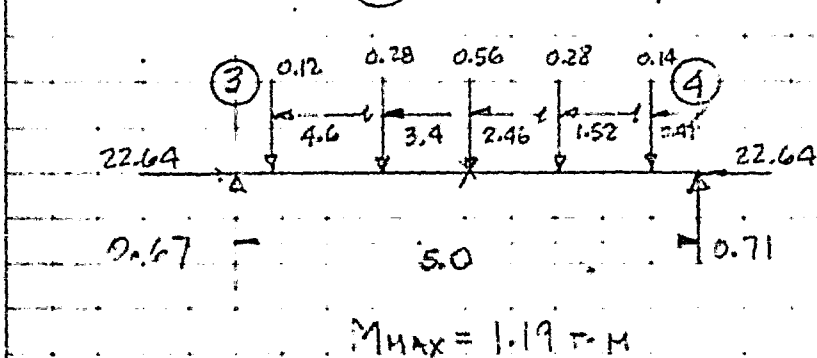


TRABES LONGITUDINALES.

TRABE EJE ① NIVEL T/S+1



TRABE EJE ② NIVEL T/S+1



PARA LAS TRABES LONGITUDINALES NO SE CONSIDERÓ EL APOYO QUE LES DA EL CONTRAVIENTO VERTICAL.

TRABES LONGITUDINALES EJE (C)

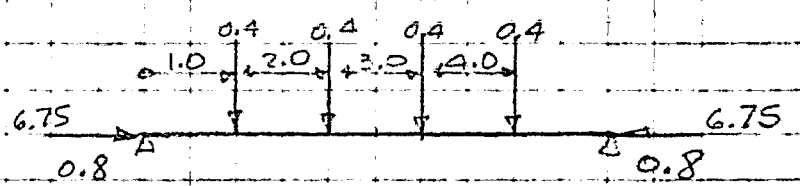
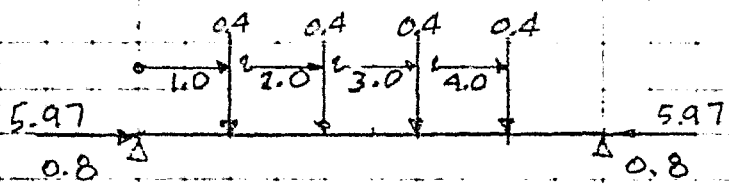
TRABES LONGITUDINALES EJE (C)

NIVEL T/S+2

NIVEL T/S+3

① ②

① ②

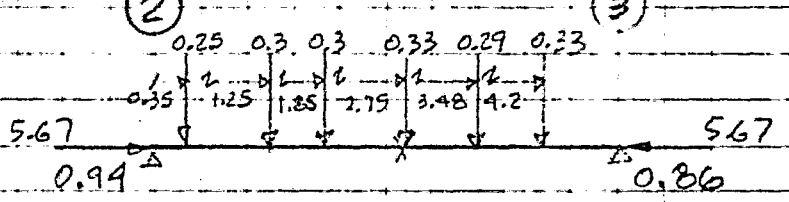
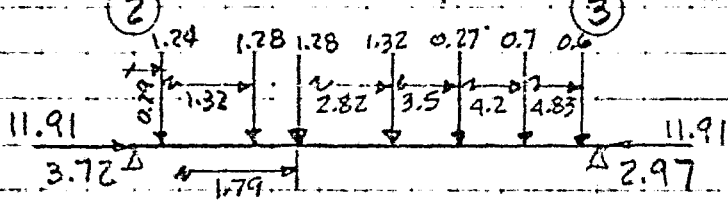


$M_{MAX} = 1.2T-M$

$M_{MAX} = 1.2T-M$

② ③

② ③

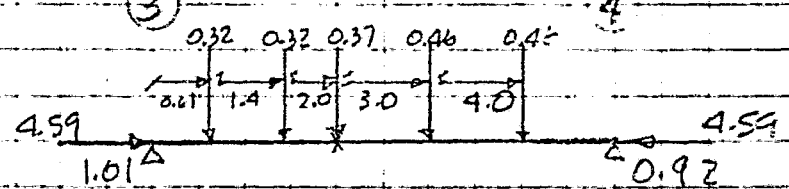
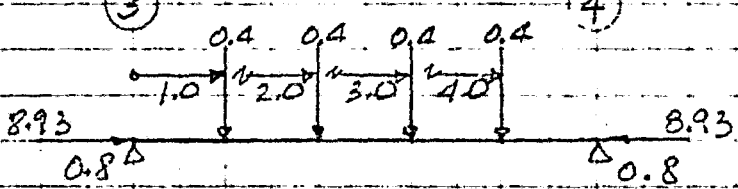


$M_{MAX} = 4.2T-M$

$M_{MAX} = 1.25T-M$

③ ④

③ ④

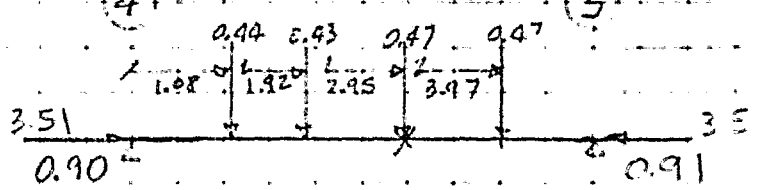
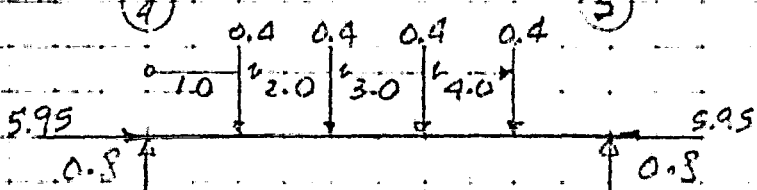


$M_{MAX} = 1.2T-M$

$M_{MAX} = 1.38T-M$

④ ⑤

④ ⑤



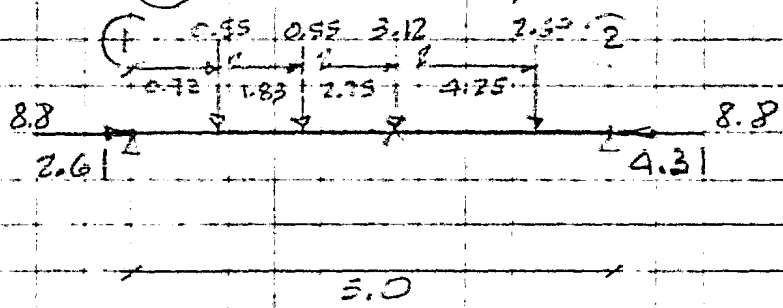
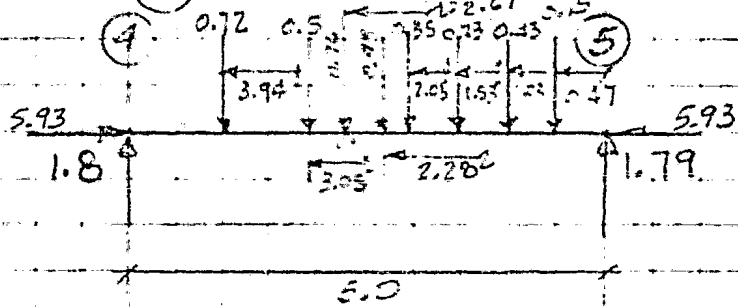
$M_{MAX} = 1.2T-M$

$M_{MAX} = 1.39T-M$

# TRABES LONGITUDINALES

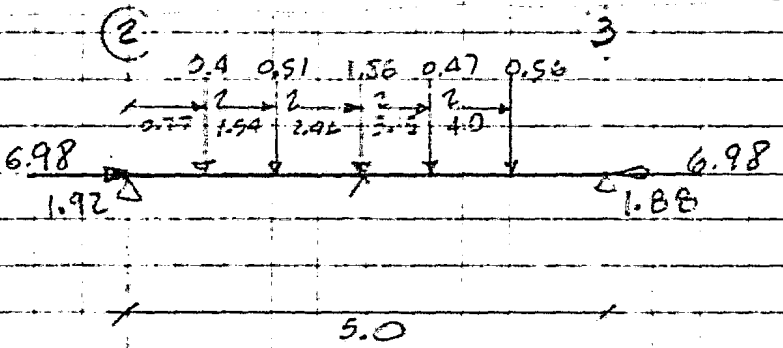
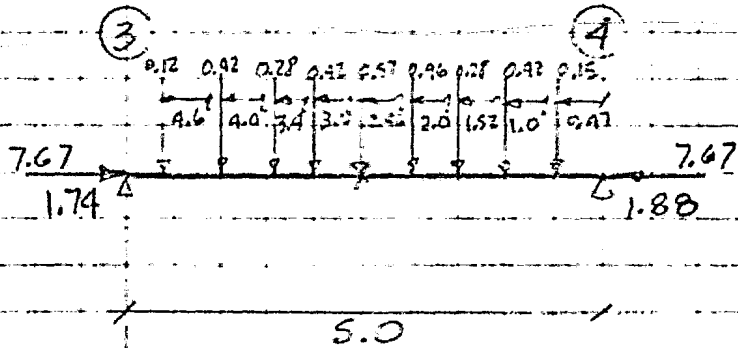
EJE (D) NIVEL T/S+3

EJE (C) NIVEL T/S+4



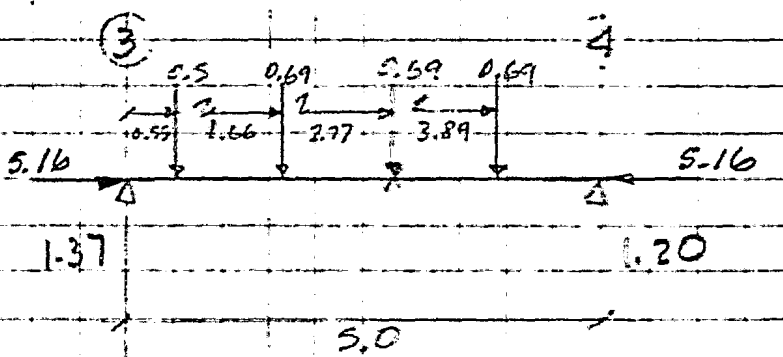
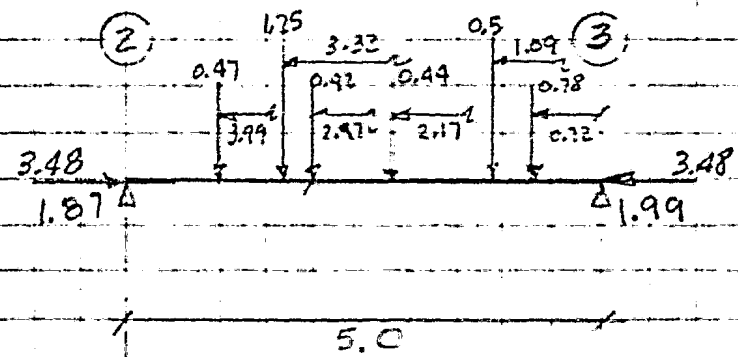
$M_{MAX} = 3.09 \text{ T-M}$

$M_{MAX} = 5.65 \text{ T-M}$



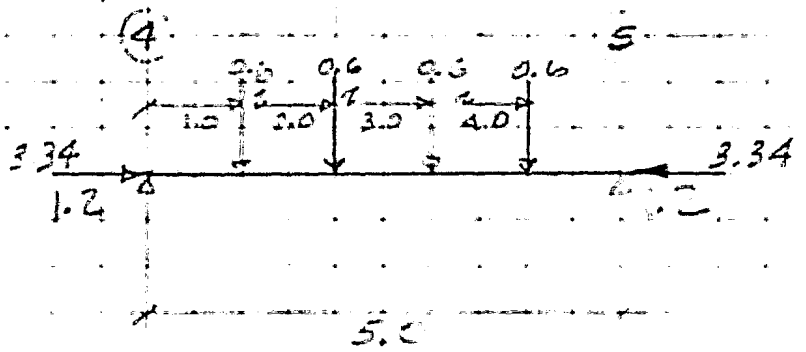
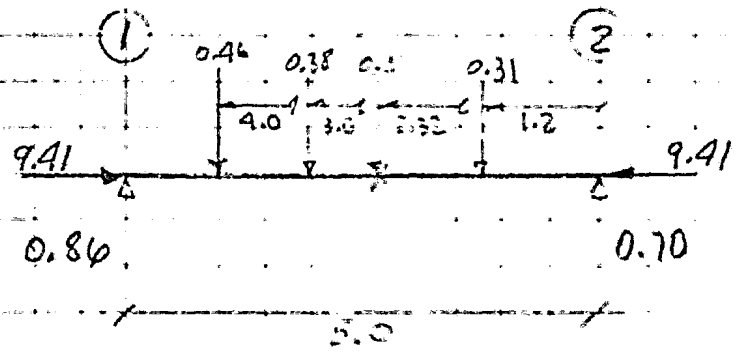
$M_{MAX} = 3.02 \text{ T-M}$

$M_{MAX} = 3.58 \text{ T-M}$



$M_{MAX} = 2.87 \text{ T-M}$

$M_{MAX} = 1.91 \text{ T-M}$



$M_{MAX} = 1.28 \text{ T-M}$

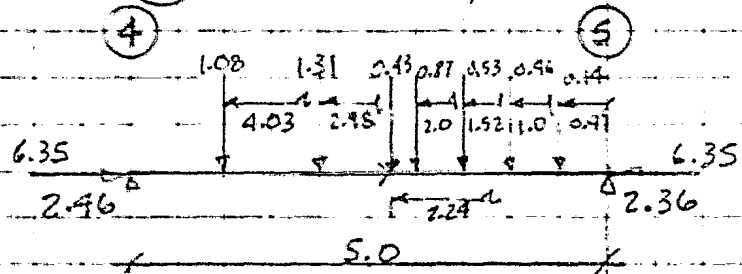
$M_{MAX} = 1.8 \text{ T-M}$



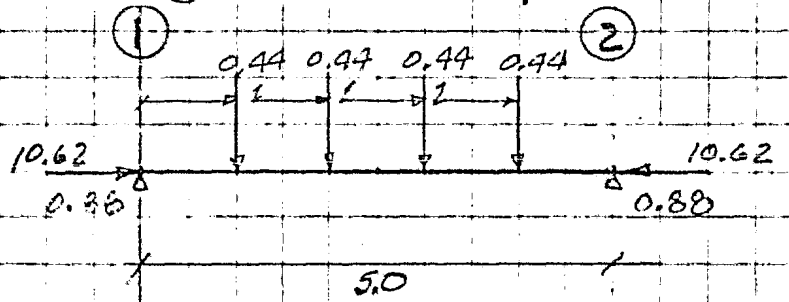
# TRABES LONGITUDINALES

EJE (D) NIVEL T/S+4

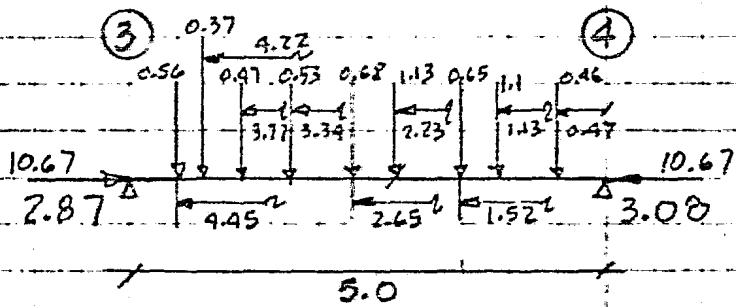
EJE (C) NIVEL T/S+5



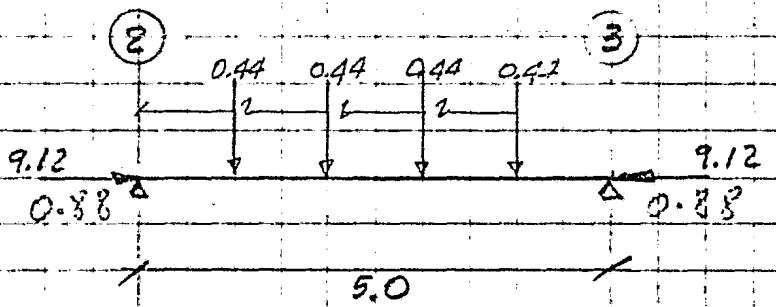
$M_{MAX} = 3.88 \text{ T-M}$



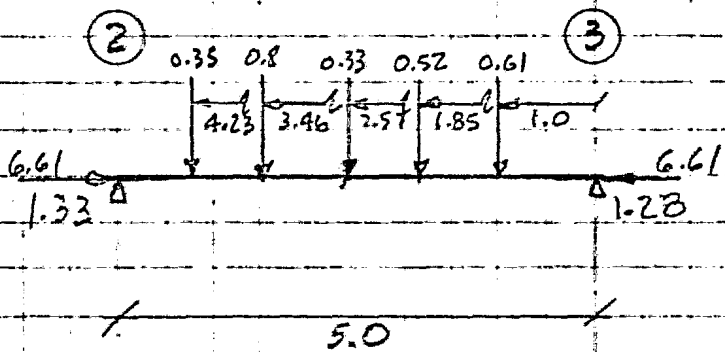
$M_{MAX} = 1.32 \text{ T-M}$



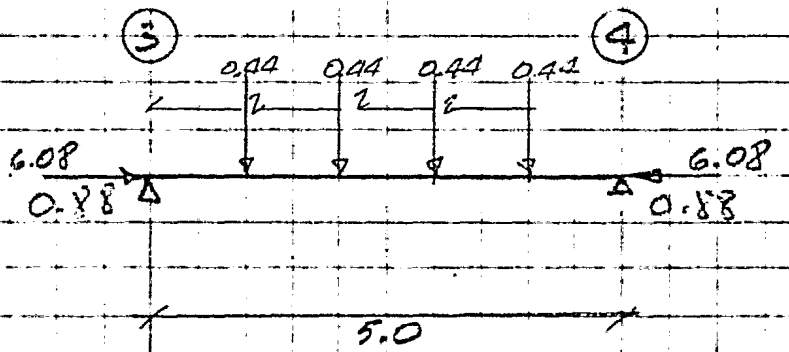
$M_{MAX} = 4.38 \text{ T-M}$



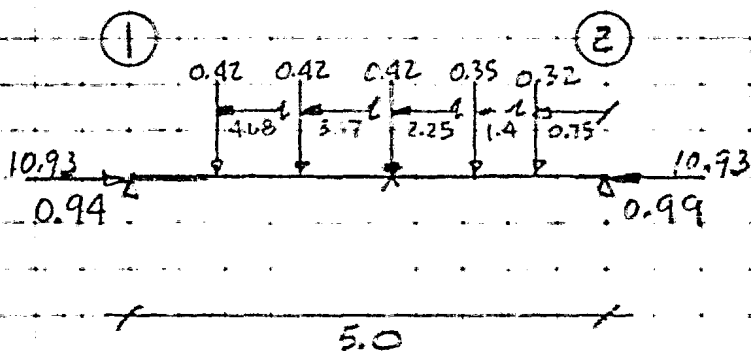
$M_{MAX} = 1.32 \text{ T-M}$



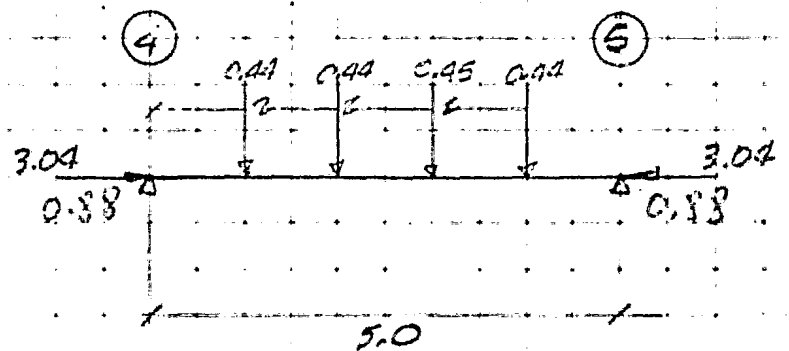
$M_{MAX} = 1.95 \text{ T-M}$



$M_{MAX} = 1.32 \text{ T-M}$



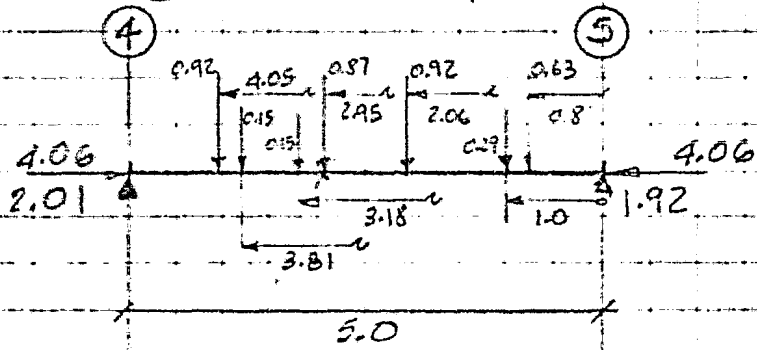
$M_{MAX} = 1.44 \text{ T-M}$



$M_{MAX} = 1.32 \text{ T-M}$

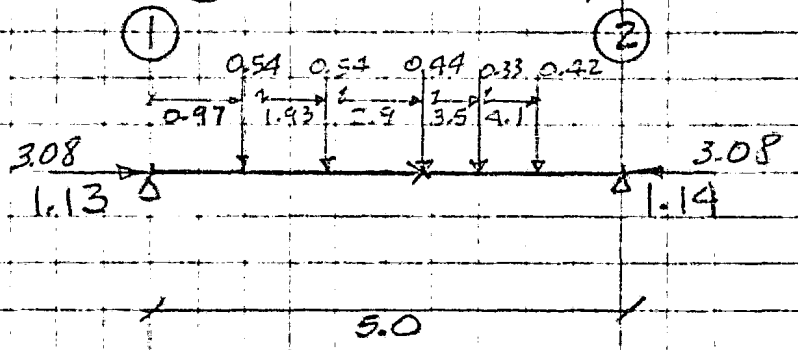
# TRABES LONGITUDINALES

EJE ④ NIVEL T/S+5



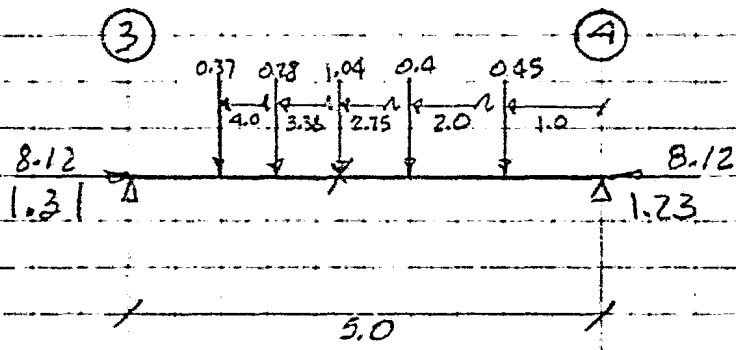
$M_{MAX} = 2.94 T-M$

EJE ③ NIVEL T/S+6

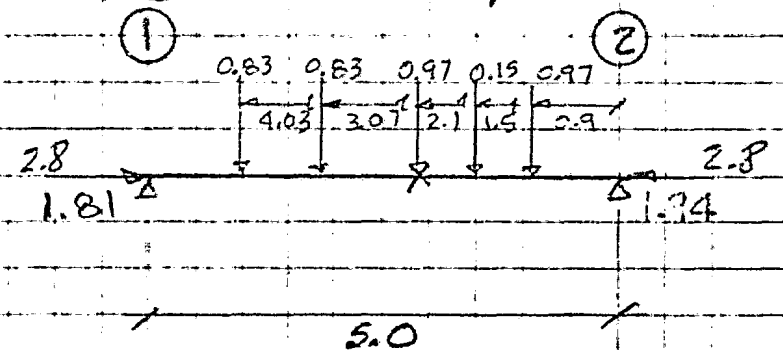


$M_{MAX} = 1.7 T-M$

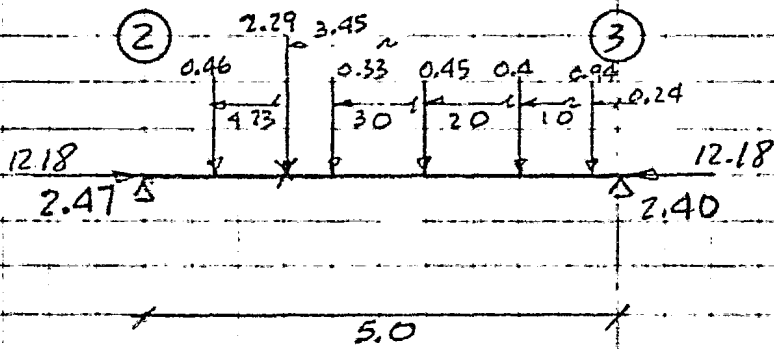
EJE ③ NIVEL T/S+6



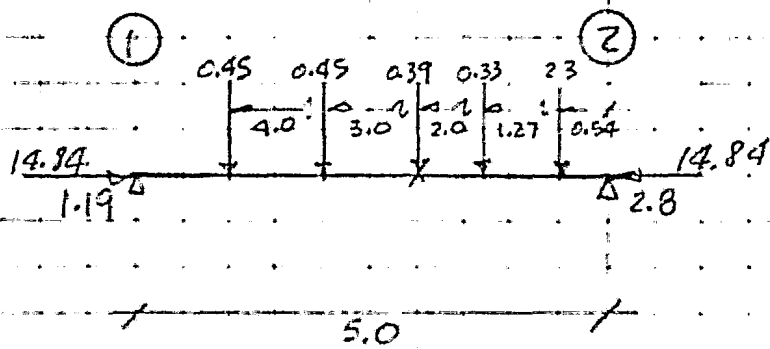
$M_{MAX} = 2.3 T-M$



$M_{MAX} = 2.83 T-M$



$M_{MAX} = 3.47$

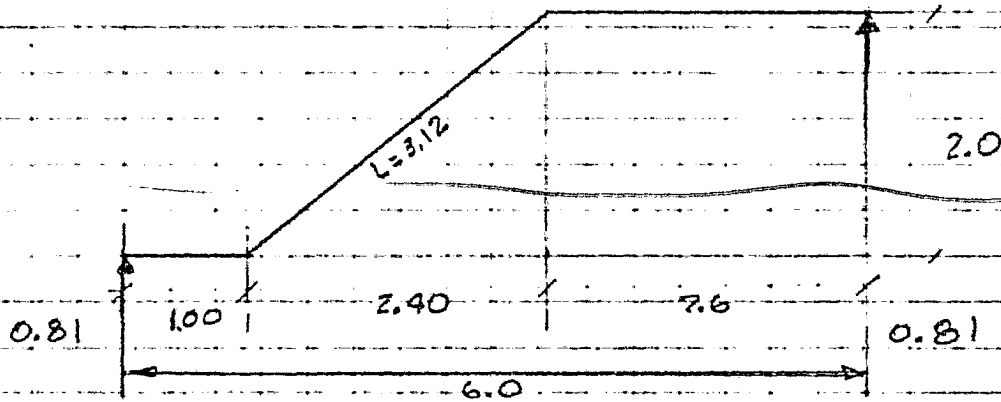


$M_{MAX} = 2.0 T-M$

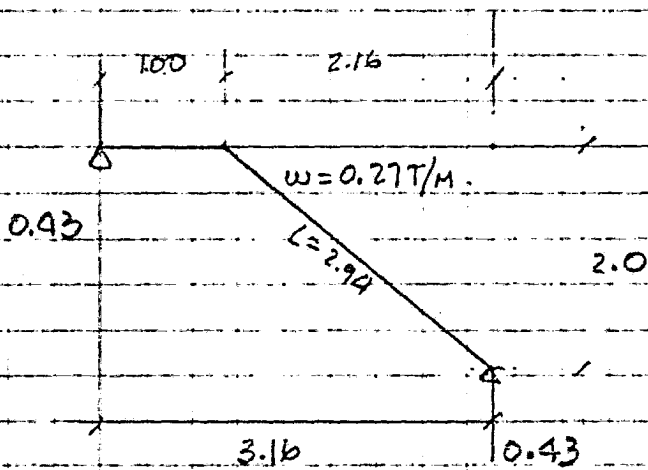
ESCALERAS:

$$w = 600 \text{ kg/m}^2$$

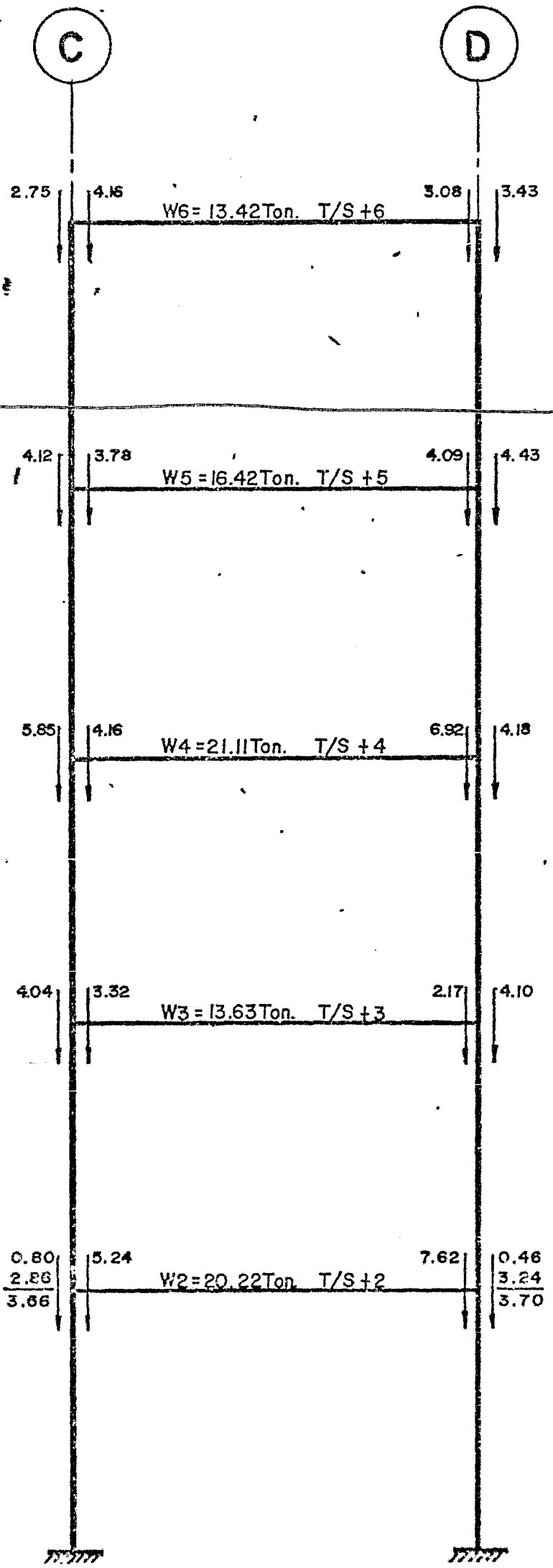
$$w = 0.6 \times 0.45 = 0.27 \text{ T/m}$$



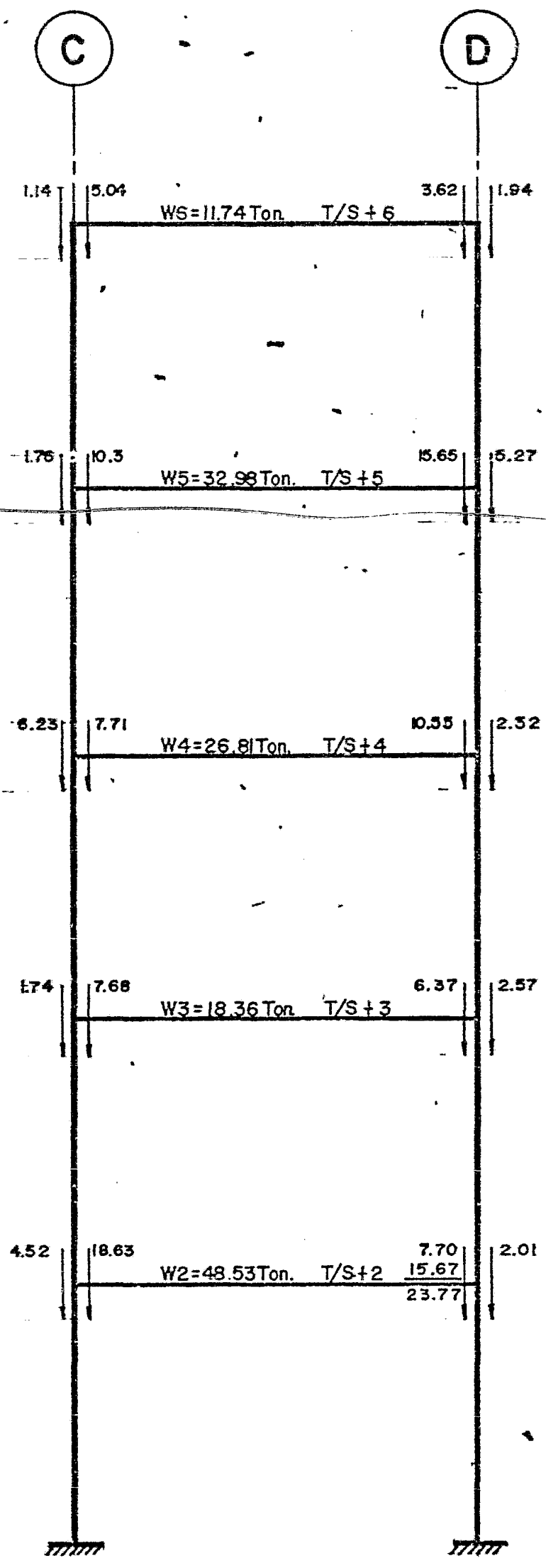
$$M_{\text{max}} = 1.22 \text{ TON-M}$$



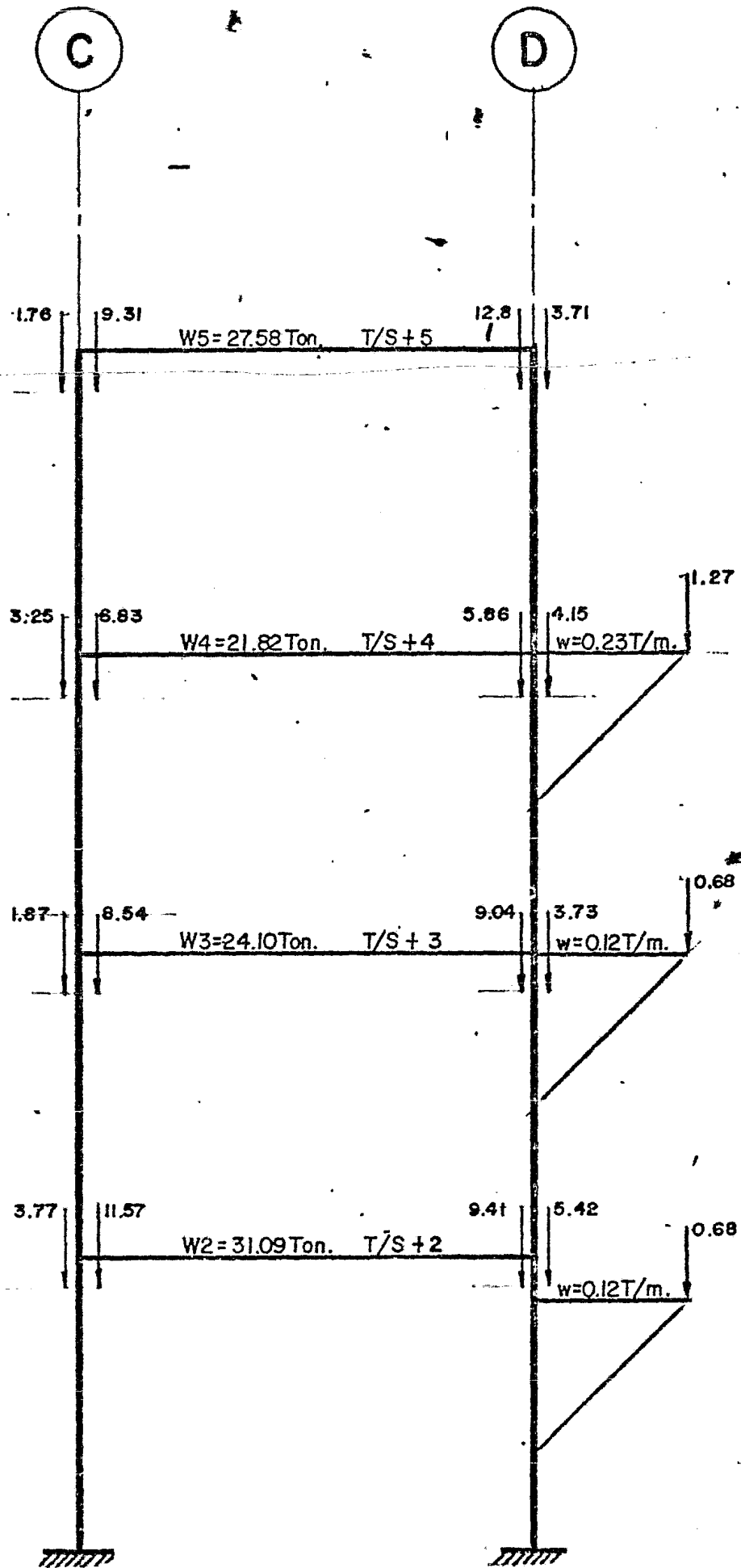
# MARCO EJE ①



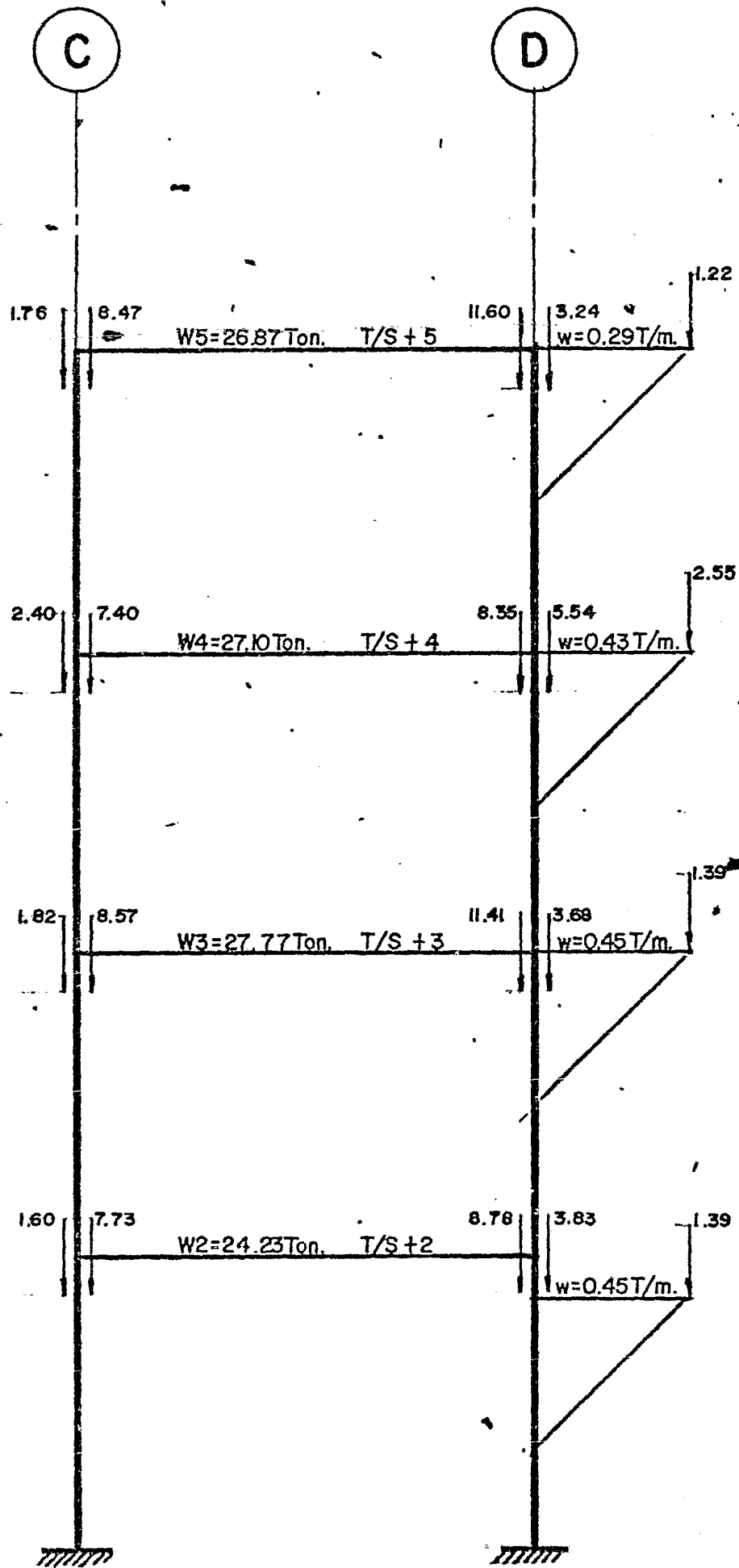
# MARCO EJE (2)



# MARCO EJE ③

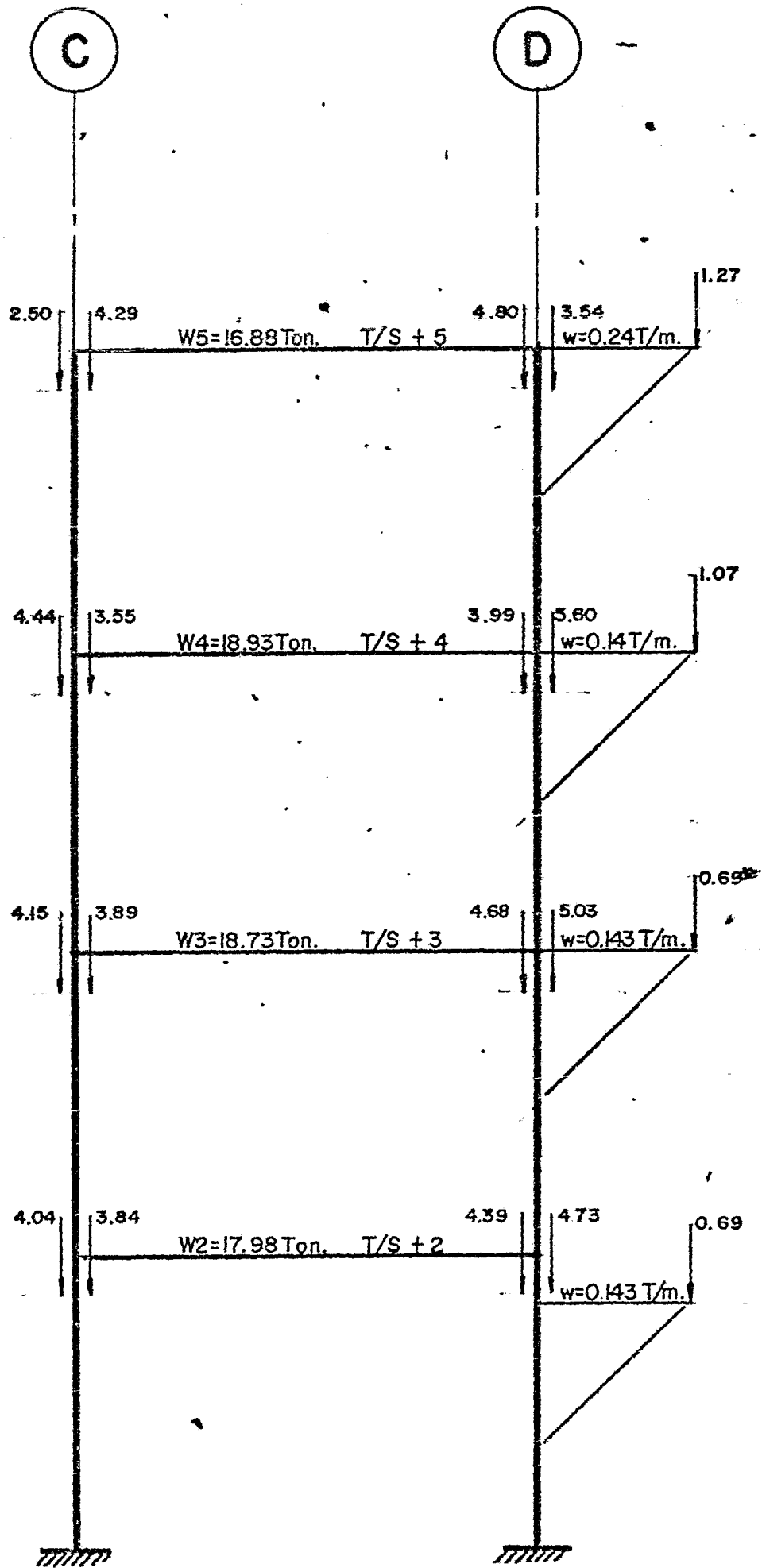


# MARCO EJE ④

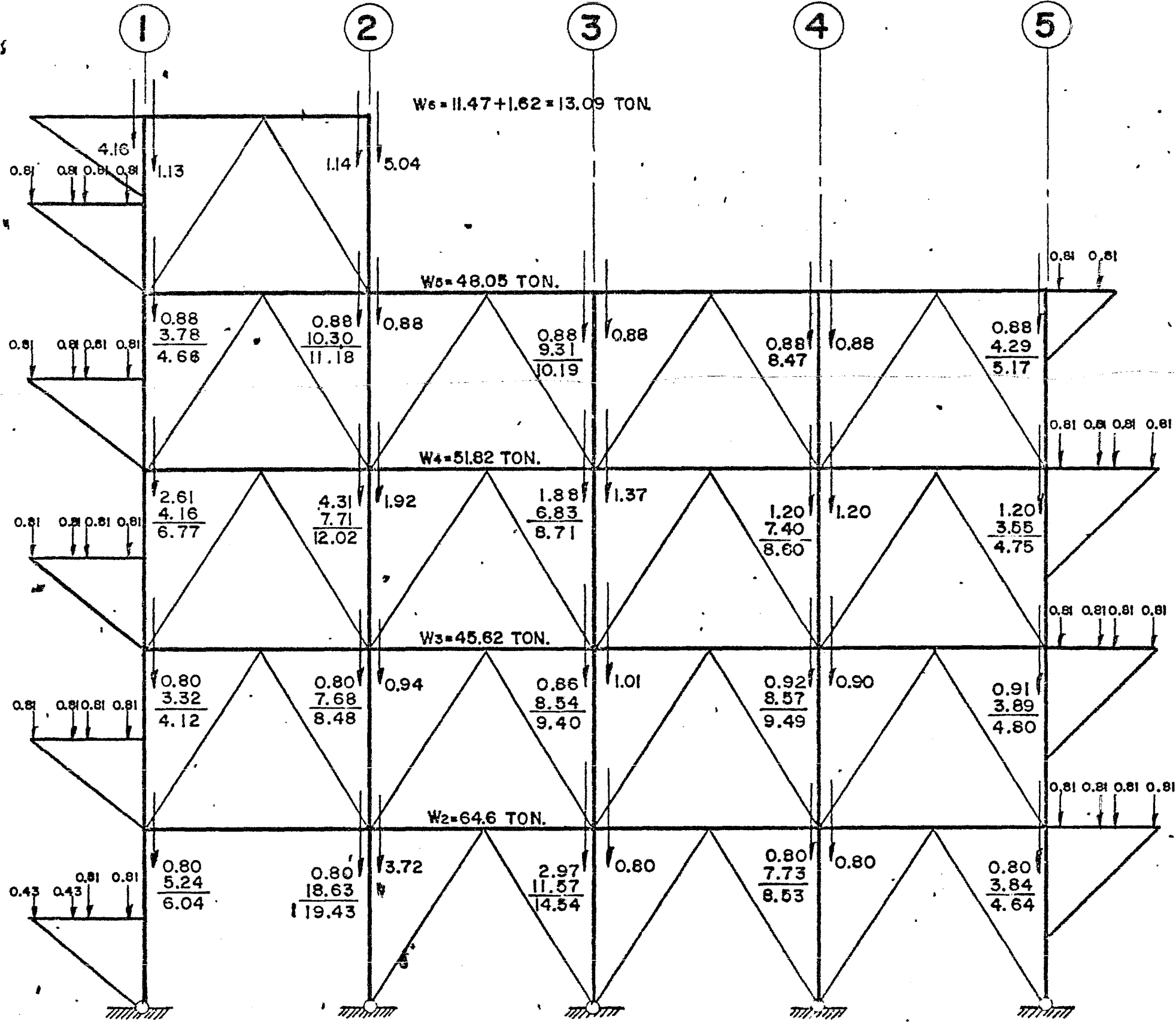


# MARCO EJE

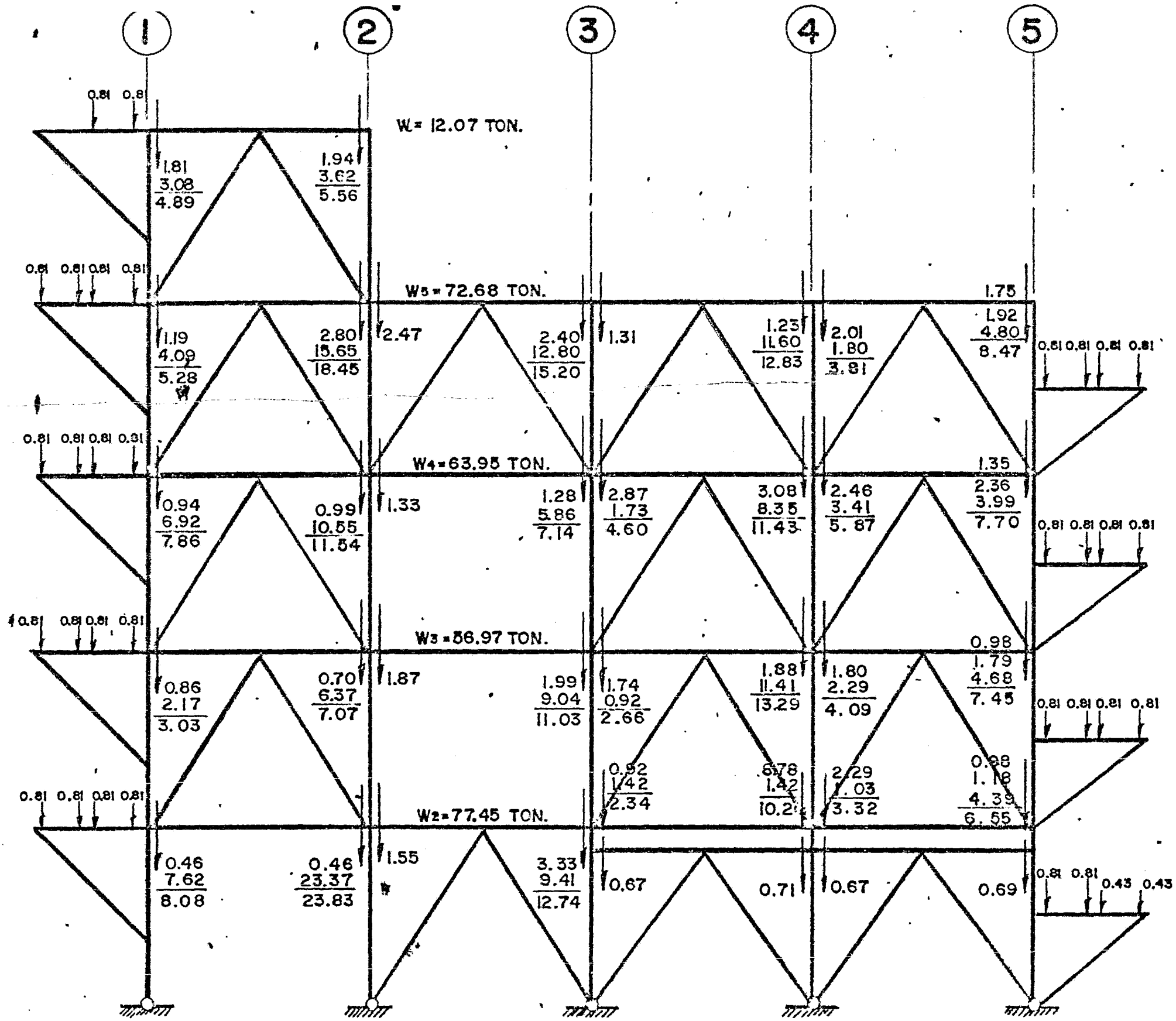
5







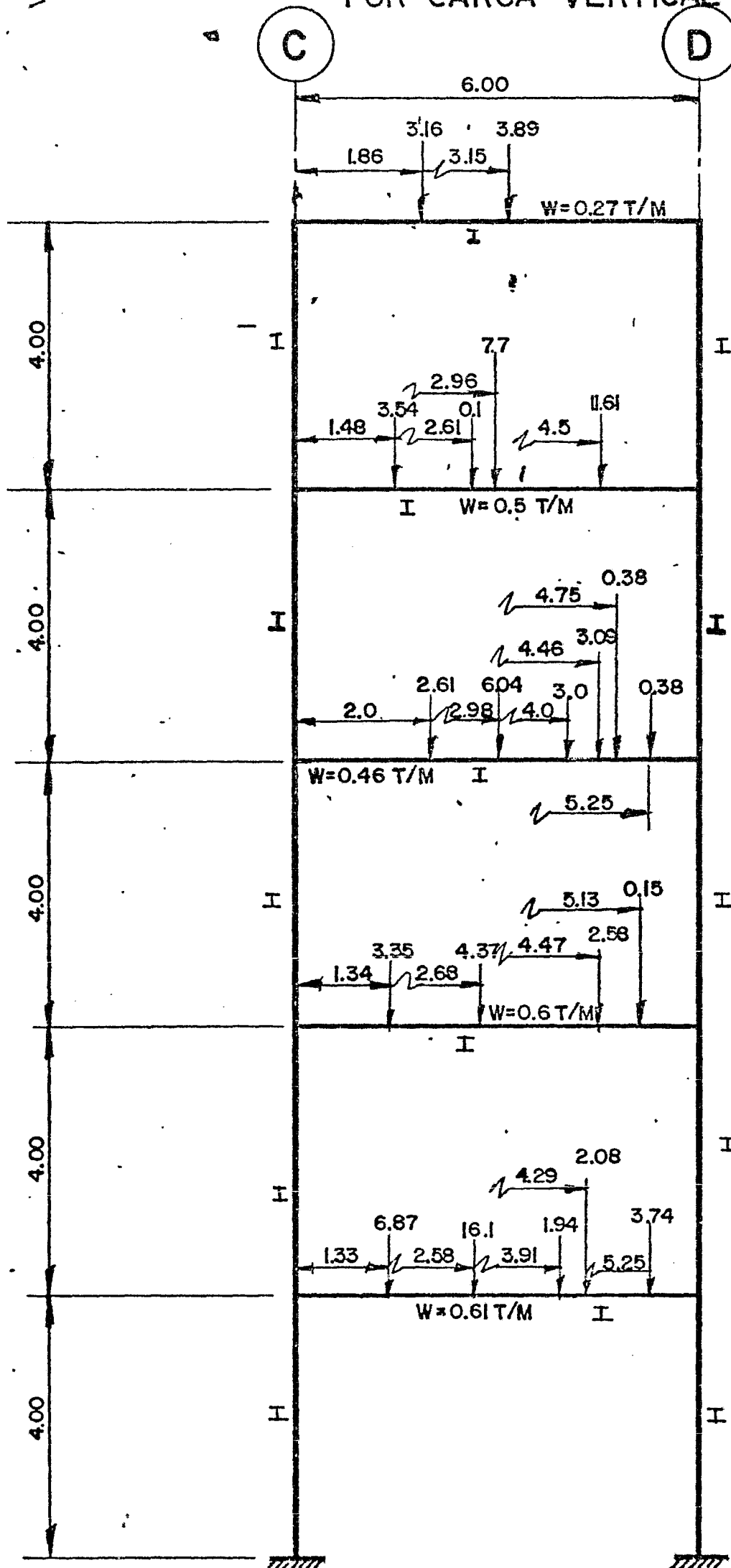
MARCO EJE (C)



MARCO EJE (D)

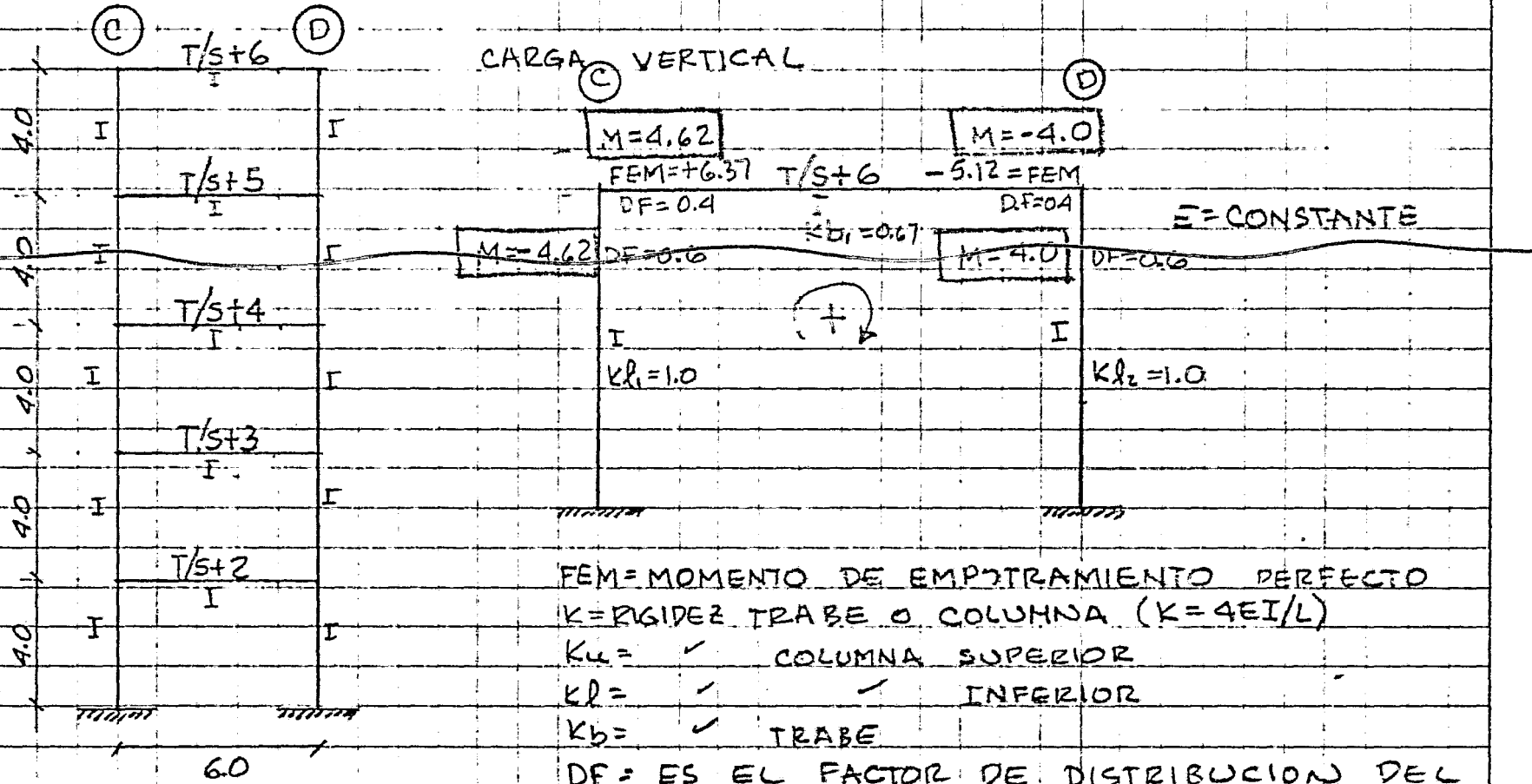
# MARCO EJE (2)

## ANALISIS PRELIMINAR POR CARGA VERTICAL



# ANALISIS PRELIMINAR DE MARCOS

ANALIZAREMOS EL MARCO DEL EJE (2) POR TENER LAS SOLICITACIONES MAYORES Y PARA PROPONER SECCIONES PARA EL ANALISIS DEFINITIVO POR COMPUTADORA.



FEM = MOMENTO DE EMPOTRAMIENTO PERFECTO  
 K = RIGIDEZ TRABE O COLUMNA ( $K = 4EI/L$ )  
 $K_u =$  ✓ COLUMNA SUPERIOR  
 $K_l =$  ✓ INFERIOR  
 $K_b =$  ✓ TRABE  
 DF = ES EL FACTOR DE DISTRIBUCION DEL NUDO

$$DF = \frac{K}{K_u + K_l + K_b + K_r}$$

CALCULO DE RIGIDECES.

CALCULO DE FACTORES DE DISTRIBUCION

$$K_{b1} = \frac{4EI}{L} = \frac{4EI}{6} = 0.67EI$$

$$DF_{(3)} = \frac{0.67}{1.67} = 0.4$$

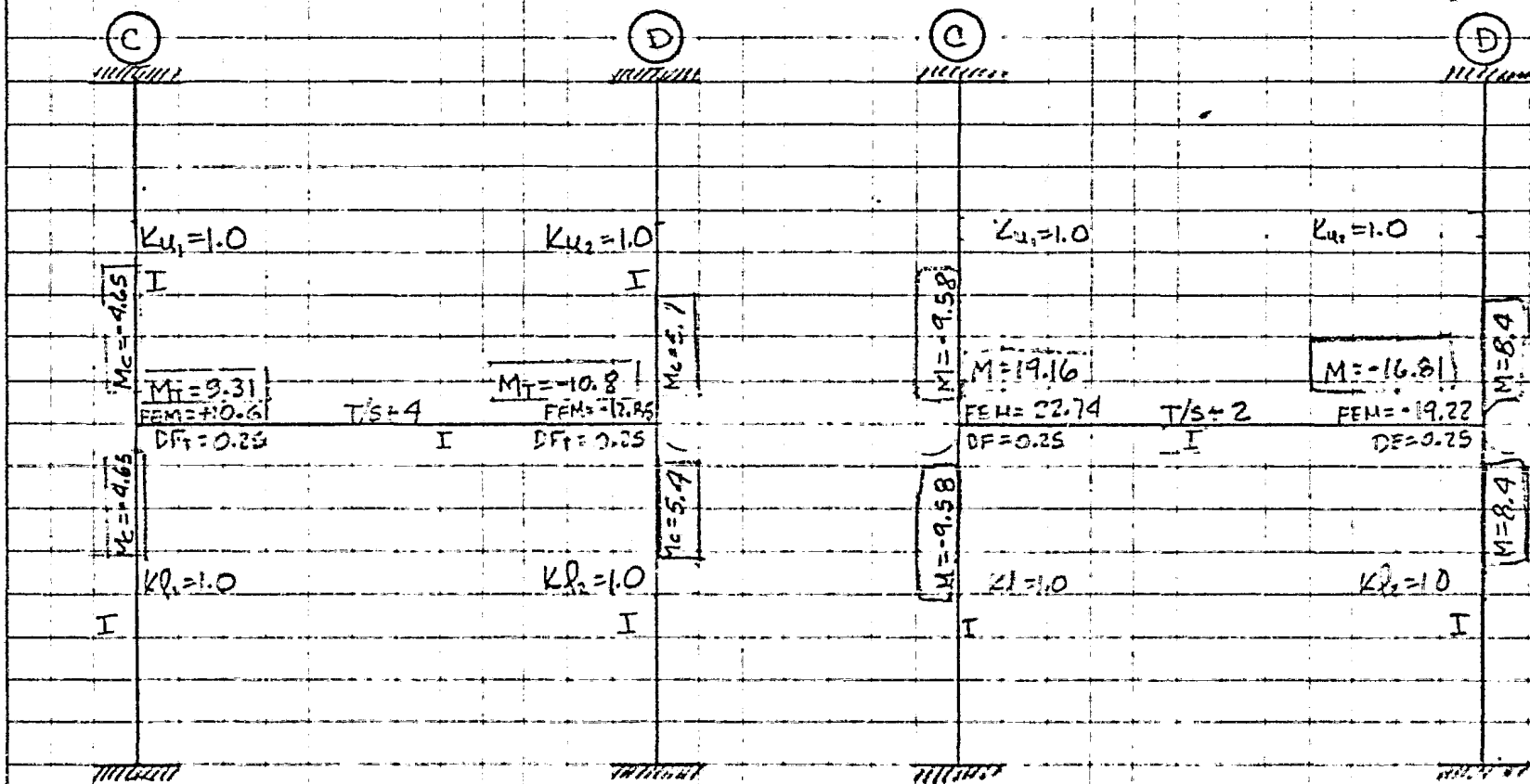
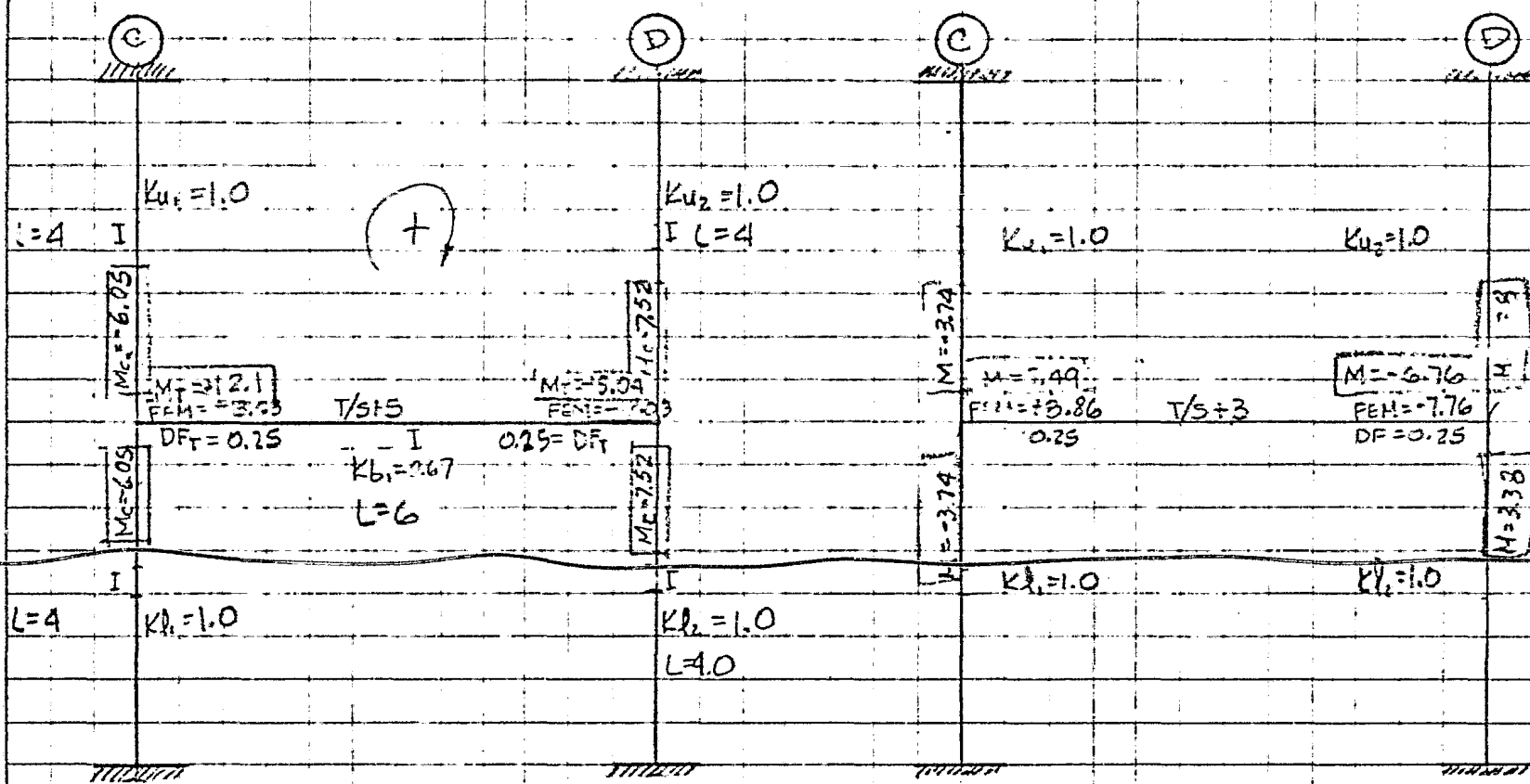
$$K_l = K_{l2} = \frac{4EI}{L} = \frac{4EI}{4} = 1.0EI$$

$$DF_{(2)} = \frac{1.0}{1.67} = 0.6$$

$$K_u = K_{u2} = 0$$

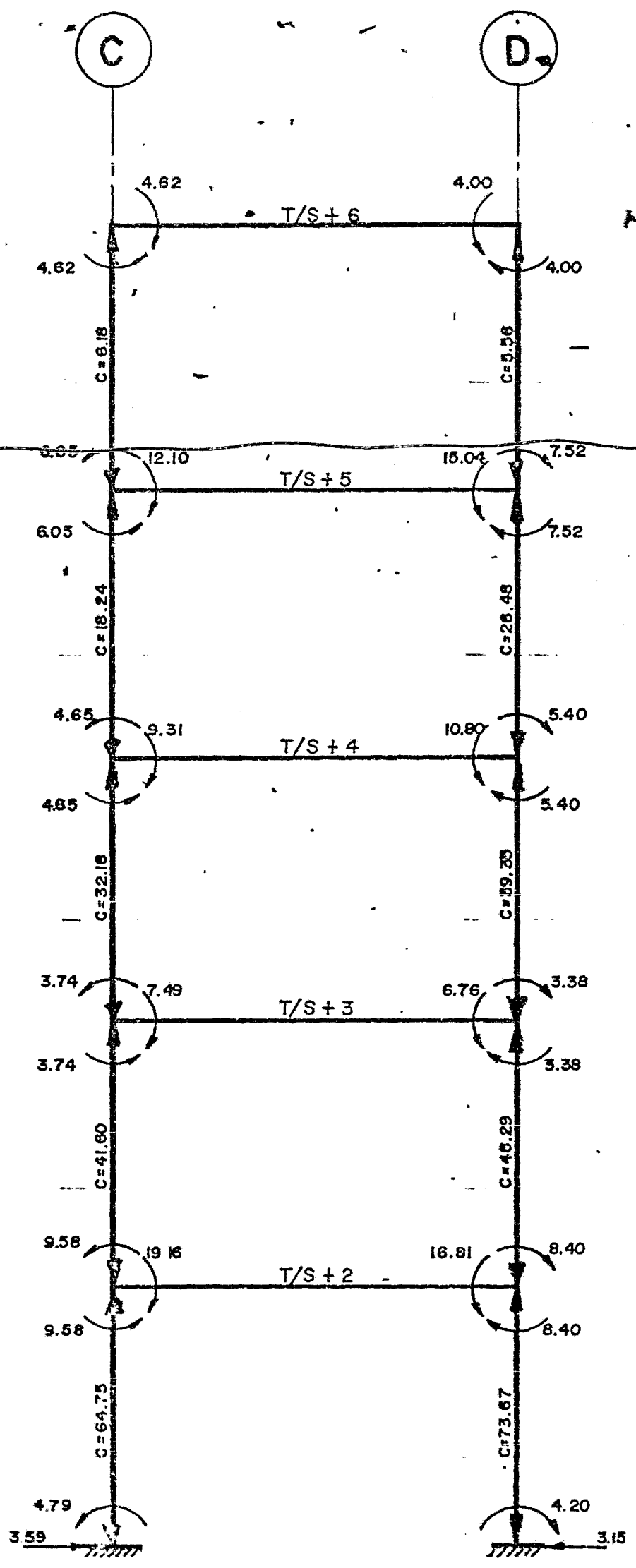
$$1.67EI$$

EL ANALISIS SE EJECUTO CON EL PROGRAMA DEL MODULO DE ANALISIS ESTRUCTURAL DE LA CALCULADORA HP 41CV. EL METODO DE ANALISIS ES EL DE DISTRIBUCION DE MOMENTOS (H. CROSS).

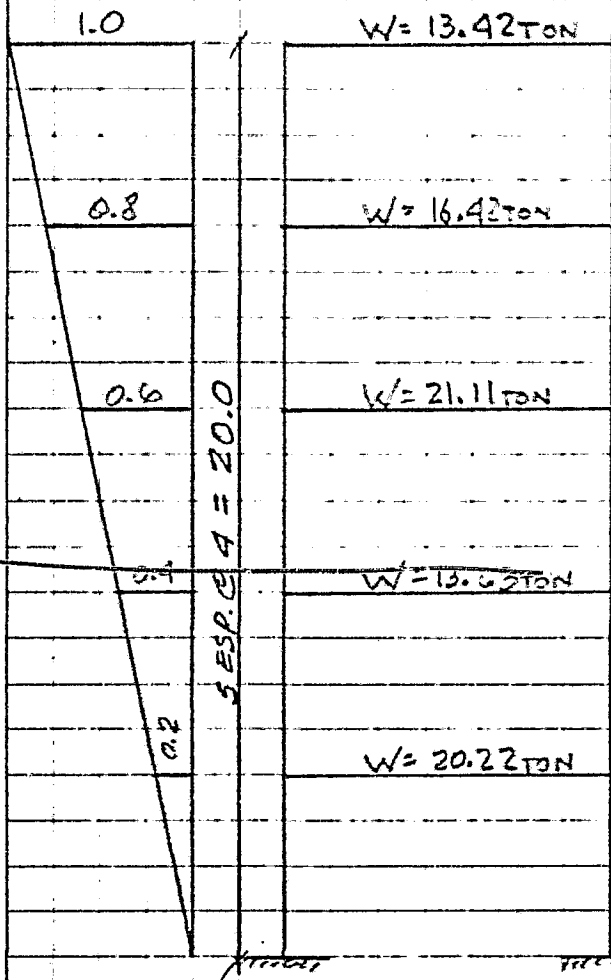


# MARCO EJE (2)

CARGA VERTICAL



# CALCULO DE FUERZAS SISMICAS PRELIMINARES.



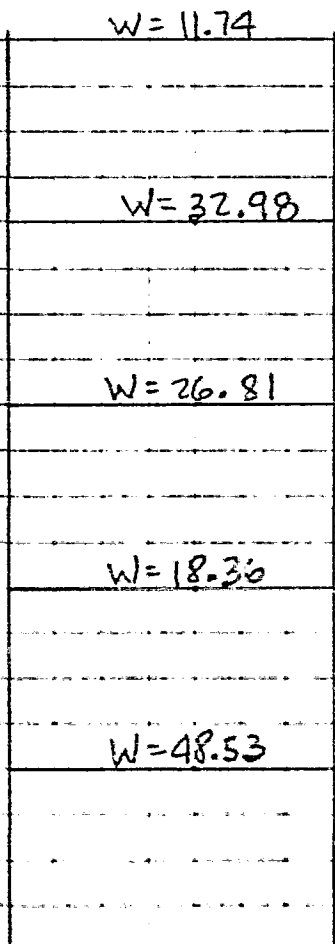
	$W_i$ (TON)	$F_s$	$V_s^i$	$V_s$	$F_s$
5	13.42	13.42	13.42	2.80	2.80
4	16.42	13.14	26.56	5.55	2.75
3	21.11	12.67	39.23	8.19	2.64
2	13.63	5.45	44.68	9.33	1.14
1	20.22	4.04	48.72	10.18	0.85

$\Sigma 84.80$

$V_b = 84.8 \times 0.12 = 10.18 \text{ TON}$

$K = \frac{10.18}{48.72} = 0.21$

## MARCO EJE ①



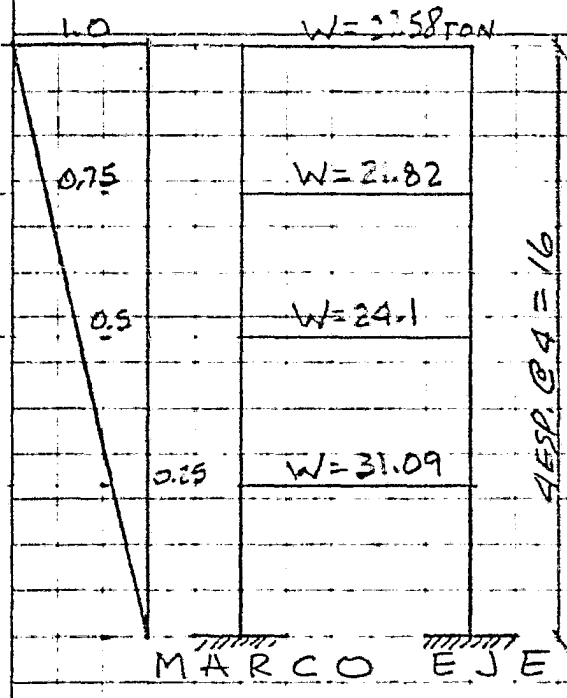
	$W_i$	$F_s$	$V_s^i$	$V_s$	$F_s$
5	11.74	11.74	11.74	2.74	2.74
4	32.98	26.38	38.12	8.89	6.15
3	26.81	16.09	54.21	12.64	3.75
2	18.36	7.34	61.55	14.35	1.71
1	48.53	9.71	71.26	16.61	2.26

$\Sigma 138.42$

$V_b = 138.42 \times 0.12 = 16.61 \text{ TON}$

$K = \frac{16.61}{71.26} = 0.23$

## MARCO EJE ②

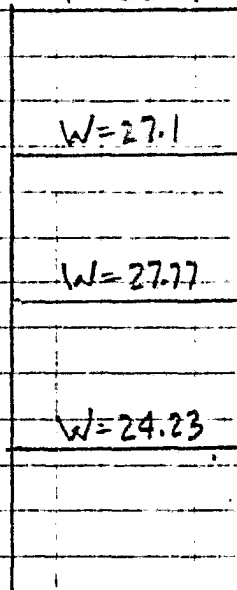


	W <sub>i</sub>	F <sub>s</sub>	V <sub>s</sub>	V <sub>s</sub>	F <sub>s</sub>
4	21.58	27.58	27.58	5.43	5.43
3	21.82	16.37	43.95	8.65	3.22
2	24.1	12.05	56.0	11.02	2.37
1	31.09	7.77	63.77	12.55	1.53
Σ	104.59				

$$V_b = 0.12 \times 104.59 = 12.55$$

$$K = 0.20$$

W = 26.87



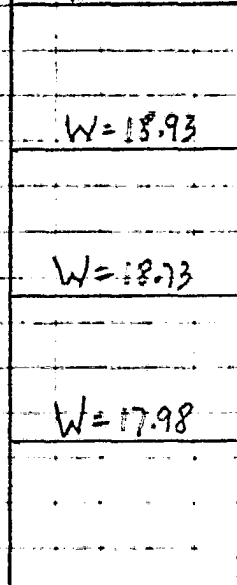
	W <sub>i</sub>	F <sub>s</sub>	V <sub>s</sub>	V <sub>s</sub>	F <sub>s</sub>
	26.87	26.87	26.87	5.09	5.09
	27.1	20.33	47.2	8.94	3.85
	27.77	13.89	61.09	11.57	2.63
	24.23	6.06	67.15	12.72	1.15
Σ	105.97				

$$\Sigma 105.97$$

$$V_b = 105.97 \times 0.12 = 12.72$$

$$K = 0.19$$

W = 16.88



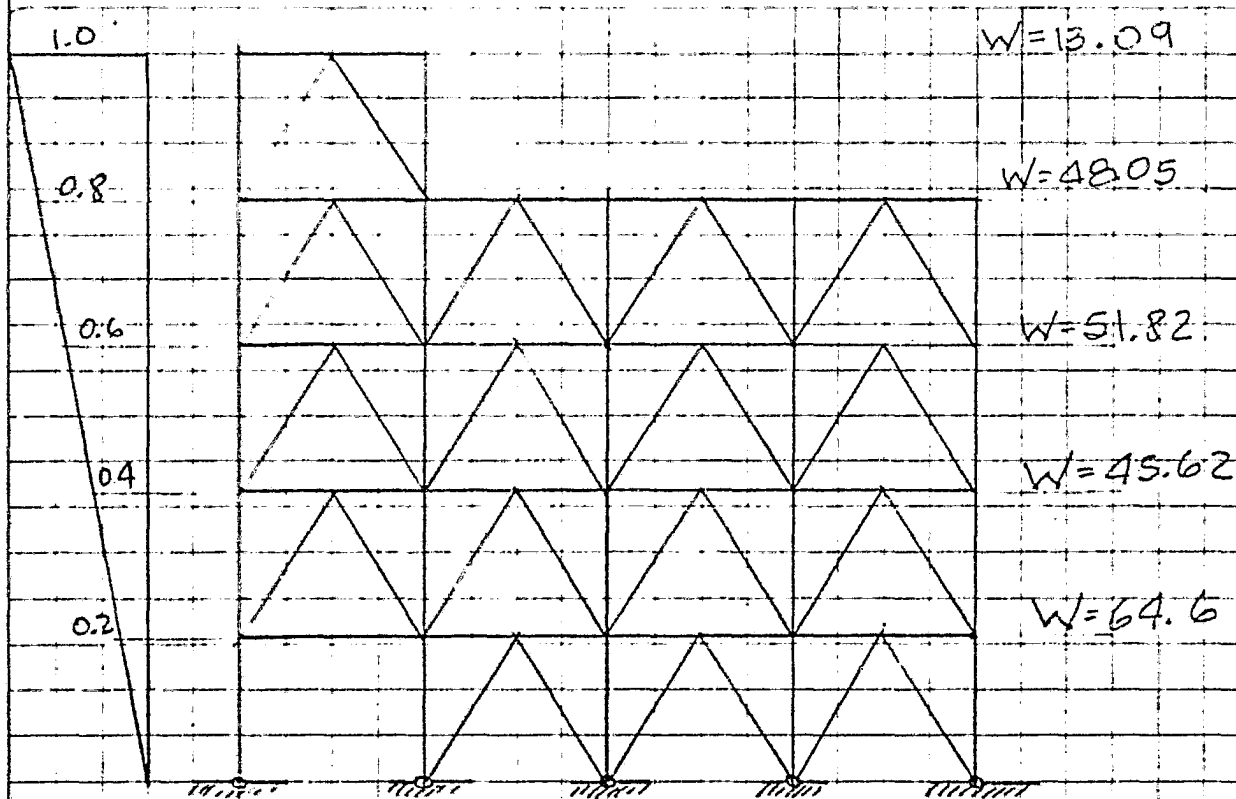
	W <sub>i</sub>	F <sub>s</sub>	V <sub>s</sub>	V <sub>s</sub>	F <sub>s</sub>
4	16.88	16.88	16.88	3.27	3.27
3	18.93	14.2	31.08	6.02	2.75
2	18.73	9.37	40.45	7.83	1.81
1	17.98	4.5	44.95	8.7	0.87
Σ	72.52				

$$\Sigma 72.52$$

$$V_b = 72.52 \times 0.12 = 8.7$$

$$K = 0.19$$





MARCO EJE (C)

	$W_i$	$F_s'$	$V_s'$	$V_s$	$F_s$
5	13.09	13.09	13.09	3.08	3.08
4	48.05	38.44	51.53	12.13	9.05
3	51.82	31.09	82.62	19.44	7.31
2	45.62	18.25	100.87	23.74	4.30
1	64.6	12.92	113.79	26.78	3.04

$\Sigma 223.18$

$$V_b = 0.12 \times 223.18 = 26.78$$

$$K = 0.24$$

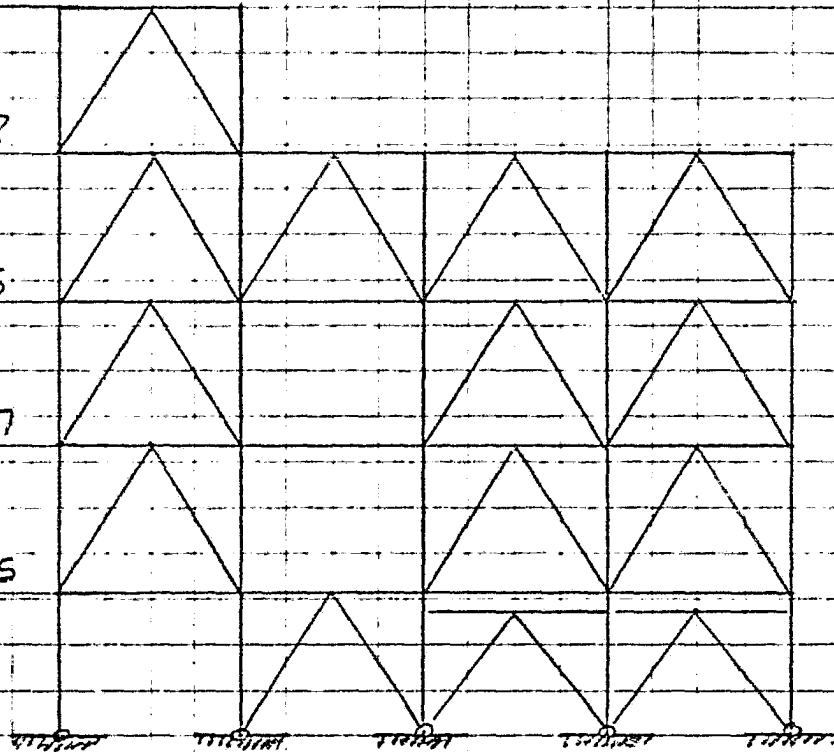
$$W = 12.07$$

$$W = 72.68$$

$$W = 63.95$$

$$W = 56.97$$

$$W = 17.45$$



MARCO EJE (D)

	W <sub>i</sub>	F <sub>s</sub>	V <sub>s</sub>	V <sub>s</sub>	F <sub>s</sub>
5	12.07	12.07	12.07	2.79	2.79
4	72.68	58.14	70.21	16.24	13.45
3	63.95	38.37	108.58	25.11	8.87
2	56.97	22.79	131.37	30.38	5.27
1	17.45	15.49	146.88	33.97	3.59

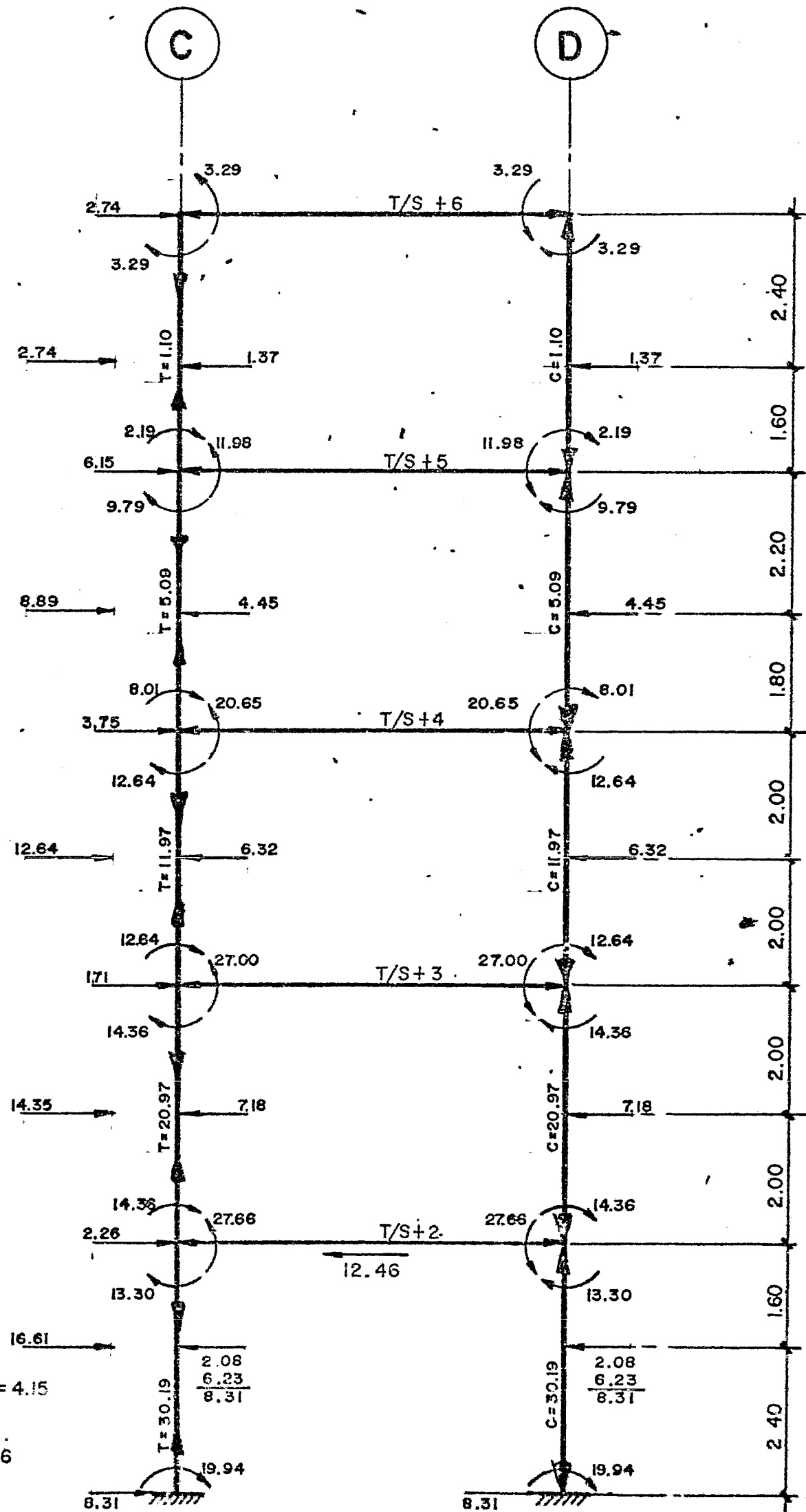
$$\Sigma 283.12$$

$$V_b = 0.12 \times 283.12 = 33.97$$

$$K = 0.23$$

# MARCO EJE (2)

## ANALISIS SISMICO PRELIMINAR



$$V_{col} = \frac{1 - 0.5}{1 + 1} \times 16.61 = 4.15$$

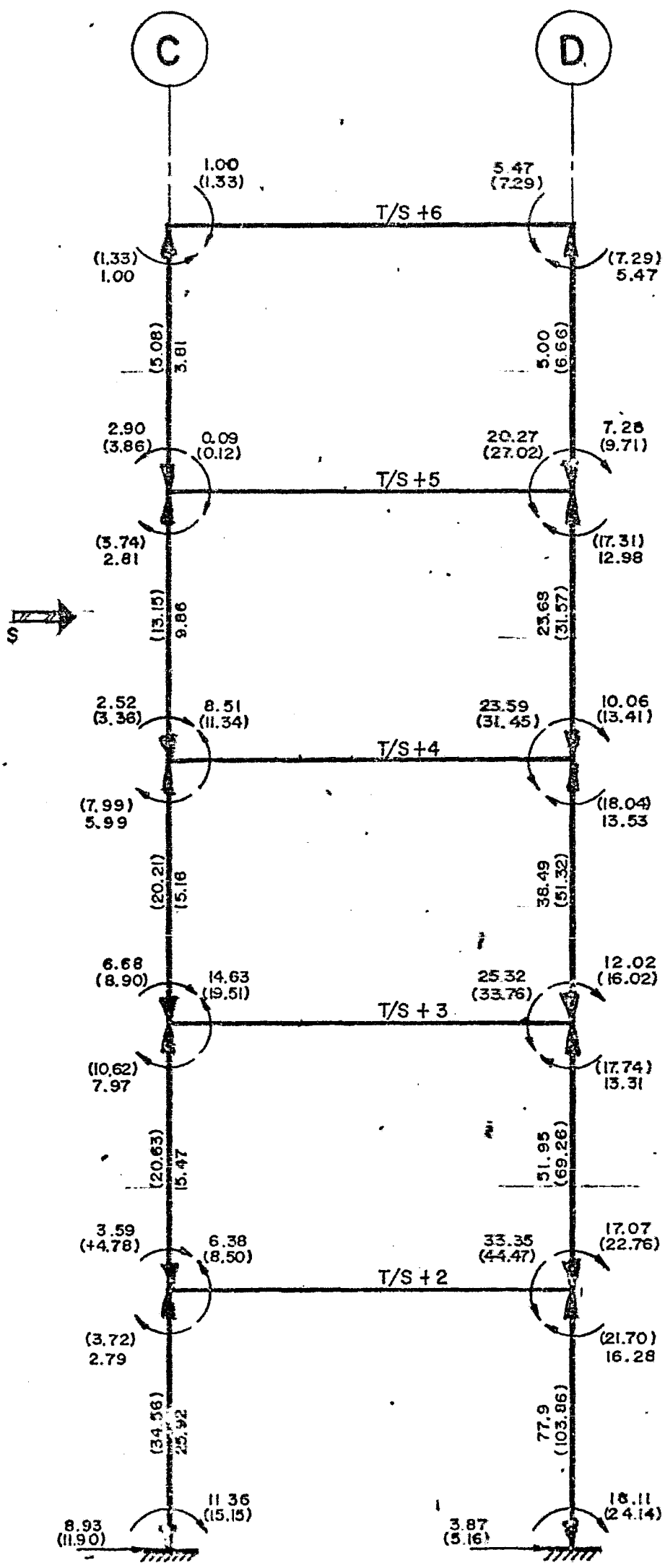
$$V_T = 16.61 - 4.15 = 12.46$$

2.08
6.23
8.31

2.08
6.23
8.31

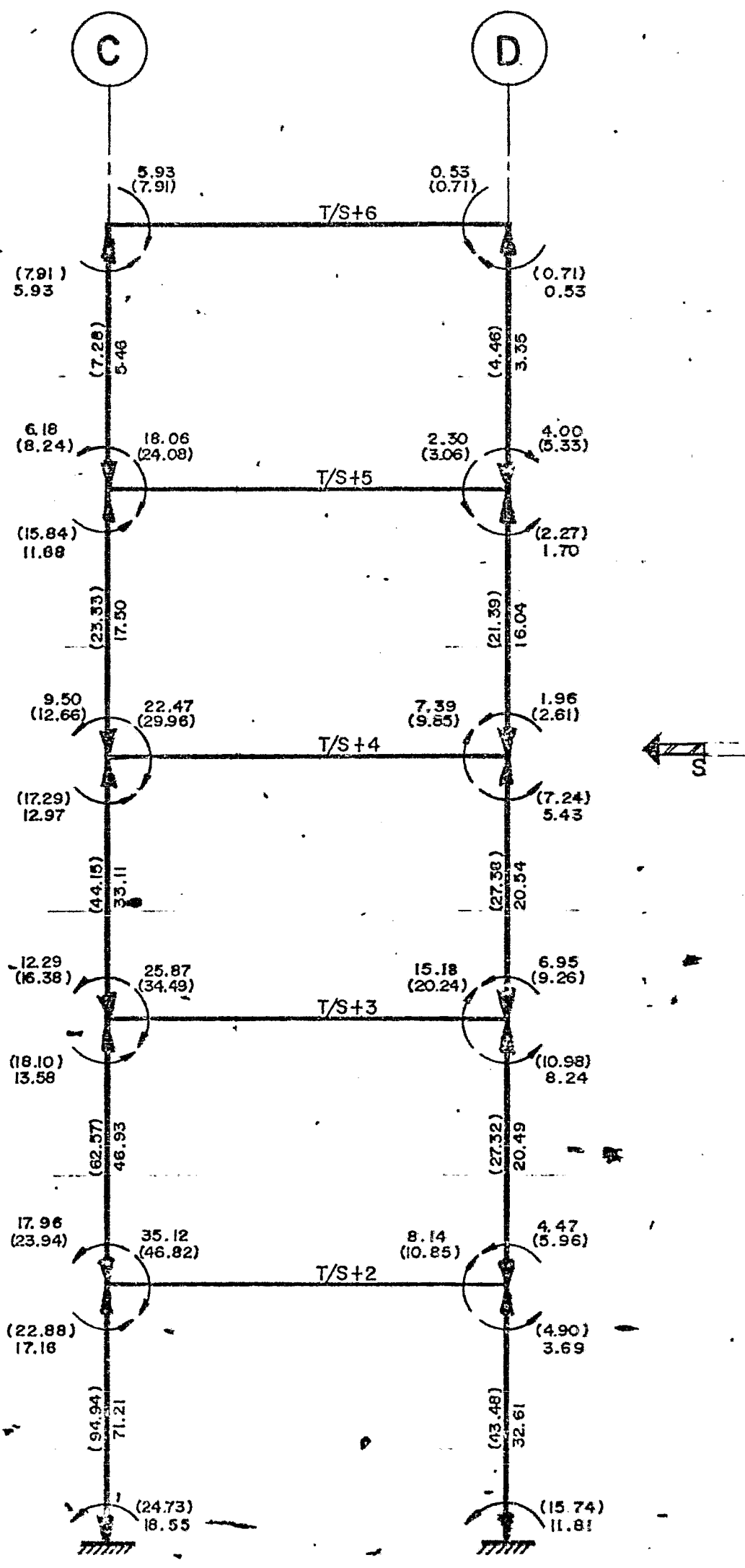
# MARCO EJE (2)

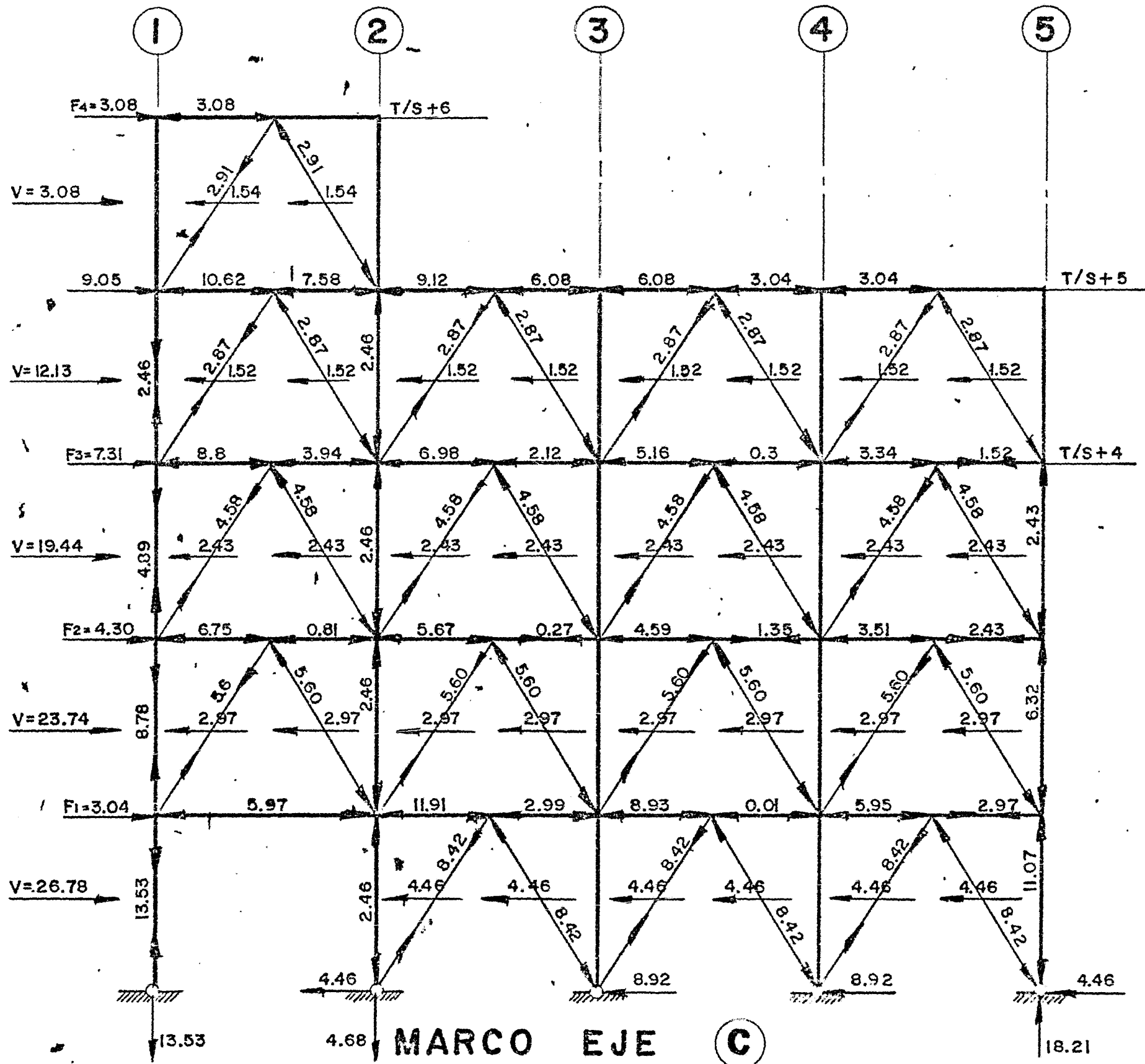
(CARGA VERTICAL + SISMO) 0.75 = X

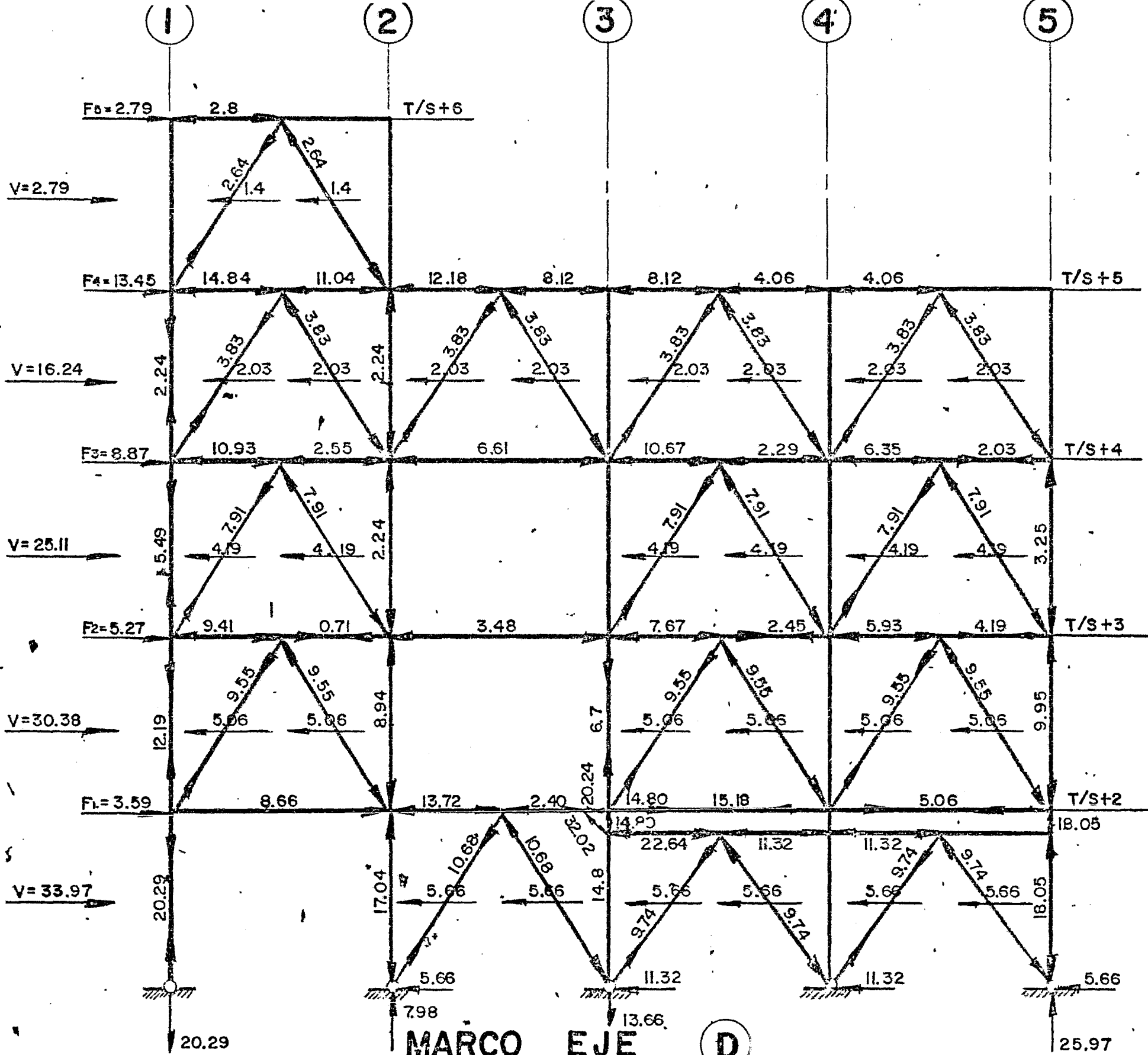


# MARCO EJE ②

(CARGA VERTICAL + SISMO) 0.75 = X





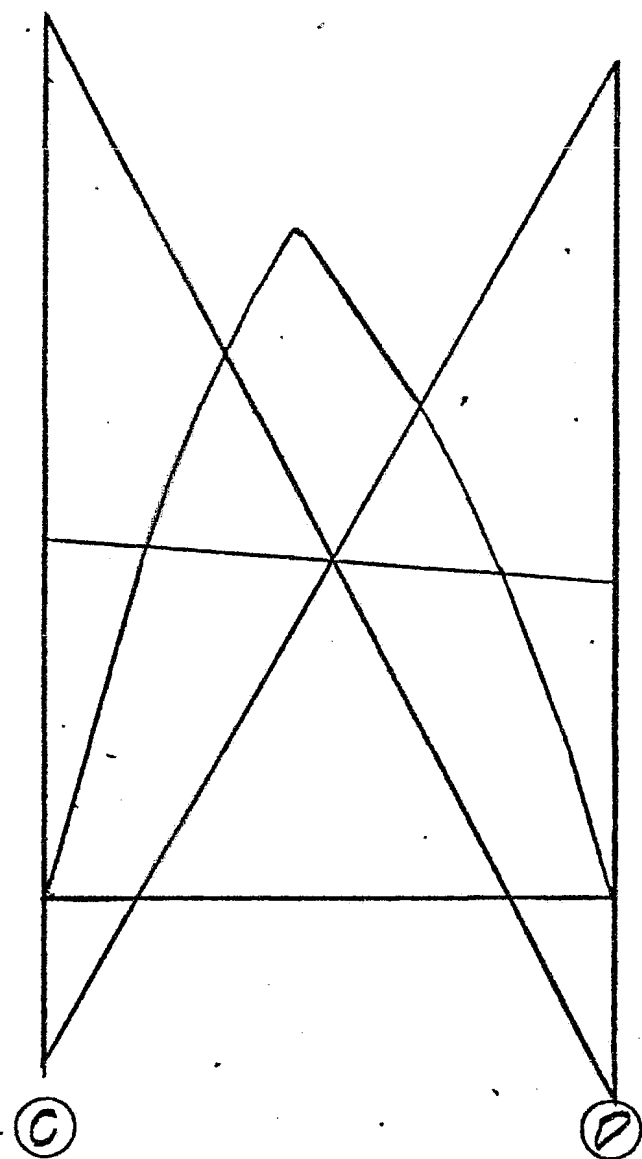


**MARCO EJE D**

**ANALISIS SISMICO PRELIMINAR**

# DISEÑO PRELIMINAR TRABES TRANSVERSALES

TRABE EJE (2) NIVEL T/S+2

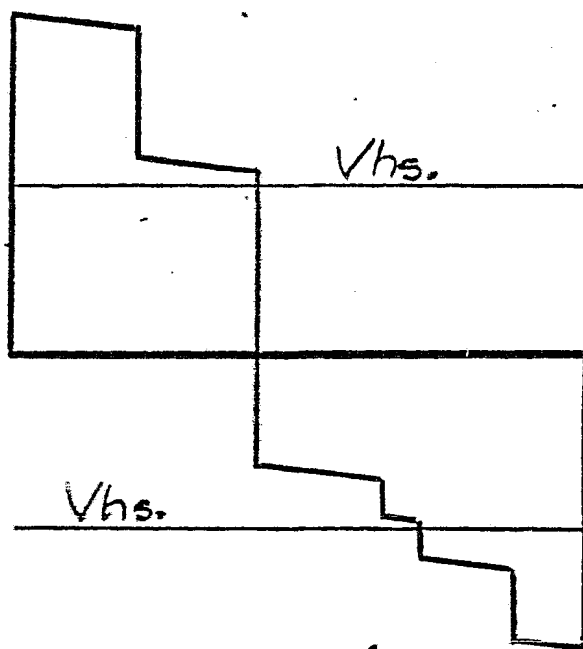


ESC: HOR. 1:100

ESC: VER. 1:500

ACERO A-36

FY = 2530 KG/CM<sup>2</sup>



## ELEMENTOS MECANICOS

### MAXIMOS

V	18.04	= 16.26
Vh	0.39	= 0.39
Vr	18.43	= 15.87

$V = (18.43 + 9.22) / 1000 \times 0.75 = 20,740 \text{ Kg.}$

$M = 35.12 \times 10^5 \text{ Kg-cm.}$

$L = 600 \text{ cms.}$

$L_b = 134 \text{ cms.}$

$V_h = \frac{19.16 - 16.81}{6} = 0.39$

$V_{hs} = \frac{27.66 + 27.66}{6} = \pm 9.22 \text{ TON.}$

Si probamos con una I.P.E. 18" 8 3/4" x 126.7

### PROPIEDADES:

$A = 161.09 \text{ cm}^2$

$d = 46.5 \text{ cm.}$

$t_w = 1.34 \text{ cm.}$

$\frac{d}{A_f} = 0.9 \text{ cm.}^{-1}$

$I_x = 59517 \text{ cm}^4$

$S_x = 2558 \text{ cm}^3$

$r_x = 19.23 \text{ cm.}$

$r_y = 5.01 \text{ cm.}$



$$f_{bx} = \frac{35.12 \times 10^5}{2558} = 1373 \text{ Kg/cm}^2$$

$$l \frac{d}{A_f} = 134 \times 0.9 = 121 < 555 \Rightarrow F_{bx} = 1520 \text{ Kg/cm}^2$$

$$\frac{f_{bx}}{F_{bx}} = 0.9 < 1.0 \text{ es correcto}$$

### CORTANTE:

$$f_{\text{VACT}} = \frac{20740}{46.5 \times 1.34} = 333 \text{ Kg/cm}^2 < f_{\text{RESIS.}} = 1012 \text{ Kg/cm}^2 \text{ es } \\ = \text{correcto}$$

### DEFLEXION:

$$S_{\text{MAX}} = 0.22 < 1.67 \text{ es correcto}$$

$$S_{\text{PERM.}} = \frac{l}{360} = \frac{600}{360} = 1.67 \text{ cms.}$$

USAR I.P.R. 18" x 8 3/4" x 126.7 lb/analysis definitivo

TRABE EJE (2) NIVEL 7/5+3

### ELEMENTOS MECANICOS MAXIMOS

$$V = 1000(7.62 + 9)0.75 = 124 \text{ Kg.}$$

$$M = 25.87 \times 10^5 \text{ Kg-cm.}$$

$$L = 600 \text{ cms.}$$

$$L_b = 180 \text{ cms.}$$

Si probamos con una I.P.R. 18" x 8 3/4" x 95.4

### PROPIEDADES:

$$A = 126.29 \text{ cm}^2$$

$$I_x = 43529 \text{ cm}^4$$

$$d = 45.4 \text{ cm}$$

$$S_x = 1917 \text{ cm}^3$$

$$t_w = 1.02 \text{ cm}$$

$$r_x = 18.95 \text{ cm}$$

$$\frac{d}{A_f} = 1.18 \text{ cm}^{-1}$$

$$r_y = 4.19 \text{ cm}$$

$$f_{bx} = \frac{25.87 \times 10^5}{1917} = 1350 \text{ Kg/cm}^2$$

$$l \frac{d}{A_f} = 180 \times 1.18 = 212 < 555 \Rightarrow F_{bx} = 1520 \text{ Kg/cm}^2$$

$$\frac{f_{bx}}{F_{bx}} = 0.88 < 1.0 \text{ es correcto}$$

### CORTANTE:

$$f_{\text{VACT}} = \frac{12470}{45.4 \times 1.02} = 269 \text{ Kg/cm}^2 < 1012 \text{ Kg/cm}^2 \text{ es correcto}$$

### DEFLEXION:

$$\delta_{\text{MAX}} = 0.56 < \delta_{\text{PERM.}} \text{ es correcto}$$

∴ Usar I.P.E 18" x 8<sup>3</sup>/<sub>4</sub>" x 95.4 para análisis definitivo

TRABE EJE (2) NIVEL T/5+4

### ELEMENTOS MECANICOS

#### MAXIMOS

$$V = (10430 + 6880) \cdot 0.75 = 12,980 \text{ Kg}$$

$$M = 23.59 \times 10^5 \text{ Kg-cm.}$$

$$L = 600 \text{ cm.}$$

$$L_b = 200 \text{ cm.}$$

si probamos con una I.P.E 18" x 8<sup>3</sup>/<sub>4</sub>" x 95.4

#### PROPIEDADES:

$$A = 121.29 \text{ cm}^2$$

$$d = 45.4 \text{ cm.}$$

$$t_w = 1.02 \text{ cm.}$$

$$d_f = 1.18 \text{ cm.}$$

$$A_f$$

$$I_x = 43529 \text{ cm}^4$$

$$S_x = 1917 \text{ cm}^3$$

$$r_x = 18.95 \text{ cm.}$$

$$r_y = 4.9 \text{ cm.}$$

$$f_{bx} = \frac{23.59 \times 10^5}{1917} = 1231 \text{ Kg/cm}^2$$

$$\frac{L_d}{A_f} = 200 \times 1.18 = 236 < 555 \implies F_{bx} = 1520 \text{ Kg/cm}^2$$

$$\frac{f_{bx}}{F_{bx}} = 0.81 < 1.0 \text{ es correcto}$$

### CORTANTE:

$$f_{\text{VACT}} = \frac{12980}{45.4 \times 1.02} = 280 \text{ Kg/cm}^2 < 1012 \text{ Kg/cm}^2 \text{ es correcto}$$

### DEFLEXION:

$$\delta_{\text{MAX}} = 0.18 < 1.67 \text{ cm es correcto}$$

Usar I.P.E 18" x 8<sup>3</sup>/<sub>4</sub>" x 95.4 para análisis definitivo

## TRABE EJE (2) NIVEL T/S+5

### ELEMENTOS MECANICOS

#### MAXIMOS

$$V = (15.41 + 3.99) 1000 \times 0.75 = 14,550 \text{ Kg.}$$

$$M = 20.3 \times 10^5 \text{ Kg-cm.}$$

$$L = 600 \text{ cm.}$$

$$L_b = 134 \text{ cm.}$$

si probamos con una I.P.R. 18" x 8 3/4" x 95.4

#### PROPIEDADES:

$$A = 121.29 \text{ cm}^2$$

$$I_x = 43529 \text{ cm}^4$$

$$d = 45.4 \text{ cm.}$$

$$S_x = 1917 \text{ cm}^3$$

$$t_w = 1.02 \text{ cm.}$$

$$r_x = 18.95 \text{ cm.}$$

$$\frac{d}{A_f} = 1.18 \text{ cm.}^{-1}$$

$$r_y = 4.9 \text{ cm.}$$

$$f_{bx} = \frac{20.3 \times 10^5}{1917} = 1059 \text{ Kg/cm}^2$$

$$l \frac{d}{A_f} = 154 \times 1.18 = 182 < 555 \implies F_{bx} = 1520 \text{ Kg/cm}^2$$

$$\frac{f_{bx}}{F_{bx}} = 0.7 < 1.0 \text{ as correcto}$$

#### CORTANTE:

$$f_{vACT} = \frac{14.55 \times 10^3}{45.4 \times 1.02} = 314 \text{ Kg/cm}^2 < 1012 \text{ Kg/cm}^2 \text{ as. correcto}$$

#### DEFLEXION:

$$\delta_{PERM} = 1.67$$

$$\delta_{MAX.} = 0.22 < 1.67 \text{ cm.}$$

\(\therefore\) usar para análisis definitivo I.P.R. 18" x 8 3/4" x 95.4

## TRABE EJE (2) NIVEL T/S+0

### ELEMENTOS MECANICOS

#### MAXIMOS

$$V = (4.94 + 1.1) 1000 \times 0.75 = 4530 \text{ Kg.}$$

$$M = 5.93 \text{ T-M}$$

$$L = 600 \text{ cm.}$$

$$L_b = 285$$

probamos con una I.P.E.  $10 \times 5^{3/4} \times 37.3$

#### PROPIEDADES

$$A = 47.42 \text{ cm}^2$$

$$d = 25.6 \text{ cm.}$$

$$t_w = 0.64 \text{ cm.}$$

$$\frac{d}{A_f} = 1.61 \text{ cm}^{-1}$$

$$S_x = 432 \text{ cm}^3$$

$$r_x = 10.82 \text{ cm.}$$

$$r_y = 3.32 \text{ cm.}$$

$$I_x = 5544 \text{ cm}^4$$

$$E = 2 \times 10^6 \text{ Kg/cm}^2$$

$$f_{bx} = \frac{M}{S_x} = \frac{5.93 \times 10^3}{432} = 1373 \text{ Kg/cm}^2$$

$$f_b \frac{d}{A_f} = 285 \times 1.61 = 459 < 555 \implies f_{bx} = 1520 \text{ Kg/cm}^2$$

$$\frac{f_{bx}}{F_{bx}} = \frac{1373}{1520} = 0.9 < 1.0 \text{ es correcto}$$

#### CORTANTE:

$$f_{\text{ACT.}} = \frac{4.53 \times 10^3}{25.6 \times 0.64} = 277 \text{ Kg/cm}^2 < f_{\text{PERM.}} = 0.4 F_y = 1012 \text{ Kg/cm}^2$$

es correcto

#### DEFLEXION:

$$d_{\text{MAX.}} = 0.70 \text{ cm.} < 1.67 \text{ es correcto}$$

$$d_{\text{PERM.}} = \frac{l}{360} = \frac{600}{360} = 1.67$$

... USAR I.P.E.  $10 \times 5^{3/4} \times 37.3$  para análisis definitivo  
aumentar el peso propio de las trabas

# DISEÑO PRELIMINAR TRAPES LONGITUDINALES PRINCIPALES

EJE (0) NIVEL T/S+1 ÷ EJES (3) y (4)

## ELEMENTOS MECANICOS

### MAXIMOS

C.M + C.V	C.M + C.V + SISMO
M = 1.19 T-M	M = 1.19 T-M
	C = 22.64 TON.

$$L_x = 500 \text{ cm.}$$

$$L_y = 500 \text{ cm.}$$

si probamos con una I.P.E 10" x 5 3/4" x 37.3

### PROPIEDADES:

$$A = 47.42 \text{ cm}^2 \quad S_x = 432 \text{ cm}^3$$
$$\frac{d}{A} = 1.61 \text{ cm}^{-1} \quad r_x = 10.82 \text{ cm.}$$
$$r_y = 3.32 \text{ cm.}$$

$$f_a = \frac{22.64 \times 10^3}{47.42} = 477 \text{ Kg/cm}^2$$

$$\frac{K_x l_x}{r_x} = \frac{1.0 \times 500}{10.82} = 46 \implies F'_{ax} = 4962 \text{ Kg/cm}^2$$

$$\frac{K_y l_y}{r_y} = \frac{1.0 \times 500}{3.32} = 151 \implies F_a = 467 \text{ Kg/cm.}$$

$$\frac{f_a}{F_a} = 1.02 > 0.15$$

$$f_{bx} = \frac{1.19 \times 10^5}{432} = 275 \text{ Kg/cm}^2$$

$$\frac{l d}{A r} = 500 \times 1.61 = 805 \implies F_{bx} = \frac{843700}{805} = 1048 \text{ Kg/cm}^2$$

$$\left(1 - \frac{f_a}{F'_{ax}}\right) F_{bx} = 947 \text{ Kg/cm}^2; \quad \frac{f_{bx}}{\left(1 - \frac{f_a}{F'_{ax}}\right) F_{bx}} = 0.29$$

$$1.02 + 0.29 = 1.31 < 1.33 \text{ es correcto}$$

∴ usar I.P.E 10" x 5 3/4" x 37.3 P/ análisis definitivo

# TRABES LONGITUDINALES T/S-2

EJE ① ÷ EJES ② y ③

C.M+ C.V

$$M = 4.2 \text{ T-M}$$

$$l_x = 500 \text{ cm.}$$

$$l_y = 250 \text{ cm.} = lb$$

C.M+ C.V+ SISMO

$$C = 11.91 \text{ TON}$$

$$M = 4.2 \text{ T-M}$$

si probamos con una I.P.R  $10'' \times 5\frac{3}{4}'' \times 31.3$

PROPIEDADES:

$$A = 39.93 \text{ cm}^2$$

$$\frac{d}{A} = 1.99 \text{ cm}^{-1}$$

$$\frac{K_x l_x}{r_x} = \frac{1.0 \times 500}{10.52} = 48 \implies F'_{bx} = 4557 \text{ Kg/cm}^2$$

$$\frac{K_y l_y}{r_y} = \frac{1.0 \times 250}{3.17} = 79 \implies F_a = 1090 \text{ Kg/cm}^2$$

$$\frac{f_a}{F_a} = 0.27 > 0.15$$

$$lb \frac{d}{A} = 250 \times 1.99 = 498 < 555 \implies F_{bx} = 1520 \text{ Kg/cm}^2$$

$$\left( \frac{1 - f_a}{F'_{bx}} \right) F_{bx} = 1421 \text{ Kg/cm}^2$$

$$f_{bx} = \frac{4.2 \times 10^5}{352} = 1193 \text{ Kg/cm}^2$$

$$\frac{f_{bx}}{\left( \frac{1 - f_a}{F_{bx}} \right)} = 0.84$$

$$0.27 + 0.84 = 1.11 < 1.33 \text{ es correcto} \quad \eta = 83\%$$

## TRABES LONGITUDINALES T/G+3

EJE (0) ÷ EJES (3) y (4)

C.M+ C.V

$$M = 3.02 \text{ T-M}$$

C.M+ C.V+ SISMO

$$M = 3.02 \text{ T-M}$$

$$C = 7.67 \text{ TON.}$$

$$l_x = 500 \text{ cm.}$$

$$l_y = l_b = 100 \text{ cm.}$$

si probamos con una I.P.E 8"x5'4"x29.8

PROPIEDADES:

$$A = 32.26 \text{ cm}^2$$

$$\frac{d}{A_f} = 1.96 \text{ cm}^{-1}$$

$$S_x = 231 \text{ cm}^3$$

$$r_x = 8.53 \text{ cm.}$$

$$r_y = 2.94 \text{ cm.}$$

$$f_a = 202 \text{ Kg/cm}^2$$

$$\frac{K_x l_x}{r_x} = \frac{1.0 \times 500}{8.53} = 59 \implies \begin{cases} F_a = 1235 \text{ Kg/cm}^2 \\ F_{lx} = 3016 \text{ Kg/cm}^2 \end{cases}$$

$$\frac{K_y l_y}{r_y} = \frac{1.0 \times 100}{2.94} = 34$$

$$\frac{f_a}{F_a} = 0.19 > 0.15$$

$$f_{bx} = \frac{3.02 \times 10^5}{231} = 1307 \text{ Kg/cm}^2$$

$$\frac{l d}{A_f} = 100 \times 1.96 = 196 < 555 \implies F_{bx} = 1520 \text{ Kg/cm}^2$$

$$\left(1 - \frac{f_a}{F_{lx}}\right) F_{bx} = 1400 \text{ Kg/cm}^2$$

$$\left(\frac{f_{bx}}{1 - f_a}\right) F_{bx} = 0.93$$

$$0.93 + 0.19 = 1.12 < 1.33 \text{ es correcto } \eta = 84\%$$

# TRABES LONGITUDINALES NIVEL T/S + 4

EJE (0) ÷ EJES (1) y (2)

$$CM + CV$$

$$M = 5.65 \text{ T-M}$$

$$C = 8.8 \text{ TON.}$$

$$M = 5.65 \text{ T-M}$$

$$l_x = 500 \text{ cm.}$$

$$l_y = l_b = 150 \text{ cm.}$$

EJE (0) ÷ EJES (3) y (4)

$$CM + CV$$

$$M = 4.38 \text{ T-M}$$

$$CM + CV + SISMO$$

$$C = 10.67 \text{ TON}$$

$$M = 4.38 \text{ T-M}$$

$$l_x = 500 \text{ cm.}$$

$$l_y = l_b = 223 \text{ cm.}$$

si probamos con una I.P.E 10' x 5 3/4" x 31.3

PROPIEDADES:

$$A = 39.93 \text{ cm}^2$$

$$\frac{d}{A_f} = 1.99 \text{ cm}^{-1}$$

$$S_x = 352 \text{ cm}^3$$

$$r_x = 10.52 \text{ cm.}$$

$$r_y = 3.17 \text{ cm.}$$

$$f_a = \frac{8800}{39.93} = 220 \text{ Kg/cm}^2$$

$$\frac{K_x l_x}{r_x} = \frac{1.0 \times 500}{10.52} = 48 \implies F_a = 1305 \text{ Kg/cm}^2$$

$$\frac{K_y l_y}{r_y} = \frac{1.0 \times 150}{3.17} = 47 \implies F_{ex} = 4557 \text{ Kg/cm}^2$$

$$\frac{f_a}{F_a} = 0.17 > 0.15$$

$$f_{bx} = \frac{5.65 \times 10^3}{352} = 1605 \text{ Kg/cm}^2$$

$$\frac{l_b d}{A_f} = 150 \times 1.99 = 299 < 555 \implies F_{bx} = 1520 \text{ Kg/cm}^2$$

$$\left(1 - \frac{f_a}{F_{ex}}\right) F_{bx} = 1447 \text{ Kg/cm}^2$$

$$\frac{F_{bx}}{\left(1 - \frac{f_a}{F_{ex}}\right) F_{bx}} = 1.11$$

$$0.17 + 1.11 = 1.28 < 1.33 \text{ es correcto } \eta = 96\%$$

$$f_a = \frac{10670}{39.93} = 267 \text{ Kg/cm}^2$$

$$\frac{K_x l_x}{r_x} = \frac{1.0 \times 500}{10.52} = 48 \implies F'_{ex} = 4557 \text{ Kg/cm}^2$$

$$\frac{K_y l_y}{r_y} = \frac{1.0 \times 233}{3.17} = 70 \implies F_a = 1157 \text{ Kg/cm}^2$$



$$\frac{f_a}{F_a} = 0.23 > 0.15$$

$$f_{bx} = \frac{4.38 \times 10^5}{352} = 1244 \text{ Kg/cm}^2$$

$$\lambda \frac{d}{A_f} = 223 \times 1.99 = 444 < 555 \implies F_{bx} = 1520 \text{ Kg/cm}^2$$

$$\left( \frac{1-f_a}{F'_{bx}} \right) F_{bx} = 1431 \text{ Kg/cm}^2$$

$$\left( \frac{f_{bx}}{1-f_a} \right) F_{bx} = 0.87$$

$0.23 + 0.87 = 1.10 < 1.33$  es correcto  $\eta = 83\%$   
 $\therefore$  USAR I.P.E 10" x 5 3/4" x 31.3  $\neq$  análisis definitivo

### TRABES LONGITUDINALES NIVEL T/S+5

EJE ①  $\div$  EJES ② y ③

CM+CV

$$M_{max} = 3.47 \text{ T-M}$$

$$l_x = 500 \text{ cm.}$$

$$l_y = l_b = 100 \text{ cm.}$$

CM+CV+SISMO

$$C = 12.18 \text{ TON}$$

$$M = 3.47 \text{ TON-M}$$

si probamos con una I.P.E 8" x 5 1/4" x 25.3

PROPIEDADES:

$$A = 32.26 \text{ cm}^2$$

$$\frac{d}{A_f} = 1.96 \text{ cm}^{-1}$$

$$S_x = 231 \text{ cm}^3$$

$$r_x = 8.53 \text{ cm.}$$

$$r_y = 2.94 \text{ cm.}$$

$$f_a = \frac{12180}{32.26} = 378 \text{ Kg/cm}^2$$

$$\frac{K_x l_x}{r_x} = \frac{1.0 \times 500}{8.53} = 59 \implies F_a = 1235 \text{ Kg/cm}^2$$

$$\frac{K_y l_y}{r_y} = \frac{1.0 \times 100}{2.94} = 34$$

$$F'_{bx} = 3016 \text{ Kg/cm}^2$$

$$\frac{f_a}{F_a} = 0.31 > 0.15$$

## DISEÑO PRELIMINAR CONTRAVENTEO VERTICAL

NIVELES T/S+2 Y T/S+3

$$L = 472 \text{ cm}$$

$$C = 10.68 \text{ TON}$$

$$\frac{KL}{r_{\min}} = 200$$

$$K = 1$$

$$r_{\min} = \frac{1 \times 472}{200} = 2.36 \text{ cm}$$

SI USAMOS 2L<sup>s</sup> JL 3x3x1/4

$$A = 18.58 \text{ cm}^2$$

$$r_x = r_y = 2.36 \text{ cm}$$

$$r_z = 1.49 \text{ cm}$$

$$\frac{KL}{r} = \frac{1 \times 472}{2.36} = 200 \Rightarrow F_a = 263 \text{ kg/cm}^2$$

$$C_{\text{RESIST}} = 263 \times 18.58 = 4887 \text{ kg} < 10,680 \text{ kg}$$

2L<sup>s</sup> JL 4x4x1/4 ( $w = 19.64 \text{ KGS/M}$ )

$$\frac{KL}{r_y} = \frac{1 \times 472}{3.13} = 148 \Rightarrow F_a = 479 \text{ kg/cm}^2$$

$$C_{\text{RESIST}} = 479 \times 2 \times 12.52 = 11,994 > 10680 \quad \text{CORRECTO}$$

SI PROBAMOS CON UNA T.P.R 5x5<sup>3/4</sup>x18.65

$$A = 23.71 \text{ cm}^2$$

$$r_y = 3.32 \text{ cm}$$

$$\frac{KL}{r} = \frac{1 \times 472}{3.32} = 142 \Rightarrow F_a = 521 \text{ kg/cm}^2$$

$$C_{\text{RESIST}} = 521 \times 23.71 \times 1.33 = 16,429 \text{ kg} > 10680 \text{ kg} \quad \text{ES CORRECTO}$$

USAR T.P.R 5x5<sup>3/4</sup>x18.65 P/ANALISIS DEFINITIVO.

NIVELES T/S+4, T/S+5 Y T/S+6

$L = 472 \text{ cm}$   
 $C = 7.91 \text{ TON.}$

SI PROBAMOS CON UNA T.P.R.  $5 \times 8^{3/4} \times 15.65$

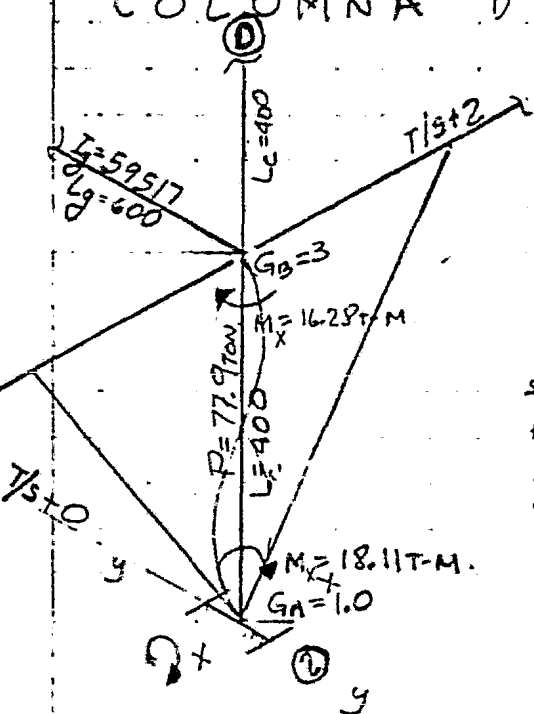
$A = 19.96 \text{ cm}^2$   
 $r_y = 3.17 \text{ cm}$

$\frac{L}{r} = \frac{1 \times 472}{3.17} = 149 \Rightarrow F_a = 473 \text{ kg/cm}^2$

$C_{RESIS} = 473 \times 19.96 = 7441 \text{ kg} > 7910 \text{ kg.}$

## DISEÑO PRELIMINAR DE COLUMNAS.

COLUMNA DEL T/S+0 AL T/S+2 Y DEL T/S+2 AL T/S+3



$\left. \begin{aligned} & \text{I.P.R } 18 \times 8^{3/4} \times 126.7 \\ & I_x = 59517 \text{ cm}^4 \\ & L_g = 600 \text{ cm} \end{aligned} \right\}$

SI SUPONEMOS COLUMNAS DE IPR  $18 \times 8^{3/4} \times 126.7$

$A = 161.09 \text{ cm}^2$   
 $d = 0.9 \text{ cm}^{-1}$   
 $A_f$   
 $I_x = 59517 \text{ cm}^4$   
 $S_x = 2558 \text{ cm}^3$   
 $r_x = 19.23 \text{ cm}$   
 $r_y = 5.01 \text{ cm}$

PROPIEDADES

$l_x = 400 \text{ cm.}$

$l_y = 400 \text{ cm}$

CALCULO DE  $K_x$  Y  $K_y$

$K_y = 1.0$

CALCULO DE "Kx"

$$G_B = \frac{\sum \frac{I_c}{L_c}}{\sum \frac{I_g}{L_g}} = \frac{\frac{59517}{400} + \frac{59517}{400}}{\frac{59517}{600}} = 3$$

$G_A = 1.0$

$K_x = 1.57$

$$f_a = \frac{P}{A} = \frac{77.9 \times 10^3}{161.09} = 484 \text{ Kg/cm}^2$$

$$\frac{K_x l_x}{r_x} = \frac{1.57 \times 400}{19.23} = 33 \Rightarrow F'_{ox} = 9628 \text{ Kg/cm}^2$$

$$\frac{K_y l_y}{r_y} = \frac{1.0 \times 400}{5.01} = 80 \Rightarrow F_a = 1082 \text{ Kg/cm}^2$$

$$\frac{f_a}{F_a} = \frac{484}{1082} = 0.45 > 0.15$$

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F'_{ox}}\right) F_{bx}} \leq 1.0$$

$$\frac{f_a}{0.6 F_y} + \frac{f_{bx}}{F_{bx}} \leq 1.0$$

$$C_{mx} = 0.85$$

$$f_{bx} = \frac{18.11 \times 10^5}{2558} = 708 \text{ Kg/cm}^2$$

$$\frac{l_d}{A_f} = 400 \times 0.9 = 360 < 555 \Rightarrow F_{bx} = 1520 \text{ Kg/cm}^2$$

$$\left(1 - \frac{f_a}{F'_{ox}}\right) F_{bx} = 1444 \text{ Kg/cm}^2$$

$$\frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F'_{ox}}\right) F_{bx}} = 0.42$$

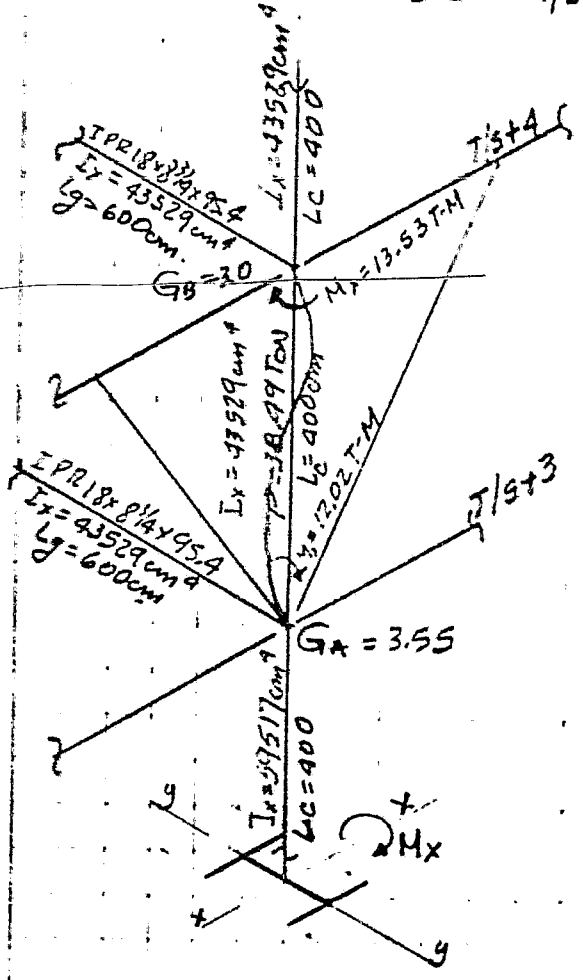
$$0.45 + 0.42 = 0.87 < 1.0 \text{ ES CORRECTO}$$

ADICIONAMENTE

$$\frac{484}{0.6 \times 2530} + \frac{708}{1520} = 0.32 + 0.47 = 0.79 < 1.0 \text{ ES CORRECTO}$$

USAR I.P.R.  $18 \times 8^{3/4} \times 126.7$  P/ANALISIS DEFINITIVO.

COLUMNAS DEL T/S+3 AL T/S+4 Y DEL T/S+4 AL T/S+5



SI PROBAMOS CON UNA I PR 18x8<sup>1</sup>/<sub>4</sub>x95.4

PROPIEDADES:

$A = 121.29 \text{ cm}^2$

$\bar{x} = 1.18 \text{ cm}$

$A_4$

$S_x = 1917 \text{ cm}^3$

$I_x = 43529 \text{ cm}^4$

$r_x = 18.95 \text{ cm}$

$r_y = 4.9 \text{ cm}$

$l_x = 400 \text{ cm}$

$l_y = 400 \text{ cm}$

$f_a = \frac{38.49 \times 10^3}{121.29} = 317 \text{ kg/cm}^2$

$K_y = 1.0$

CALCULO DE "K<sub>x</sub>"

$G_A = \frac{\frac{59517}{400} + \frac{43529}{400}}{\frac{43529}{600}} = 3.55$

$G_B = \frac{\frac{43529}{400} + \frac{43529}{400}}{\frac{43529}{600}} = 3.0$

$K_x = 1.9$

CALCULO DE F<sub>a</sub>

$\frac{K_x l_x}{r_x} = \frac{1.9 \times 400}{18.95} = 40 \Rightarrow F_{ax} = 6553 \text{ kg/cm}^2$

$\frac{K_y l_y}{r_y} = \frac{1.0 \times 400}{4.9} = 82 \Rightarrow F_a = 1066 \text{ kg/cm}^2$

$\frac{f_a}{F_a} = 0.30 > 0.15$

$\frac{f_a}{F_a} + \frac{C_{m_x} f_{b_x}}{\left(1 - \frac{f_a}{F_{ax}}\right) F_{b_x}} \leq 1.0$

$\frac{f_a}{0.3 F_y} + \frac{f_{b_x}}{F_{b_x}} \leq 1.0$

$C_{m_x} = 0.85$

$$f_{bx} = \frac{13.53 \times 10^5}{1917} = 706 \text{ kg/cm}^2$$

$$l \frac{d}{A_f} = 400 \times 1.18 = 472 < 555 \Rightarrow F_{bx} = 1520 \text{ kg/cm}^2$$

$$\left(1 - \frac{f_a}{F_{ex}}\right) F_{bx} = 1446 \text{ kg/cm}^2$$

$$\frac{C_{m_x} f_{bx}}{\left(1 - \frac{f_a}{F_{ex}}\right) F_{bx}} = 0.41$$

$$\left(1 - \frac{f_a}{F_{ex}}\right) F_{bx}$$

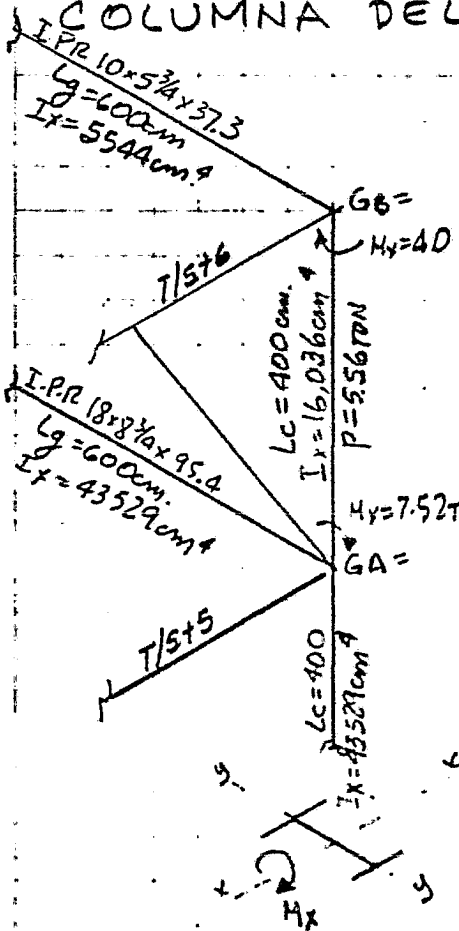
$$0.3 + 0.41 = 0.71 < 1.0 \text{ ES CORRECTO}$$

ADICIONALMENTE

$$\frac{317}{1520} + \frac{706}{1520} = 0.67 < 1.0 \text{ ES CORRECTO}$$

∴ USAR I.P.R. 18 × 8 3/4 × 95.4 P/ANÁLISIS DEFINITIVO.

COLUMNA DEL T/S+5 AL T/S+6



SI PROBAMOS CON UNA I.P.R. 14 × 6 3/4 × 56.6

PROPIEDADES:

$$A = 72.06 \text{ cm}^2$$

$$\frac{d}{A_f} = 1.61 \text{ cm}^{-1}$$

$$S_x = 895 \text{ cm}^3$$

$$r_x = 14.9 \text{ cm}$$

$$r_y = 3.78 \text{ cm}$$

$$l_x = l_y = 400 \text{ cm}$$

$$f_a = \frac{5560}{72.06} = 77 \text{ kg/cm}^2$$

$$K_y = 1.0$$

CALCULO DE "K<sub>x</sub>"

$$G_A = \frac{43529}{400} + \frac{16036}{400} = 2.0$$

$$G_B = \frac{16036}{400} + \frac{5544}{600} = 4.3$$

$$K_x = 1.85$$

$$\frac{K_x l_x}{\Gamma_x} = \frac{1.85 \times 400}{14.9} = 50$$

$$\frac{K_y l_y}{\Gamma_y} = \frac{1.0 \times 400}{3.78} = 106 \Rightarrow F_a = 860 \text{ kg/cm}^2$$

$$\frac{f_a}{F_a} = 0.09 < 0.15$$

$$f_{bx} = \frac{7.52 \times 10^5}{895} = 840 \text{ kg/cm}^2$$

$$\lambda \frac{d}{A_f} = 400 \times 1.61 = 644 > 555 \Rightarrow F_{bx} = 1310 \text{ kg/cm}^2$$

$$\frac{f_{bx}}{F_{bx}} = 0.64$$

$$0.09 + 0.64 = 0.73 < 1.0 \text{ ES CORRECTO}$$

SI PROBAMOS CON UNA I.P.R. 14 x 6<sup>3/4</sup> x 50.7

PROPIEDADES:

$$A = 64.52 \text{ cm}^2$$

$$\frac{d}{A_f} = 1.81 \text{ cm}^{-1}$$

$A_f$

$$I_x = 14117 \text{ cm}^4$$

$$S_x = 795 \text{ cm}$$

$$\Gamma_x = 14.8 \text{ cm}$$

$$\Gamma_y = 3.7 \text{ cm}$$

$$K_y = 1.0$$

CALCULO DE "K<sub>x</sub>"

$$G_A = \frac{\frac{43529}{400} + \frac{14117}{400}}{\frac{43529}{600}} = 2.0$$

$$G_B = \frac{\frac{14117}{400}}{\frac{5544}{600}} = 3.8$$

$$K_x = 1.8$$

$$\frac{K_x l_x}{\Gamma_x} = \frac{1.8 \times 400}{14.8} = 49$$

$$\frac{K_y l_y}{\Gamma_y} = \frac{1.0 \times 400}{3.7} = 108 \Rightarrow F_a = 841 \text{ kg/cm}^2$$

$$\frac{f_a}{F_a} = 0.10 < 0.15$$

$$f_{bx} = \frac{7.52 \times 10^5}{795} = 946 \text{ kg/cm}^2$$

$$\lambda \frac{d}{A_f} = 400 \times 1.81 = 724 > 555 \Rightarrow F_{bx} = 165 \text{ kg/cm}^2$$

$$\frac{f_{bx}}{F_{bx}} = 0.81$$

$$0.10 + 0.81 = 0.91 < 1.0$$

ES CORRECTO

USAR I.P.R. 14 x 6<sup>3/4</sup> x 50.7

P/ANÁLISIS DEFINITIVO

## CAPITULO 3

### ANALISIS ESTRUCTURAL

Una vez obtenidas las secciones preliminares, se procedió al análisis de finitivo.

Para la obtención de las rigideces de piso se realizó una corrida preliminar aplicando una fuerza unitaria en cada nivel la cual nos provoca -- desplazamientos lineales, la rigidez de entrepiso se obtiene dividiendo el cortante entre el desplazamiento relativo, es decir  $K = \frac{V}{\Delta}$

El desplazamiento relativo es la diferencia entre los desplazamientos superior e inferior de un entepiso.

#### Torsión

Una vez obtenidas las rigideces de entre piso, se determinaron los centros de torsión en todos los niveles de la estructura y se calcularon -- las excentricidades torsionales, es decir, la distancia entre el centro de torsión y el punto de aplicación del cortante en el nivel (medida perpendicular a la dirección del cortante).

Dichas excentricidades se aumentaron en un 50% y se les sumo una cantidad igual a 0.10 de la mayor dimensión del nivel analizado, medida normal al sentido del cortante. Así se llegó a obtener los momentos torsionantes en los distintos niveles, los cuales producen incrementos o decrementos en el cortante directo de los marcos, de hecho, los cortantes correspondientes a un marco cualquiera, en un nivel determinado fueron:

$$V_{ai} = V_{dai} + \frac{M_{ti} K_{ai} d}{(\sum K_{ixy}^2 + \sum K_{iyx}^2)}$$



En la cual:

$V_{ai}$  = corte total en el marco a del nivel i.

$V_{dai}$  = corte directo en el marco a del nivel i =  $\frac{V_{ai}}{\sum K_{ai}} V_i$

$M_{ti}$  = Momento de torsión en el nivel i =  $\frac{V_i (1.5e + 0.1 D)}{V_i (e - 0.1D)}$

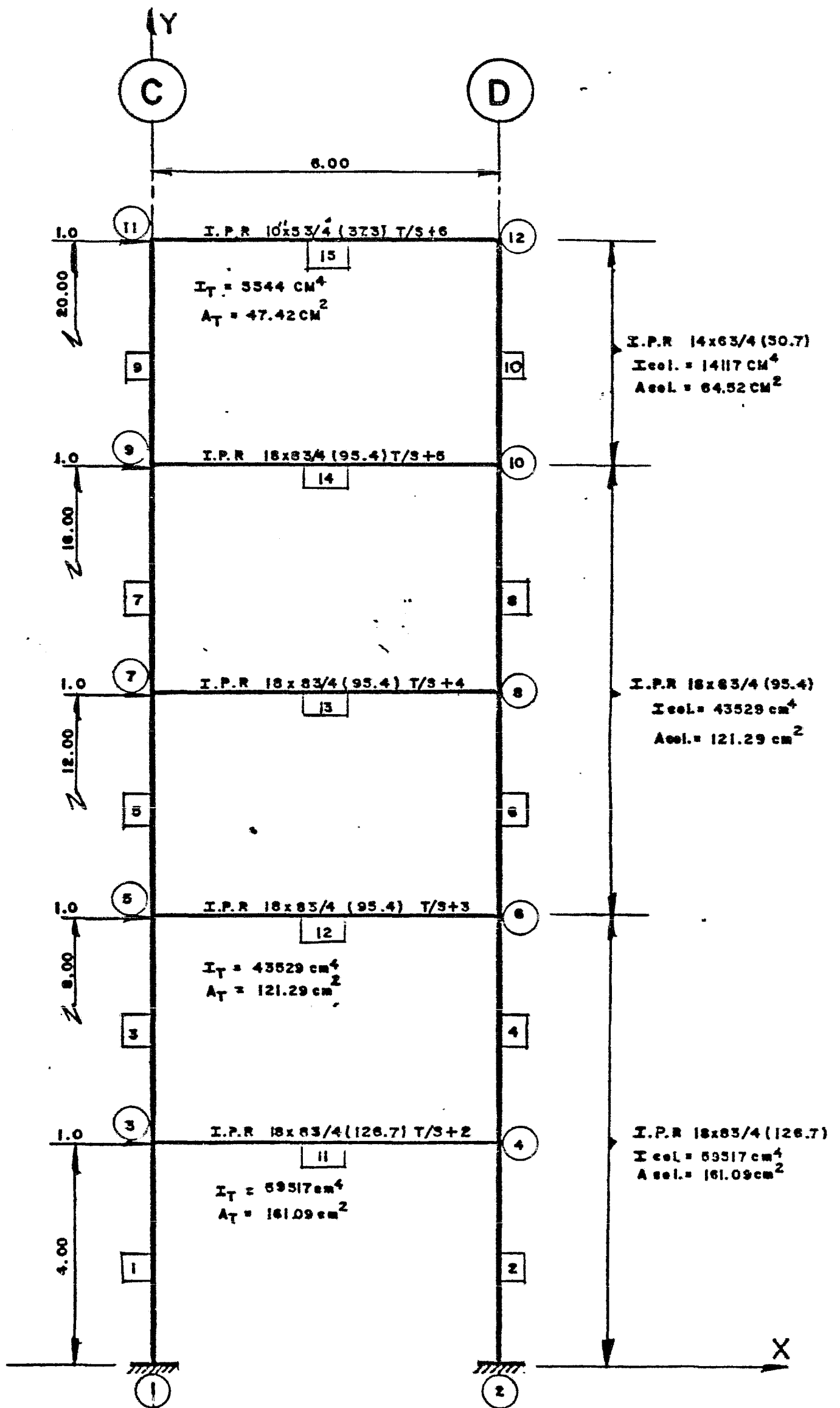
$K_{ai}$  = Rigidez del marco a en el nivel i.

$d$  = Distancia del marco a al centro de torsión.

$(\sum K_{ixy}^2 + \sum K_{iyx}^2)$  = Suma de los productos de las rigideces de los marcos por el cuadrado de sus distancias al centro de torsión.

Una vez conocidos los cortantes en cada nivel de cada marco, se procedió a determinar los elementos mecánicos en las distintas secciones. Ya con estos datos se realizaron las corridas de los marcos por computadora - tanto para las cargas gravitacionales como para las accidentales. De la misma forma que para el análisis preliminar se hicieron las combinaciones de carga para determinar las condiciones más desfavorables para cada una de las secciones de los marcos.

# MARCO EJE ① y ②



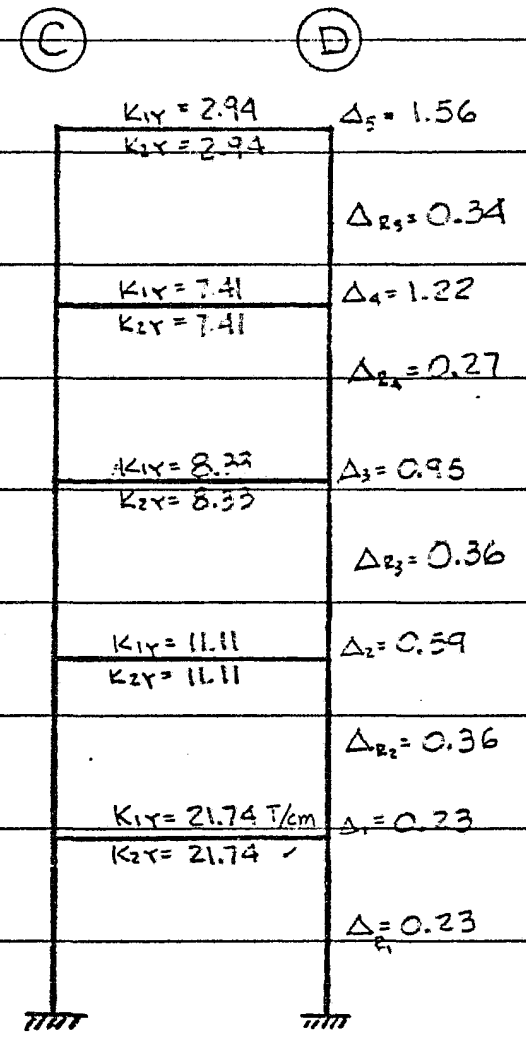
LISTADO DEL ARCHIVO DE DATOS : MARCO EJ  
ES 1 Y 2

6704944-8

28-10-83

JESUS PONCE S

12	15	1	2100			
1	0	0	0	0	0	
2	0	0	0	6	0	
3	1	1	1	0	4	
4	1	1	1	6	4	
5	1	1	1	0	8	
6	1	1	1	6	8	
7	1	1	1	0	12	
8	1	1	1	6	12	
9	1	1	1	0	16	
10	1	1	1	6	16	
11	1	1	1	0	20	
12	1	1	1	6	20	
1	1	3	1	1	161.09	59517
2	2	4	1	1	161.09	59517
3	3	5	1	1	161.09	59517
4	4	6	1	1	161.09	59517
5	5	7	1	1	121.29	43529
6	6	8	1	1	121.29	43529
7	7	9	1	1	121.29	43529
8	8	10	1	1	121.29	43529
9	9	11	1	1	64.52	14117
10	10	12	1	1	64.52	14117
11	3	4	1	1	161.09	59517
12	5	6	1	1	121.29	43529
13	7	8	1	1	121.29	43529
14	9	10	1	1	121.29	43529
15	11	12	1	1	47.42	5544



1  
SISMO  
5 0  
3 1 0 0  
5 1 0 0  
7 1 0 0  
9 1 0 0  
11 1 0 0

6704944-8  
28-10-83  
PROBLEMA DE: JESUS PONCE S

NUMERO DE NODOS : 12  
NUMERO DE MIEMBROS : 15  
NUMERO DE CARGAS : 1  
MODULO DE ELASTICIDAD, MTONS/CM2: 2100

NODO	XCOORD	YCOORD	XREL	YREL	MZREL
1	0.00	0.00000			
2	3.00	0.00000			
3	0.00	4.00111			
4	6.00	4.00111			
5	0.00	8.00111			

6	5.00	3.00111
7	0.00	12.00111
8	5.00	12.00111
9	0.00	16.00111
10	5.00	12.00111
11	0.00	20.00111
12	5.00	20.00111

MIEMBRO LU.JT HI.JT LOW.FIX HI.FIX (CM)2

(CM)4	L.METER			
1	1	311	161.09	59517.00
4.00				
2	2	411	161.09	59517.00
4.00				
3	3	511	161.09	59517.00
4.00				
4	4	611	161.09	59517.00
4.00				
5	5	711	121.29	43529.00
4.00				
6	6	811	121.29	43529.00
4.00				
7	7	911	121.29	43529.00
4.00				
8	8	1011	121.29	43529.00
4.00				
9	9	1111	64.52	14117.00
4.00				
10	10	1211	64.52	14117.00
4.00				
11	3	411	161.09	59517.00
6.00				
12	5	611	121.29	43529.00
6.00				
13	7	811	121.29	43529.00
6.00				
14	9	1011	121.29	43529.00
6.00				
15	11	1211	47.42	5544.00
6.00				

GLOBAL STIFF MATRIX ROWS=31 COLS=9 ELEME

NTS=279

SISMO

NODO NO.	X=DESPL CMETER	Y=DESPL CM
ETER	ROTACION GRADO	
1	0.0000	0.00
00	0.0000	
2	0.0000	0.00
00	0.0000	
3	0.2319	0.00
91	0.0358	
4	0.2311	-0.00
91	0.0357	
5	0.5908	0.01
52	0.0426	
6	0.5896	-0.01
52	0.0426	
7	0.9490	0.02
01	0.0350	

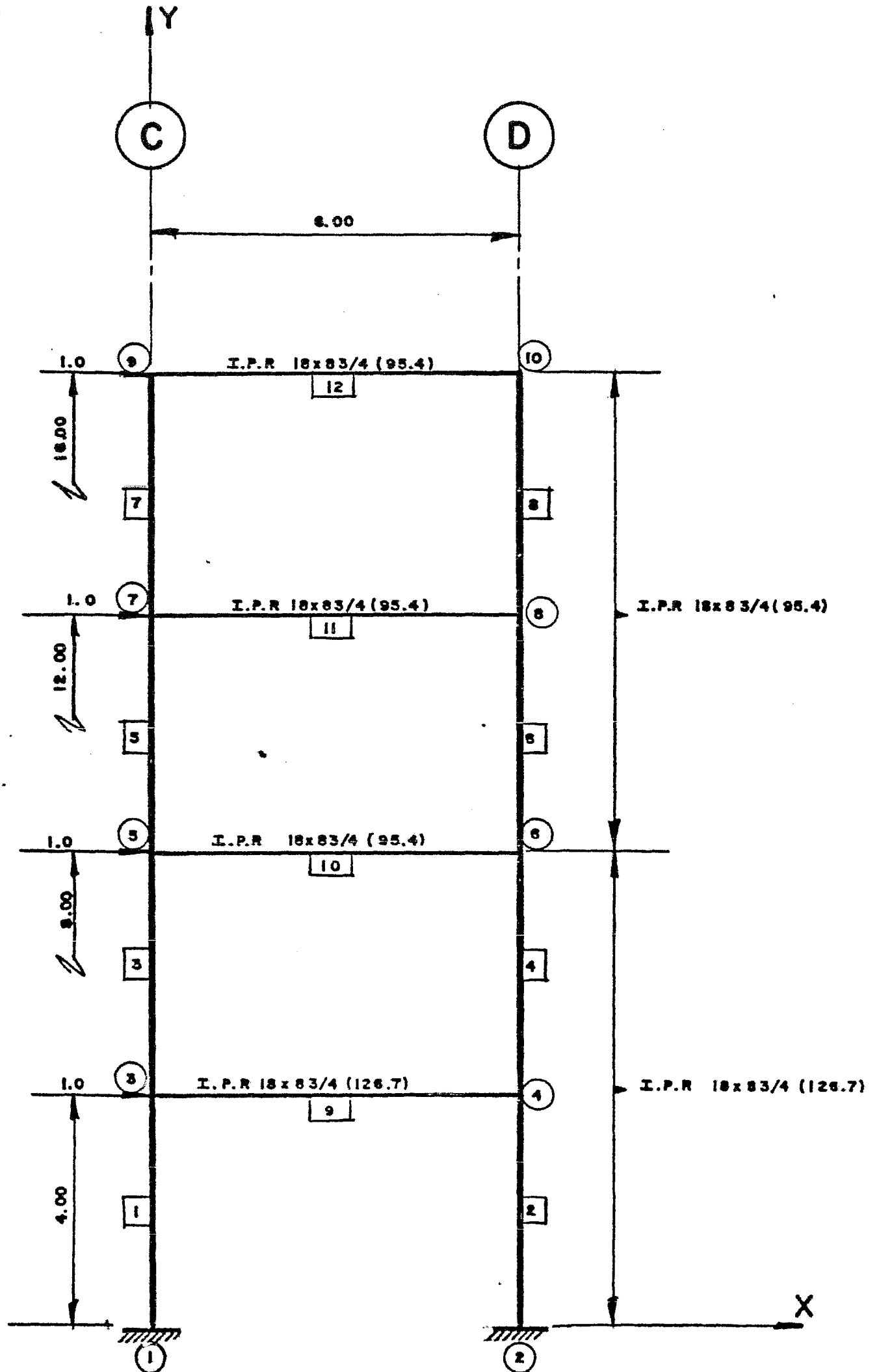
3	0.9478	-0.02
01	0.0350	
9	1.2220	0.02
24	0.0266	
10	1.2208	-0.02
24	0.0265	
11	1.5559	0.02
32	0.0433	
12	1.5529	-0.02
32	0.0429	

MIEMBRO AXIAL CORTE COLILLA EN TO  
NS CORTE MOMENTO EN M-TON

NO.	FORCE TONS		LO.JT	HI.JT
	LO.JT	HI.JT		
1	7.68	-3.06	2.51	-2.51
	-6.96			
2	-7.68	-3.04	2.49	-2.49
	-6.94			
3	5.21	-3.63	2.00	-2.00
	-4.37			
4	-5.21	-3.63	2.00	-2.00
	-4.38			
5	3.10	-3.30	1.50	-1.50
	-2.70			
6	-3.10	-3.30	1.50	-1.50
	-2.70			
7	1.45	-2.33	1.00	-1.00
	-1.66			
8	-1.45	-2.34	1.00	-1.00
	-1.66			
9	0.26	-0.79	0.50	-0.50
	-1.22			
10	-0.26	-0.78	0.50	-0.50
	-1.21			
11	-0.49	7.42	-2.47	2.47
	7.43			
12	-0.50	6.33	-2.11	2.11
	6.33			
13	-0.50	4.97	-1.66	1.66
	4.97			
14	-0.50	3.55	-1.18	1.18
	3.55			
15	-0.50	0.78	-0.26	0.26
	0.79			

FIN DE CASO

# MARCO EJE ③ ④ y ⑤



LISTADO DEL ARCHIVO DE DATOS : MARCO EJ

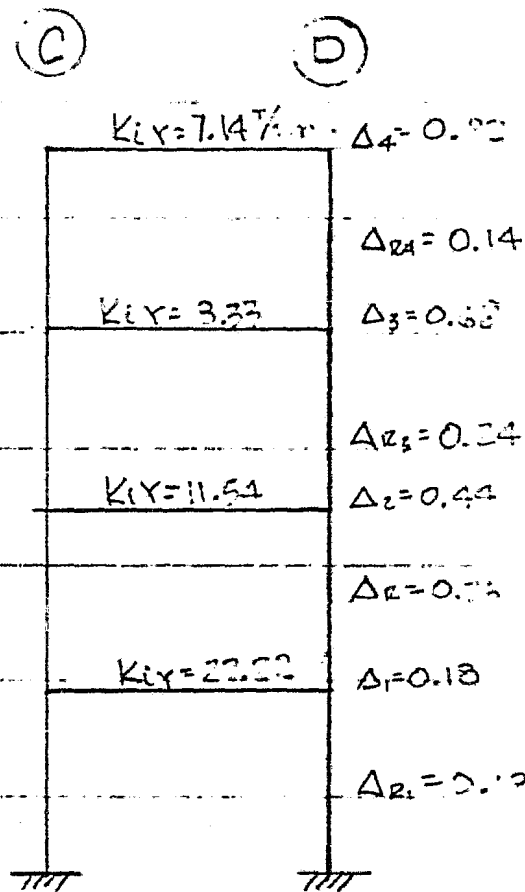
ES 3 4 Y 5

6704944-8

28-10-83

JESUS PONCE S

10	12	1	2100			
1	0	0	0	0	0	
2	0	0	0	0	0	
3	1	1	1	0	4	
4	1	1	1	0	4	
5	1	1	1	0	8	
6	1	1	1	0	8	
7	1	1	1	0	12	
8	1	1	1	0	12	
9	1	1	1	0	16	
10	1	1	1	0	16	
1	1	3	1	1	161.09	59517
2	2	4	1	1	161.09	59517
3	3	5	1	1	161.09	59517
4	4	6	1	1	161.09	59517
5	5	7	1	1	121.29	43529
6	6	8	1	1	121.29	43529
7	7	9	1	1	121.29	43529
8	8	10	1	1	121.29	43529
9	3	4	1	1	161.09	59517
10	5	6	1	1	121.29	43529
11	7	8	1	1	121.29	43529
12	9	10	1	1	121.29	43529



1  
SISMO  
4 0  
3 1 0 0  
5 1 0 0  
7 1 0 0  
9 1 0 0

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28-10-83

PROBLEMA DE: JESUS PONCE S

NUMERO DE NODOS : 10  
 NUMERO DE MIEMBROS : 12  
 NUMERO DE CARGAS : 1  
 MODULO DE ELASTICIDAD, MTONS/CM2: 2100  
 NODO YCOORD YCOORD XREL YREL MZREL

1	0.00	0.00000
2	0.00	0.00000
3	0.00	4.00111
4	0.00	4.00111
5	0.00	8.00111
6	0.00	8.00111
7	0.00	12.00111
8	0.00	12.00111
9	0.00	16.00111
10	0.00	16.00111

MIEMBRO LO.JT HI.JT LOW.FIX HI.FIX (CM)2  
 (CM)4 L.METER

1	1	311	161.09	59517.00
4.00				
2	2	411	161.09	59517.00
4.00				
3	3	511	161.09	59517.00
4.00				
4	4	611	161.09	59517.00
4.00				
5	5	711	121.29	43529.00
4.00				
6	6	811	121.29	43529.00
4.00				
7	7	911	121.29	43529.00
4.00				
8	8	1011	121.29	43529.00
4.00				
9	3	411	161.09	59517.00
6.00				
10	5	611	121.29	43529.00
6.00				
11	7	811	121.29	43529.00
6.00				
12	9	1011	121.29	43529.00
6.00				

GLOBAL STIFF MATRIX ROWS=25 COLS=9 ELEME  
NTS=225

SISMO

NODO NO.	X-DESPL CMETER	Y-DESPL CM
1	0.0000	0.00
2	0.0000	0.00
3	0.1804	0.00
4	0.1795	-0.00
5	0.4435	0.00
6	0.4423	-0.00
7	0.6788	0.01
8	0.6776	-0.01
9	0.8192	0.01
10	0.8180	-0.01

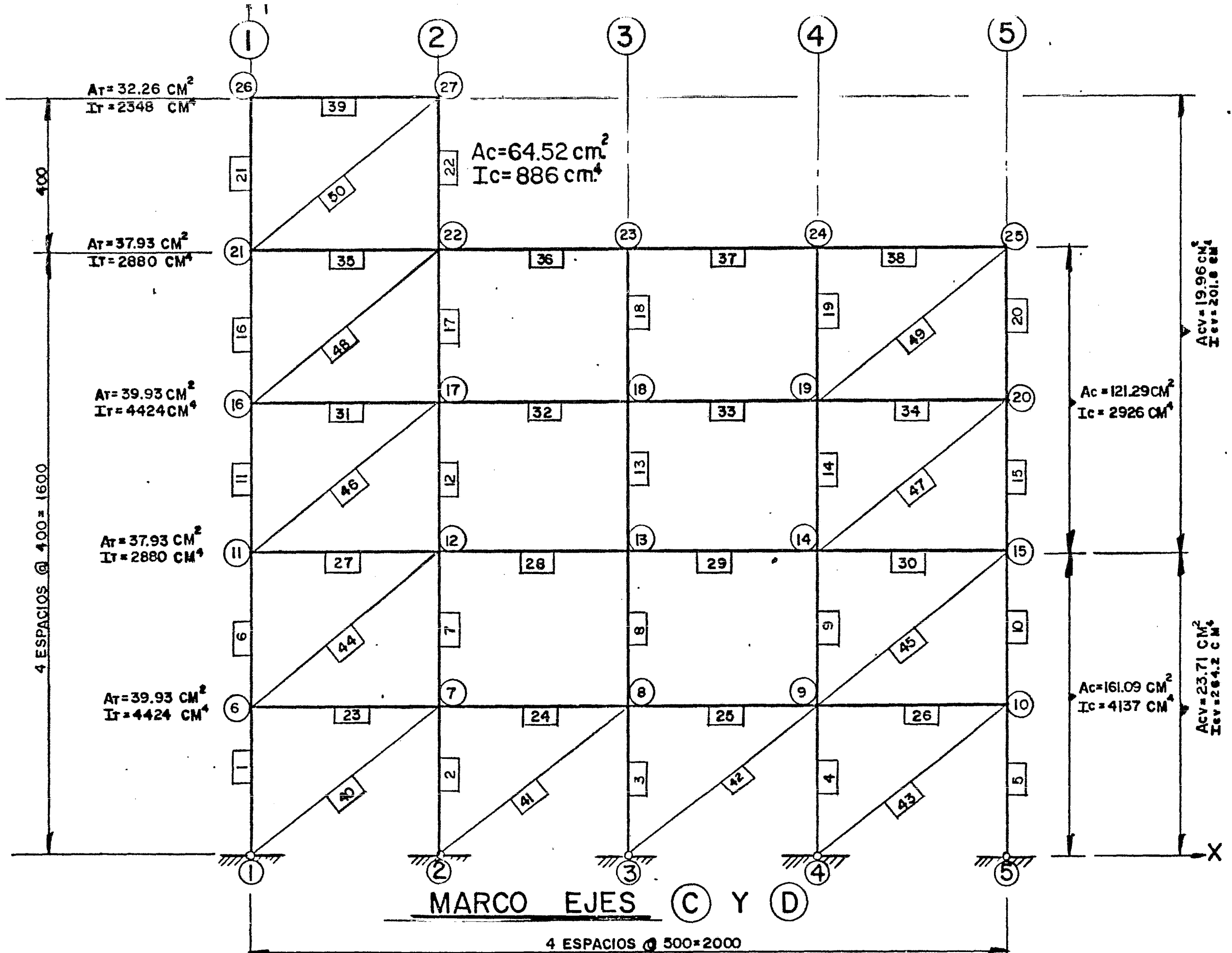
MIEMBRO AXIAL CORTE COLILLA EN TO  
NS CORTE MOMENTO EN M-TON

NO.	FORCE TONS	LO.JT	HI.JT
1	4.84	2.01	-2.01
2	-4.84	1.99	-1.99
3	2.95	1.50	-1.50
4	-2.95	1.50	-1.50
5	1.45	1.00	-1.00
6	-1.45	1.00	-1.00



7	-1.64	-2.38		
	0.46		0.50	-0.50
8	-0.61	-1.39		
	-0.46		0.50	-0.50
	-0.61	-1.39		
	-0.49		-1.89	1.89
	5.68	5.67		
	-0.50		-1.50	1.50
	4.49	4.47		
11	-0.50		-0.99	0.99
	2.97	2.87		
12	-0.50		-0.46	0.46
	1.39	1.39		

FIN DE CASO



ESTADO DEL ARCHIVO DE DATOS : MARCO

EJES C Y D

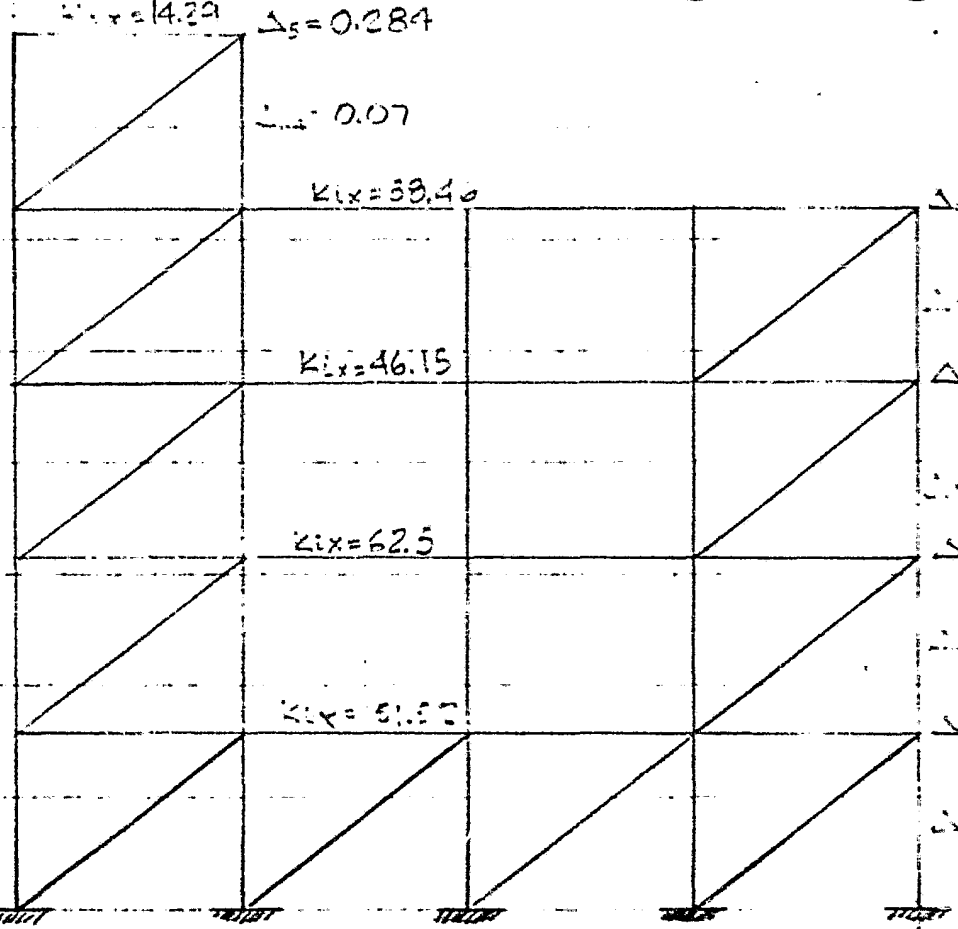
6704744-8

4-NOV-83

JESUS PONCE S



27	50	1	2100		
1	0	0	0	0	0
2	0	0	5	0	
3	0	0	10	0	
4	0	0	15	0	
5	0	0	20	0	
6	1	1	0	4	
7	1	1	5	4	
8	1	1	10	4	
9	1	1	15	4	
10	1	1	20	4	
11	1	1	0	8	
12	1	1	5	8	
13	1	1	10	8	
14	1	1	15	8	
15	1	1	20	8	
16	1	1	0	12	
17	1	1	5	12	
18	1	1	10	12	
19	1	1	15	12	
20	1	1	20	12	
21	1	1	0	16	
22	1	1	5	16	
23	1	1	10	16	
24	1	1	15	16	
25	1	1	20	16	
26	1	1	0	20	
27	1	1	5	20	



1	1	6	0	1	161.09	4137
2	2	7	0	1	161.09	4137
3	3	8	0	1	161.09	4137
4	4	9	0	1	161.09	4137
5	5	10	0	1	161.09	4137
6	6	11	1	1	161.09	4137
7	7	12	1	1	161.09	4137
8	8	13	1	1	161.09	4137
9	9	14	1	1	161.09	4137
10	10	15	1	1	161.09	4137
11	11	16	1	1	121.29	2926
12	12	17	1	1	121.29	2926
13	13	18	1	1	121.29	2926
14	14	19	1	1	121.29	2926
15	15	20	1	1	121.29	2926
16	16	21	1	1	121.29	2926
17	17	22	1	1	121.29	2926
18	18	23	1	1	121.29	2926
19	19	24	1	1	121.29	2926
20	20	25	1	1	121.29	2926
21	21	26	1	1	84.52	88.6
22	22	27	1	1	84.52	88.6
23	3	7	1	1	39.93	4424
24	7	8	1	1	39.93	4424
25	8	9	1	1	39.93	4424
26	9	10	1	1	39.93	4424
27	11	12	1	1	37.93	2880
28	12	13	1	1	37.93	2880

27	13	14	1	1	37.93	2880
30	14	15	1	1	37.93	2880
31	16	17	1	1	39.93	4424
32	17	18	1	1	39.93	4424
33	18	19	1	1	39.93	4424
34	19	20	1	1	39.93	4424
35	21	22	1	1	37.93	2880
36	22	23	1	1	37.93	2880
37	23	24	1	1	37.93	2880
38	24	25	1	1	37.93	2880
39	26	27	1	1	32.26	2348
40	1	7	0	0	23.71	264.2
41	2	8	0	0	23.71	264.2
42	3	9	0	0	23.71	264.2
43	4	10	0	0	23.71	264.2
44	6	12	0	0	23.71	264.2
45	9	15	0	0	23.71	264.2
46	11	17	0	0	19.96	201.8
47	14	20	0	0	19.96	201.8
48	16	22	0	0	19.96	201.8
49	19	25	0	0	19.96	201.8
50	21	27	0	0	19.96	201.8

SISMO  
5 0  
6 1 0 0  
11 1 0 0  
16 1 0 0  
21 1 0 0  
26 1 0 0

6704944-8  
4-NOV-83  
PROBLEMA DE: JESUS PONCE S

NUMERO DE NODOS : 27  
NUMERO DE MIEMBROS : 50  
NUMERO DE CARGAS : 1  
MODULO DE ELASTICIDAD, MTONS/CM2: 2100

NODO	XCOORD	YCOORD	XREL	YREL	MZREL
1	0.00	0.00000			
2	5.00	0.00000			
3	10.00	0.00000			
4	15.00	0.00000			
5	20.00	0.00000			
6	0.00	4.00111			
7	5.00	4.00111			
8	10.00	4.00111			
9	15.00	4.00111			
10	20.00	4.00111			
11	0.00	8.00111			
12	5.00	8.00111			
13	10.00	8.00111			
14	15.00	8.00111			
15	20.00	8.00111			
16	0.00	12.00111			

17	5.00	12.00	111
18	10.00	12.00	111
19	15.00	12.00	111
20	20.00	12.00	111
21	0.00	16.00	111
22	5.00	16.00	111
23	10.00	16.00	111
24	15.00	16.00	111
25	20.00	16.00	111
26	0.00	20.00	111
27	5.00	20.00	111

MIEMBRO LO.JT HI.JT LOW.FIX HI.FIX (CM)2  
(CM)4 L.METER

1	1	601	161.09	4137.00
4.00				
2	2	701	161.09	4137.00
4.00				
3	3	801	161.09	4137.00
4.00				
4	4	901	161.09	4137.00
4.00				
5	5	1001	161.09	4137.00
4.00				
6	6	1111	161.09	4137.00
4.00				
7	7	1211	161.09	4137.00
4.00				
8	8	1311	161.09	4137.00
4.00				
9	9	1411	161.09	4137.00
4.00				
10	10	1511	161.09	4137.00
4.00				
11	11	1611	121.29	2926.00
4.00				
12	12	1711	121.29	2926.00
4.00				
13	13	1811	121.29	2926.00
4.00				
14	14	1911	121.29	2926.00
4.00				
15	15	2011	121.29	2926.00
4.00				
16	16	2111	121.29	2926.00
4.00				
17	17	2211	121.29	2926.00
4.00				
18	18	2311	121.29	2926.00
4.00				
19	19	2411	121.29	2926.00
4.00				
20	20	2511	121.29	2926.00
4.00				
21	21	2611	64.52	886.00
4.00				
22	22	2711	64.52	886.00
4.00				
23	23	711	39.93	4424.00
5.00				

24	7	811	39.93	4424.00
5.00				
25	8	911	39.93	4424.00
5.00				
26	9	1011	39.93	4424.00
5.00				
27	11	1211	37.93	2880.00
5.00				
28	12	1311	37.93	2880.00
5.00				
29	13	1411	37.93	2880.00
5.00				
30	14	1511	37.93	2880.00
5.00				
31	16	1711	39.93	4424.00
5.00				
32	17	1811	39.93	4424.00
5.00				
33	18	1911	39.93	4424.00
5.00				
34	19	2011	39.93	4424.00
5.00				
35	21	2211	37.93	2880.00
5.00				
36	22	2311	37.93	2880.00
5.00				
37	23	2411	37.93	2880.00
5.00				
38	24	2511	37.93	2880.00
5.00				
39	26	2711	32.26	2348.00
5.00				
40	1	700	23.71	264.20
6.40				
41	2	800	23.71	264.20
6.40				
42	3	900	23.71	264.20
6.40				
43	4	1000	23.71	264.20
6.40				
44	6	1200	23.71	264.20
6.40				
45	9	1500	23.71	264.20
6.40				
46	11	1700	19.96	201.80
6.40				
47	14	2000	19.96	201.80
6.40				
48	16	2200	19.96	201.80
6.40				
49	19	2500	19.96	201.80
6.40				
50	21	2700	19.96	201.80
6.40				

GLOBAL STIFF MATRIX ROWS=87 COLS=21 ELEMENTS=1407

SISMO  
 NODO NO. X-DESPL CMETER Y-DESPL CM  
 ITER ROTACION GRADO

1	0.0000		0.00
00	0.0000		
2	0.0000		0.00
00	0.0000		
3	0.0000		0.00
00	0.0000		
4	0.0000		0.00
00	0.0000		
5	0.0000		0.00
00	0.0000		
6	0.0520		0.00
47	0.0059		
7	0.0357		-0.00
59	0.0034		
8	0.0278	0.033	-0.00
12	0.0031		
9	0.0271		0.00
80	0.0033		
10	0.0222		-0.00
53	0.0041		
11	0.1147		0.00
77	0.0078		
12	0.1004		-0.01
05	0.0062		
13	0.0971	0.097	-0.00
12	0.0061		
14	0.0939		0.00
54	0.0064		
15	0.0809		-0.00
98	0.0080		
16	0.1758		0.01
01	0.0064		
17	0.1646		-0.01
44	0.0039		
18	0.1611	0.162	-0.00
13	0.0027		
19	0.1576		0.00
67	0.0037		
20	0.1482		-0.01
80	0.0064		
21	0.2316		0.01
14	0.0067		
22	0.2192		-0.01
67	0.0040		
23	0.2124	0.214	-0.00
13	0.0015		
24	0.2058		0.00
67	0.0030		
25	0.1995		-0.01
43	0.0053		
26	0.2880		0.01
14	0.0045	0.024	
27	0.2806		-0.01
91	0.0056		

MIEMBRO AXIAL CORTE COLLILLA EN TO  
 NS CORTE MOMENTO EN M-TON

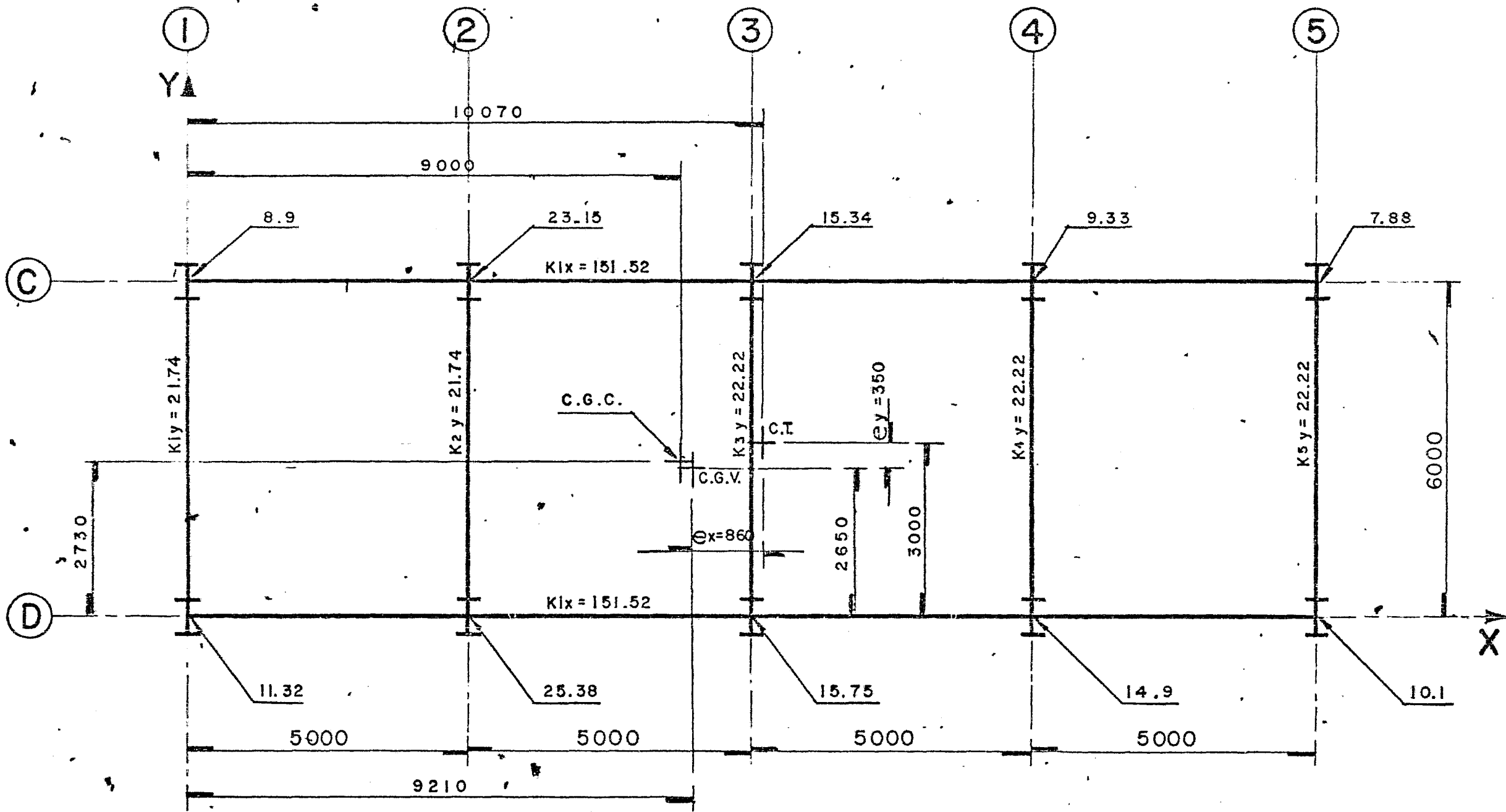
NO.	FORCE TONS	LO.JT	HI.JT
	LO.JT	HI.JT	
1	3.98	0.00	-0.00

	0.00	-0.02		
2	-5.02		0.00	-0.00
	0.00	-0.02		
3	-1.03		0.00	-0.00
	0.00	-0.01		
4	2.56		0.00	-0.00
	0.00	-0.01		
5	-4.48		-0.00	0.00
	0.00	0.01		
6	2.57		0.02	-0.02
	-0.06	-0.04		
7	-3.85		0.05	-0.05
	-0.11	-0.09		
8	-0.01		0.06	-0.06
	-0.13	-0.11		
9	2.04		0.05	-0.05
	-0.12	-0.10		
10	-3.77		0.03	-0.03
	-0.07	-0.04		
11	1.52		0.01	-0.01
	-0.02	-0.03		
12	-2.49		0.03	-0.03
	-0.06	-0.07		
13	-0.01		0.04	-0.04
	-0.07	-0.09		
14	0.78		0.03	-0.03
	-0.06	-0.07		
15	-2.09		0.02	-0.02
	-0.03	-0.04		
16	0.80		0.01	-0.01
	-0.02	-0.02		
17	-1.48		0.03	-0.03
	-0.06	-0.06		
18	-0.01		0.04	-0.04
	-0.08	-0.09		
19	-0.01		0.03	-0.03
	-0.06	-0.06		
20	-0.81		0.01	-0.01
	-0.02	-0.03		
21	0.01		0.01	-0.01
	-0.01	-0.01		
22	-0.79		0.01	-0.01
	-0.02	-0.02		
23	-2.74		-0.03	0.03
	0.07	0.06		
24	-1.32		-0.03	0.03
	0.07	0.07		
25	-0.11		-0.03	0.03
	0.07	0.07		
26	-0.82		-0.02	0.02
	0.05	0.06		
27	-2.27		-0.02	0.02
	0.07	0.06		
28	-0.54		-0.04	0.04
	0.09	0.09		
29	-0.51		-0.04	0.04
	0.09	0.09		
30	-2.07		-0.03	0.03
	0.07	0.07		
31	-1.87		-0.02	0.02



	0.05	0.04		
32	-0.59		-0.04	0.04
	0.10	0.09		
33	-0.59		-0.03	0.03
	0.08	0.08		
34	-1.59		-0.03	0.02
	0.05	0.06		
35	-1.98		-0.01	0.01
	0.03	0.02		
36	-1.08		-0.02	0.02
	0.06	0.05		
37	-1.04		-0.02	0.02
	0.04	0.04		
38	-1.01		-0.01	0.01
	0.02	0.03		
39	-0.99		-0.01	0.01
	0.01	0.02		
40	1.88		0.00	0.00
	0.00	0.00		
41	1.63		0.00	0.00
	0.00	0.00		
42	1.79		0.00	0.00
	0.00	0.00		
43	1.09		0.00	-0.00
	0.00	0.00		
44	2.20		0.00	0.00
	0.00	0.00		
45	2.64		0.00	0.00
	0.00	0.00		
46	1.65		-0.00	0.00
	0.00	0.00		
47	2.02		-0.00	0.00
	0.00	0.00		
48	1.12		-0.00	0.00
	0.00	0.00		
49	1.28		0.00	0.00
	0.00	0.00		
50	1.26		-0.00	0.00
	0.00	0.00		

FIN DE CASO



PLANTA NIVEL + 2

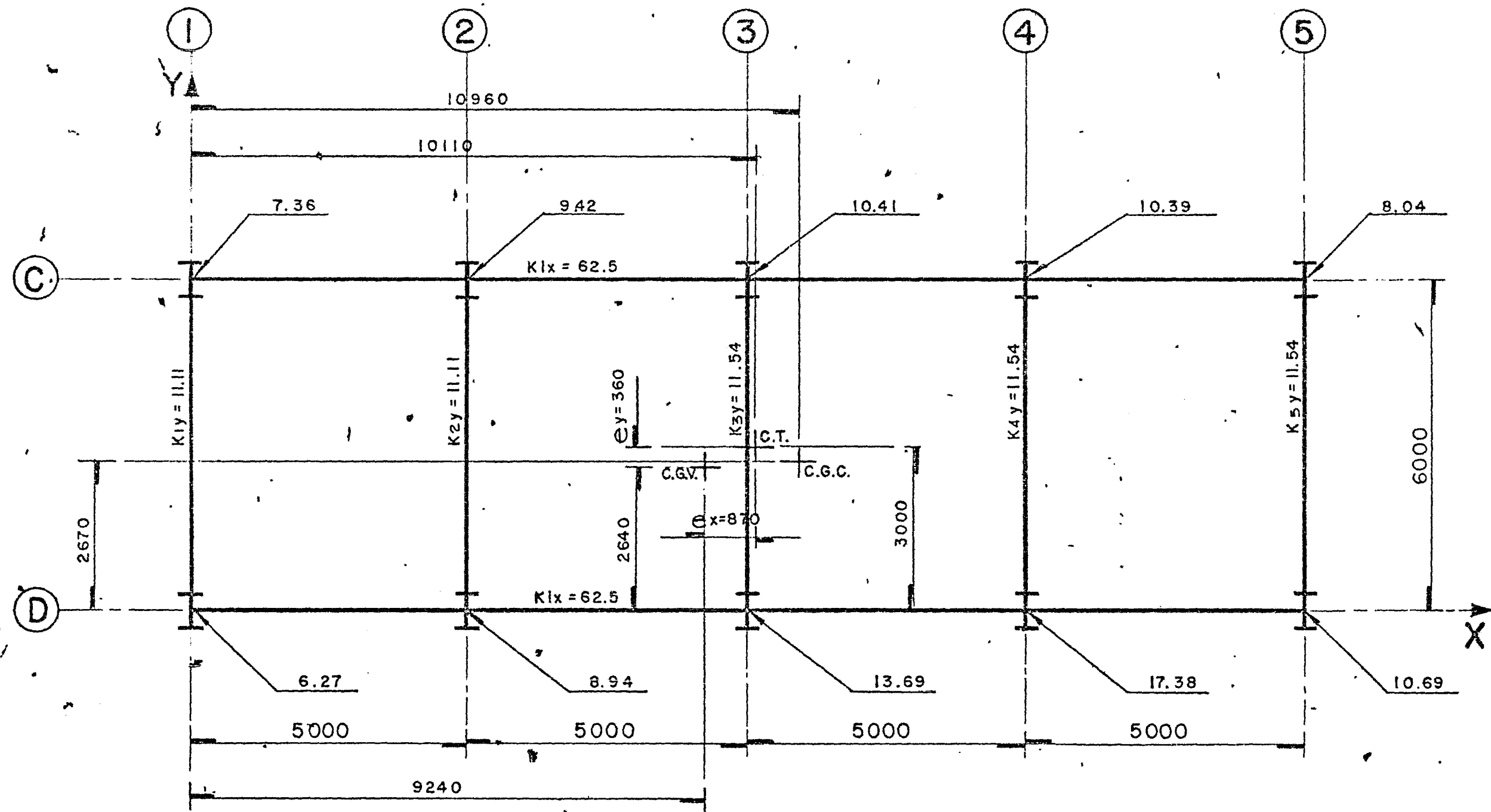
# PLANTA NIVEL + 2

# C.G DE CARGAS

COLUMNA	W (TON.)	COORDENADAS		W · X	W · Y
		X	Y		
C - 1	8.9	0	6	0	53.4
C - 2	23.15	5	6	115.75	138.9
C - 3	15.34	10	6	153.40	92.04
C - 4	9.33	15	6	139.95	55.98
C - 5	7.88	20	6	157.60	47.28
D - 1	11.32	0	0	0	0
D - 2	25.38	5	0	126.90	0
D - 3	15.75	10	0	157.50	0
D - 4	14.90	15	0	223.50	0
D - 5	10.10	20	0	202.00	0
Σ	142.05			1276.60	387.6

$$\bar{X} = \frac{\sum W \cdot X}{\sum W} = \frac{1276.60}{142.05} = 9.0 \text{ MTS.}$$

$$\bar{Y} = \frac{\sum W \cdot Y}{\sum W} = \frac{387.6}{142.05} = 2.73 \text{ MTS.}$$



PLANTA NIVEL + 3

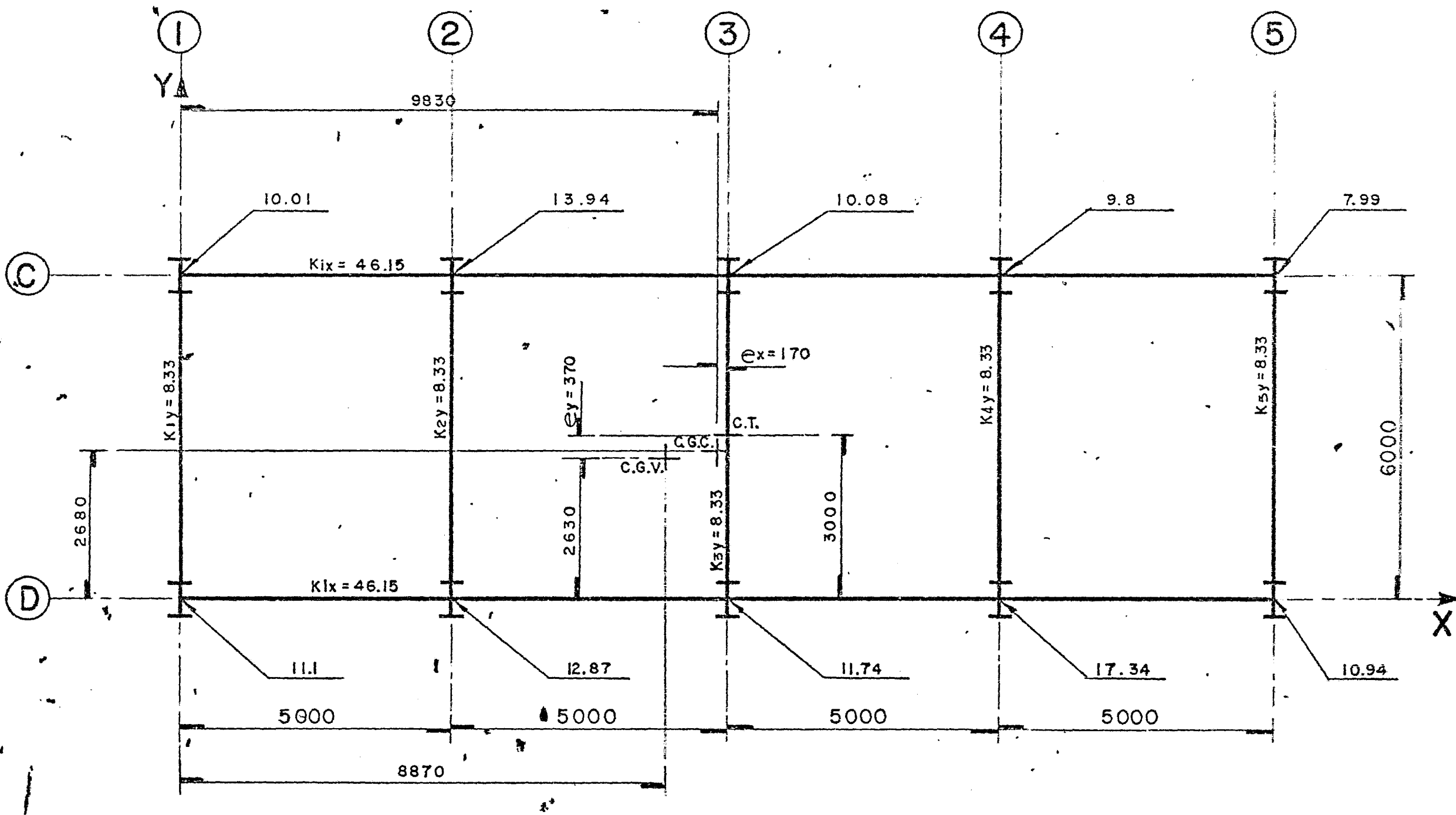
# PLANTA NIVEL + 3

# C.G DE CARGAS

COLUMNA	W (TON.)	COORDENADAS		W · X	W · Y
		X	Y		
C - 1	7.36	0	6	0	44.16
C - 2	9.42	5	6	47.1	56.52
C - 3	10.41	10	6	104.1	62.46
C - 4	10.39	15	6	155.85	62.34
C - 5	8.04	20	6	160.80	48.24
D - 1	6.27	0	0	0	0
D - 2	8.94	5	0	44.7	0
D - 3	13.69	10	0	136.9	0
D - 4	17.38	15	0	260.7	0
D - 5	10.69	20	0	213.8	0
Σ	102.59			1123.95	273.72

$$\bar{X} = \frac{\sum W \cdot X}{\sum W} = \frac{1123.95}{102.59} = 10.96 \text{ MTS.}$$

$$\bar{Y} = \frac{\sum W \cdot Y}{\sum W} = \frac{273.72}{102.59} = 2.67 \text{ MTS.}$$



PLANTA NIVEL + 4

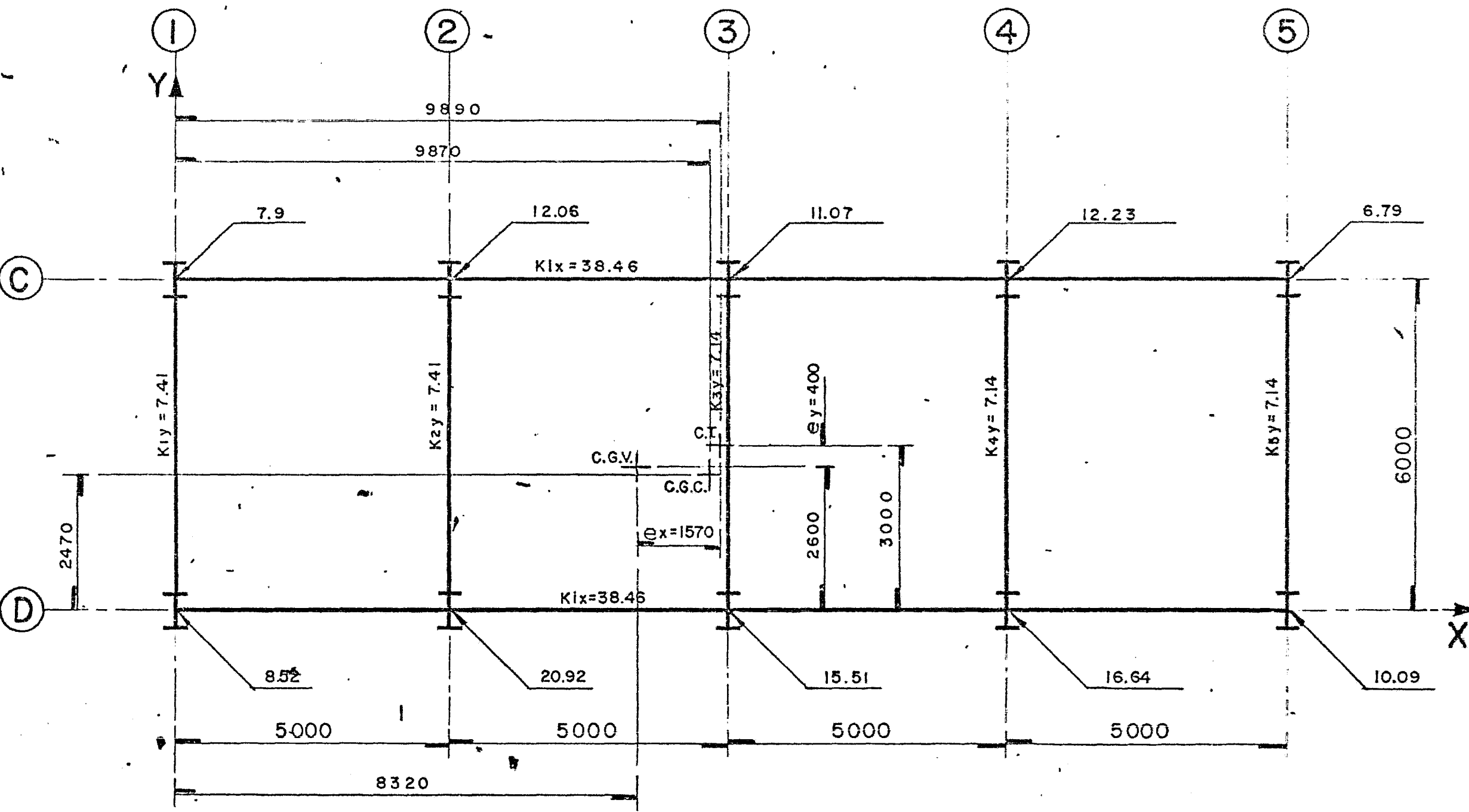
# PLANTA NIVEL + 4

# C.G DE CARGAS

COLUMNA	W (TON.)	COORDENADAS		W·X	W·Y
		X	Y		
C-1	10.01	0	6	0	60.06
C-2	13.94	5	6	69.7	83.64
C-3	10.08	10	6	100.8	60.48
C-4	9.8	15	6	147.0	58.80
C-5	7.99	20	6	159.8	47.94
D-1	11.10	0	0	0	0
D-2	12.87	5	0	64.35	0
D-3	11.74	10	0	117.40	0
D-4	17.34	15	0	260.10	0
D-5	10.94	20	0	218.8	0
Σ	115.81			1137.95	310.92

$$\bar{X} = \frac{\sum W \cdot X}{\sum W} = \frac{1137.95}{115.81} = 9.83 \text{ MTS.}$$

$$\bar{Y} = \frac{\sum W \cdot Y}{\sum W} = \frac{310.92}{115.81} = 2.68 \text{ MTS.}$$



PLANTA NIVEL + 5



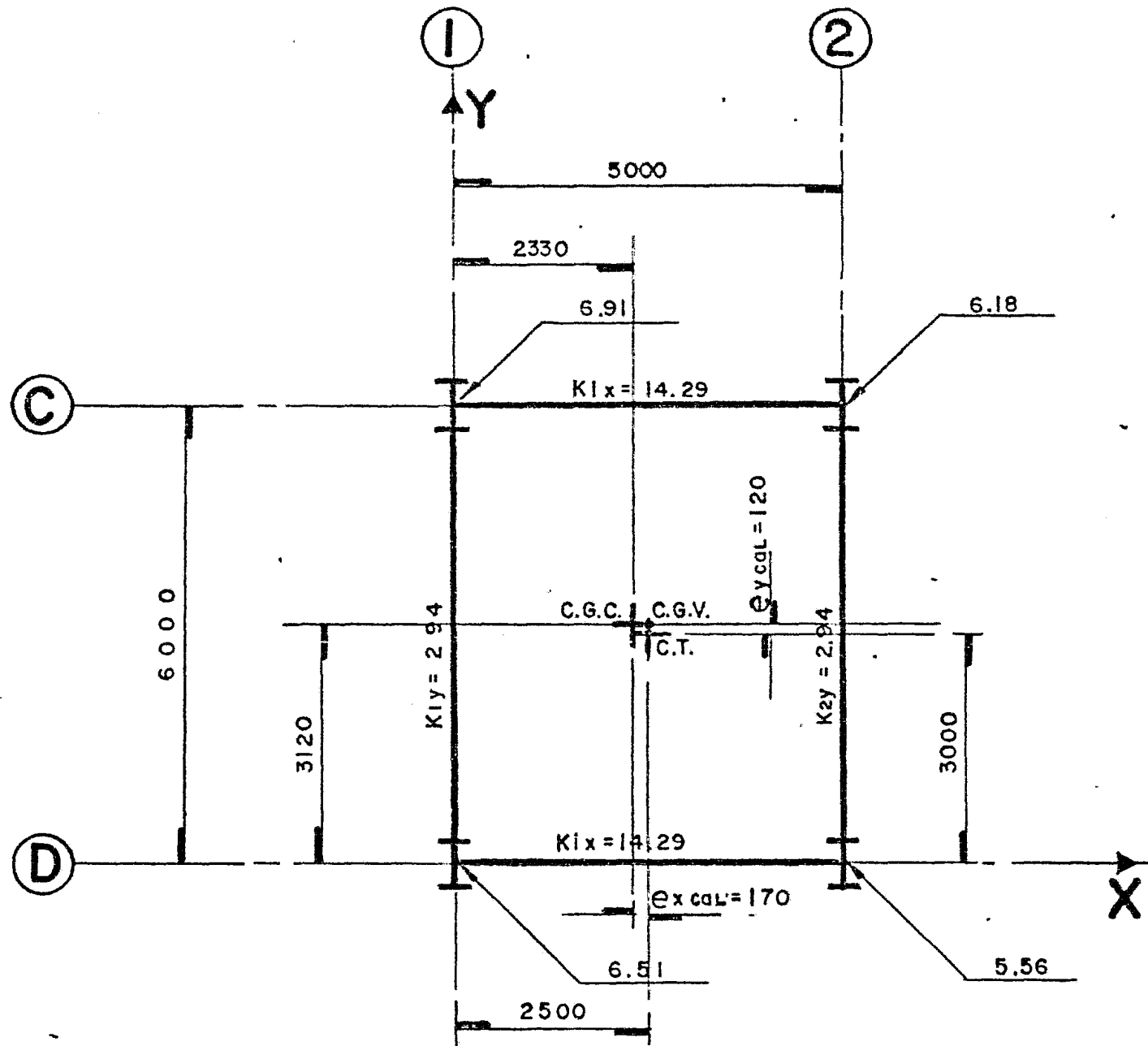
PLANTA NIVEL + 5

C.G DE CARGAS

COLUMNA	W (TON.)	COORDENADAS		W·X	W·Y
		X	Y		
C-1	7.9	0	6	0	47.4
C-2	12.06	5	6	60.3	72.36
C-3	11.07	10	6	110.7	66.42
C-4	12.23	15	6	183.45	73.38
C-5	6.79	20	6	135.80	40.74
D-1	8.52	0	0	0	0
D-2	20.92	5	0	104.60	0
D-3	15.51	10	0	155.10	0
D-4	16.64	15	0	249.60	0
D-5	10.09	20	0	201.80	0
Σ	121.73			1201.35	300.3

$$\bar{X} = \frac{\sum W \cdot X}{\sum W} = \frac{1201.35}{121.73} = 9.87 \text{ MTS.}$$

$$\bar{Y} = \frac{\sum W \cdot Y}{\sum W} = \frac{300.3}{121.73} = 2.47 \text{ MTS.}$$



PLANTA NIVEL + 6

# PLANTA NIVEL + 6

# C.G DE CARGAS

COLUMNA	W (TON.)	COORDENADAS		W·X	W·Y
		X	Y		
C-1	6.91	0	6	0	41.46
C-2	6.18	5	6	30.9	37.08
D-1	6.51	0	0	0	0
D-2	5.56	5	0	27.8	0
Σ	25.16			58.7	78.54

$$\bar{X} = \frac{\sum W \cdot X}{\sum W} = \frac{58.7}{25.16} = 2.33 \text{ MTS.}$$

$$\bar{Y} = \frac{\sum W \cdot Y}{\sum W} = \frac{78.54}{25.16} = 3.12 \text{ MTS.}$$

NIVEL	ENTRE-PISO	W <sub>i</sub>	h <sub>i</sub>	W <sub>i</sub> h <sub>i</sub>	F <sub>ix</sub>	F <sub>ly</sub>	CORTANTES		C. DE G. C.		F <sub>ix</sub> · $\bar{Y}$	F <sub>ly</sub> · $\bar{X}$	F <sub>ix</sub> · $\bar{Y}$	F <sub>ly</sub> · $\bar{X}$	POSICION DE CORTANTE	
							V <sub>x</sub>	V <sub>y</sub>	$\bar{Y}$	$\bar{X}$					$Y = \frac{\sum F_{ix} \cdot \bar{Y}}{V_x}$	$X = \frac{\sum F_{ly} \cdot \bar{X}}{V_y}$
6		25.16	20	503.2	5.86	5.86			3.12	2.33	18.28	13.65	18.28	13.65		
	5						5.86	5.86							3.12	3.12
5		121.73	16	1947.68	22.67	22.67			2.47	9.87	56.0	223.75	74.28	237.40		
	4						28.53	28.53							2.60	8.32
4		115.81	12	1389.72	16.18	16.18			2.68	9.83	43.36	159.05	117.64	396.45		
	3						44.71	44.71							2.63	8.87
3		102.59	8	820.72	9.55	9.55			2.67	10.96	25.50	104.67	143.14	501.12		
	2						54.26	54.26							2.64	9.24
2		142.05	4	568.20	6.61	6.61			2.73	9.0	18.05	59.49	161.19	560.61		
	1						60.87	60.87							2.65	9.21

507.34

5229.52

$$F_i = \frac{W_i h_i}{\sum W_i h_i} \cdot C \cdot W_i$$

$$V_b = C \sum W_i = 60.88$$

# DISTRIBUCION DE CORTANTES ENTRE LOS ELEMENTOS RESISTENTES EN CADA PISO

ENTREPISO: I  
 $V_x = V_y = 60.87$

$Y_v = 2.65 \text{ MTS.}$   
 $X_v = 9.21 \text{ MTS.}$

$e_x \text{ CAL} = 0.86 \text{ MTS.}$   
 $e_y \text{ CAL} = 0.35 \text{ MTS.}$

EJE	Kiy	Xi	Kiy - Xi	Xit	Kiy Xit	Kiy X <sup>2</sup> <sub>it</sub>	EFECTO DE Vy			EFECTO DE Vx
							DIRECTO	TORSION	TOTAL	TORSION
1	21.74	0	0	-10.07	-218.92	2204.54	12.02	5.33	17.35	-1.82
2	21.74	5	108.7	-5.07	-110.22	558.83	12.02	2.69	14.71	-0.92
3	22.22	10	222.2	-0.07	-1.56	0.11	12.28	0.04	12.32	-0.01
4	22.22	15	333.3	4.93	109.55	540.06	12.28	0.93	13.21	0.91
5	22.22	20	444.4	9.93	220.65	2191.0	12.28	1.86	14.14	1.84

SUMAS 110.14

1108.6

5494.54

$X_T = 10.07 \text{ MTS.}$

$$M_{tx1} = 60.87 (1.5 \times 0.35 + 0.1 \times 6) = 68.48 \text{ T.M.}$$

$$M_{tx2} = 60.87 (0.35 - 0.1 \times 6) = 15.22 \text{ T.M.}$$

$$M_{ty1} = 60.87 (1.5 \times 0.86 + 0.1 \times 20) = 200.26 \text{ T.M.}$$

$$M_{ty2} = 60.87 (0.86 - 0.1 \times 20) = 69.39 \text{ T.M.}$$

$$\sum Kiy X_{it}^2 + \sum Kix Y_{it}^2 = 8221.9$$

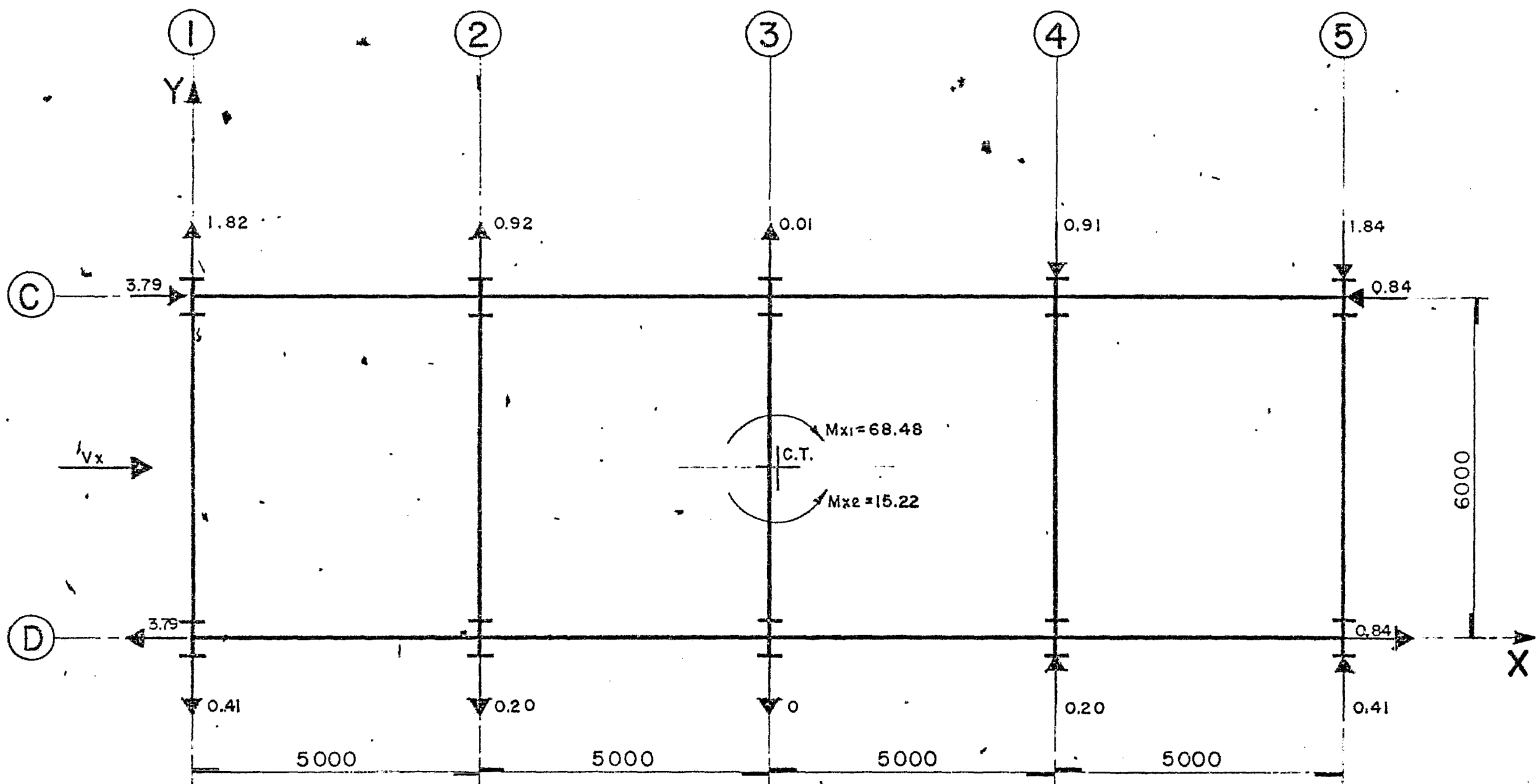
EJE	Kix	Yi	Kix Yi	Yit	Kix Yit	Kix Y <sup>2</sup> <sub>it</sub>	EFECTO DE Vx			EFECTO DE Vy
							DIRECTO	TORSION	TOTAL	TORSION
C	151.52	6	909.12	3	454.56	1363.68	30.44	3.79	34.23	11.07
D	151.52	0	0	-3	-454.56	1363.68	30.44	0.84	31.28	-11.07

SUMAS 303.04

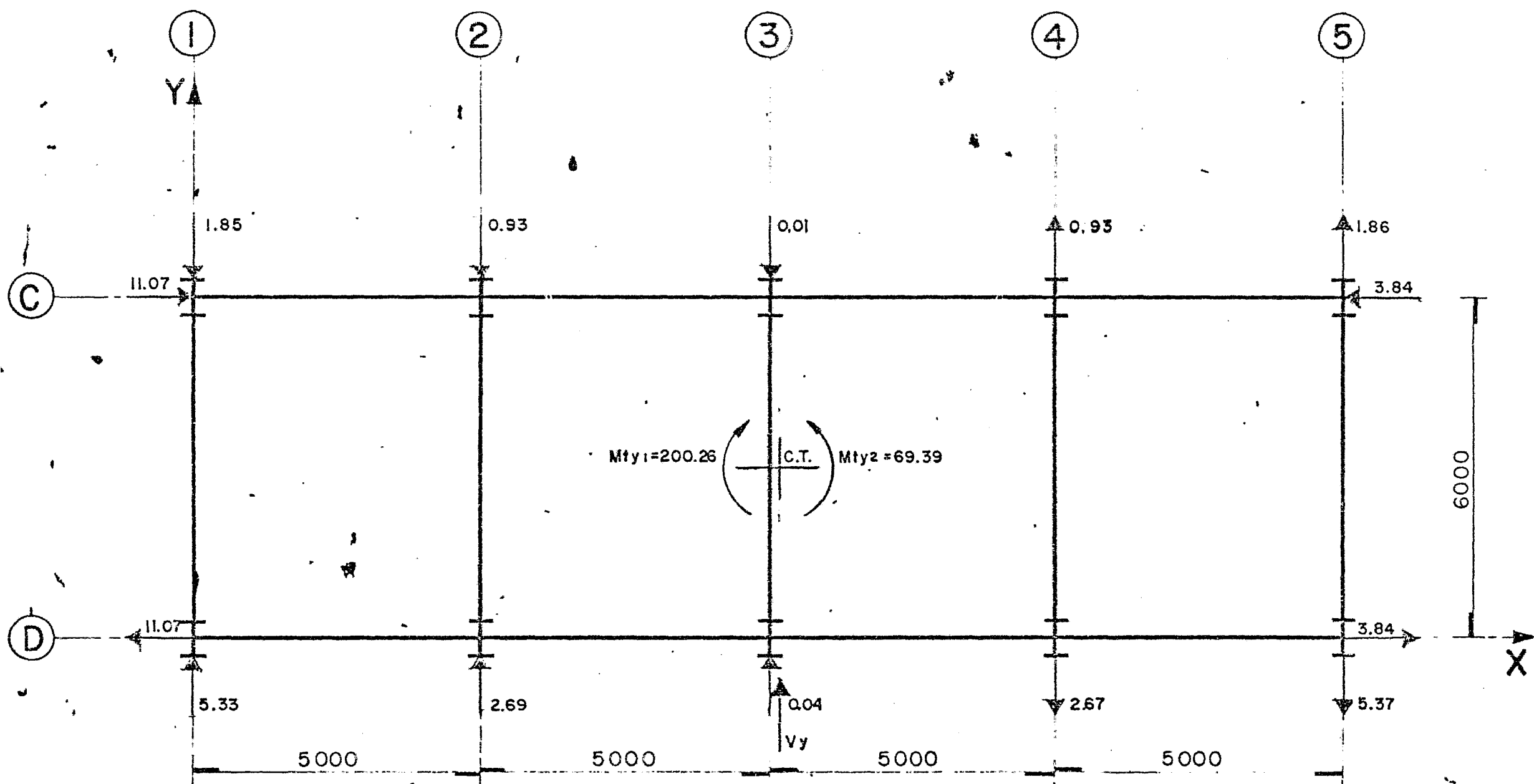
909.12

2727.36

$Y_T = 3.0 \text{ MTS.}$



PLANTA NIVEL + 2



PLANTA NIVEL + 2

# DISTRIBUCION DE CORTANTES ENTRE LOS ELEMENTOS RESISTENTES EN CADA PISO

ENTREPISO: 2

$Y_v = 2.64$  MTS.

$e_x \text{ CAL} = 0.87$  MTS.

$V_x = V_y = 54.26$  TON.  $X_v = 9.24$  MTS.

$e_y \text{ CAL} = 0.36$  MTS.

EJE	Kiy	Xi	Kiy-Xi	Xit	Kiy Xit	Kiy X <sup>2</sup> it	EFECTO DE Vy			EFECTO DE Vx
							DIRECTO	TORSION	TOTAL	TORSION
1	11.11	0	0	-10.11	-112.52	1135.58	10.61	5.09	15.7	-1.76
2	11.11	5	55.55	-5.11	-56.77	290.11	10.61	2.57	13.18	-0.89
3	11.54	10	115.40	-0.11	-1.27	0.14	11.02	0.06	11.08	-0.02
4	11.54	15	173.10	4.89	56.43	275.95	11.02	1.27	12.29	0.88
5	11.54	20	230.80	9.89	114.13	1128.75	11.02	2.57	15.59	1.78

SUMAS 56.84

574.85

2830.53

$X_T = 10.11$

$$M_{tx1} = (1.5 \times 0.36 + 0.1 \times 6) 54.26 = 61.86 \text{ T.M.}$$

$$M_{tx2} = (0.36 - 0.1 \times 6) 54.26 = 13.02 \text{ T.M.}$$

$$M_{ty1} = (1.5 \times 0.87 + 0.1 \times 20) 54.26 = 179.33 \text{ T.M.}$$

$$M_{ty2} = (0.36 - 0.1 \times 20) 54.26 = 88.99 \text{ T.M.}$$

$$\sum Kiy X_{it}^2 + \sum Kix Y_{it}^2 = 3955.53$$

EJE	Kix	Yi	Kix Yi	Yit	Kix Yit	Kix Y <sup>2</sup> it	EFECTO DE Vx			EFECTO DE Vy
							DIRECTO	TORSION	TOTAL	TORSION
C	62.5	6	375	3	187.5	562.5	27.13	2.93	30.06	8.5
D	62.5	0	0	-5	-187.5	562.5	27.13	0.62	27.75	-8.5

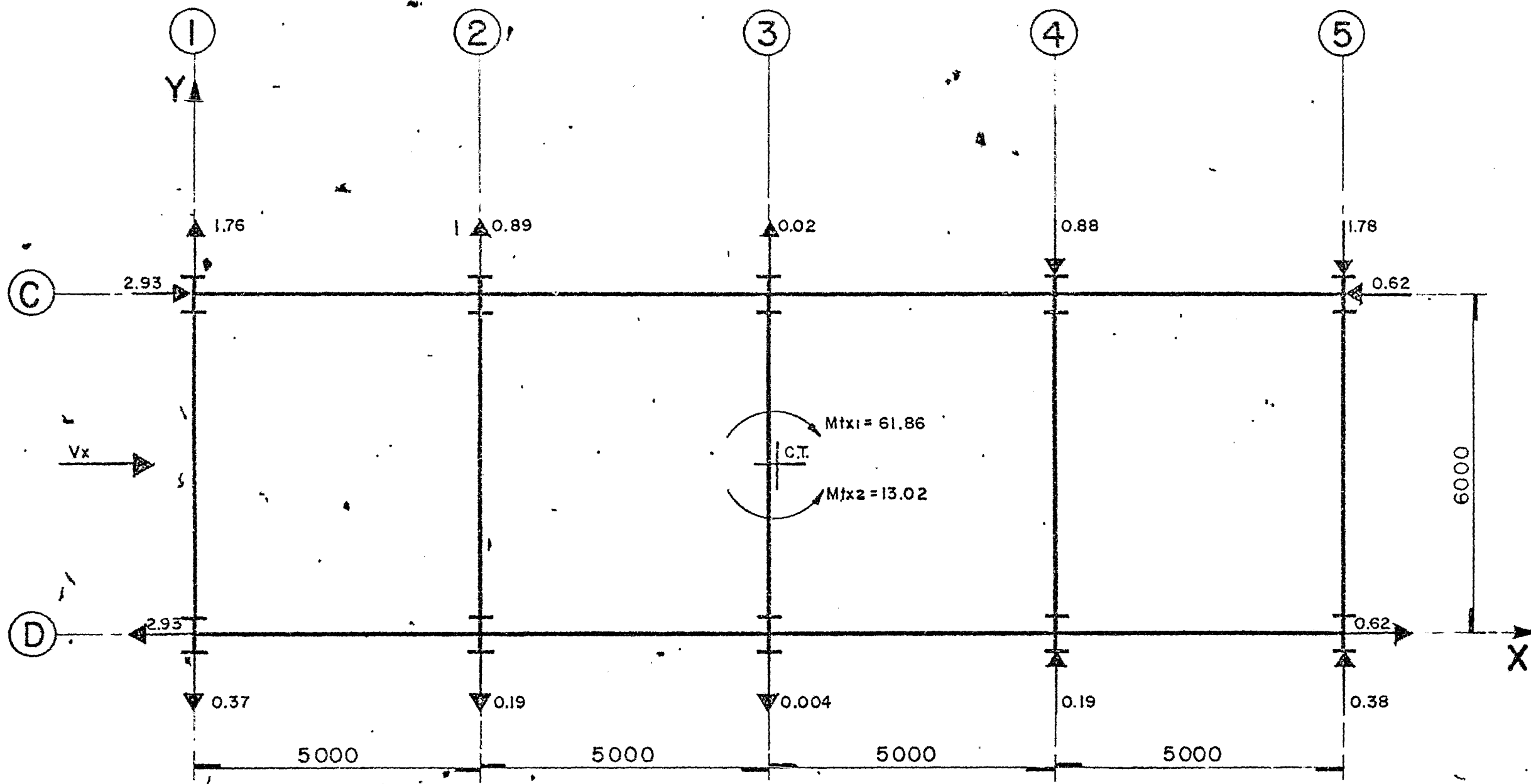
SUMAS 125

375

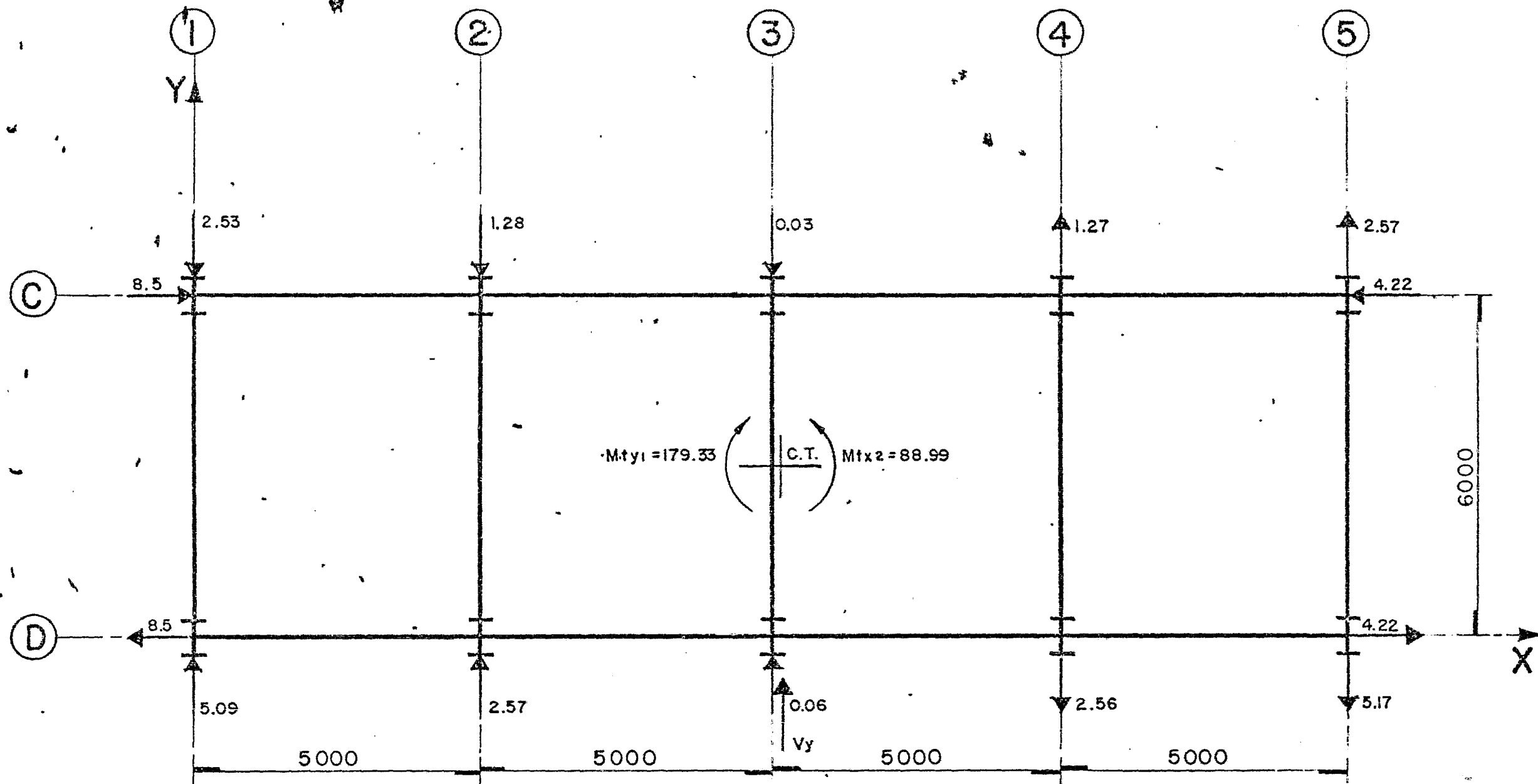
1125

$Y_T = 3$  MTS.





PLANTA NIVEL + 3



PLANTA NIVEL + 3

# DISTRIBUCION DE CORTANTES ENTRE LOS ELEMENTOS RESISTENTES EN CADA PISO

ENTREPISO: 3

$Y_v = 2.63$  MTS.

$e_x \text{ CAL} = 0.17$  MTS.

$V_x = V_y = 44.71$  TON.

$e_y = 8.87$

$e_y \text{ CAL} = 0.37$  MTS.

EJE	Kiy	Xi	Kiy - Xi	Xit	Kiy Xit	Kiy X <sup>2</sup> <sub>it</sub>	EFECTO DE Vy			EFFECTO DE Vx
							DIRECTO	TORSION	TOTAL	TORSION
1	8.33	0	0	-10	-83.3	833	8.94	2.88	11.82	-1.48
2	8.33	5	41.65	-5	-41.65	208.25	8.94	1.44	10.38	-0.74
3	8.33	10	83.3	0	0	0	8.94	0	8.94	0
4	8.33	15	124.95	5	41.65	208.25	8.94	1.04	9.98	0.74
5	8.33	20	166.6	10	83.30	833	8.94	2.08	11.02	1.48

SUMAS 41.65

416.5

2082.5

$$X_T = \frac{416.5}{41.65} = 10$$

$$M_{tx1} = 44.71 (1.5 \times 0.37 + 0.1 \times 6) = 51.64 \text{ T.M.}$$

$$M_{tx2} = 44.71 (0.37 - 0.1 \times 6) = -10.20 \text{ T.M.}$$

$$M_{ty1} = 44.71 (1.5 \times 0.17 + 0.1 \times 20) = 100.92 \text{ T.M.}$$

$$M_{ty2} = 44.71 (0.37 - 0.1 \times 20) = -72.88 \text{ T.M.}$$

$$\sum Kiy X_{it}^2 + \sum Kix Y_{it}^2 = 2913.2$$

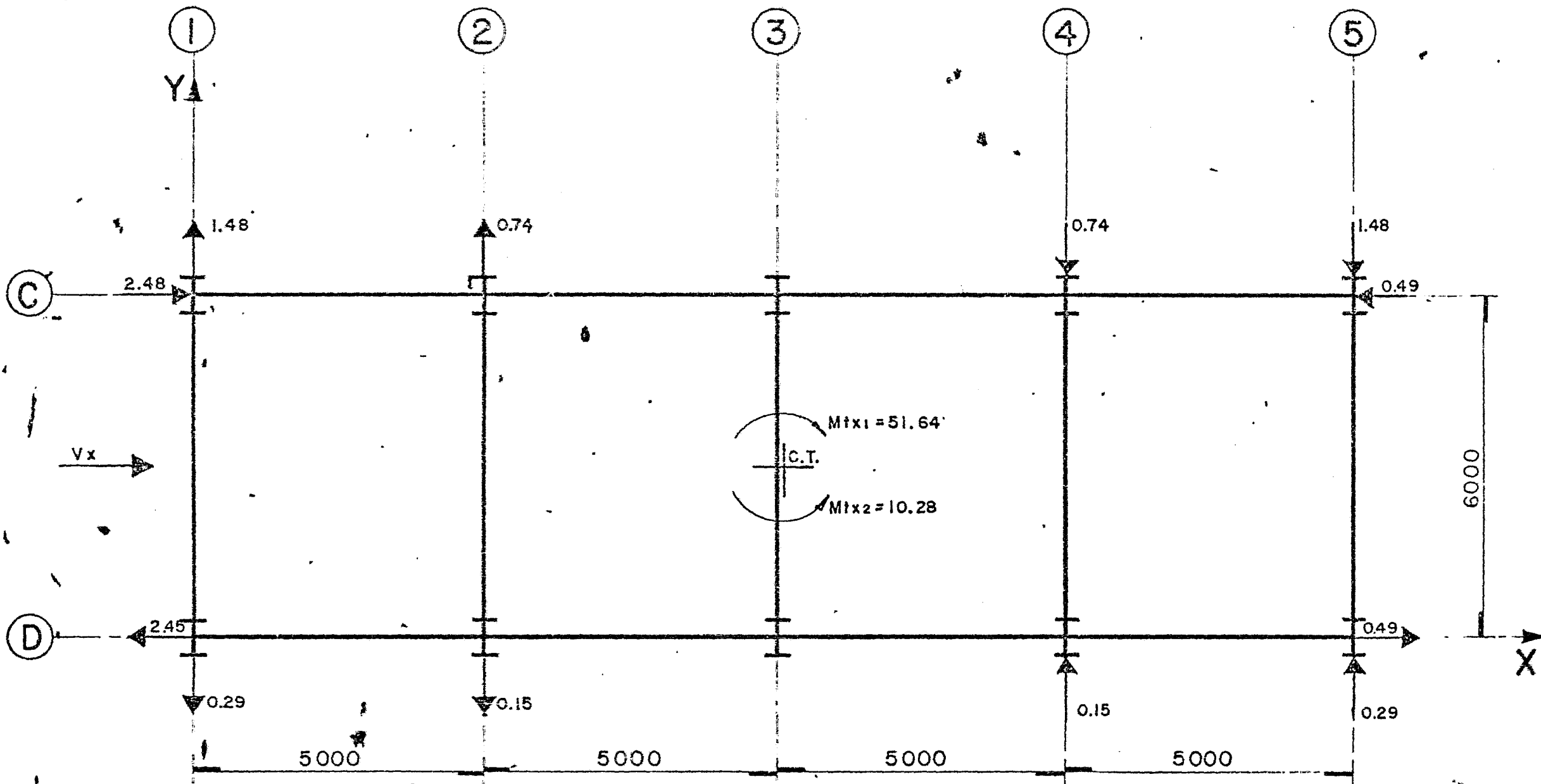
EJE	Kix	Yi	Kix Yi	Yit	Kix Yit	Kix Y <sup>2</sup> <sub>it</sub>	EFECTO DE Vx			EFFECTO DE Vy
							DIRECTO	TORSION	TOTAL	TORSION
C	46.15	6	276.9	3	138.45	415.35	22.35	2.45	24.80	4.79
D	46.15	0	0	-3	-138.45	415.35	22.35	0.49	22.84	-4.79

SUMAS 92.3

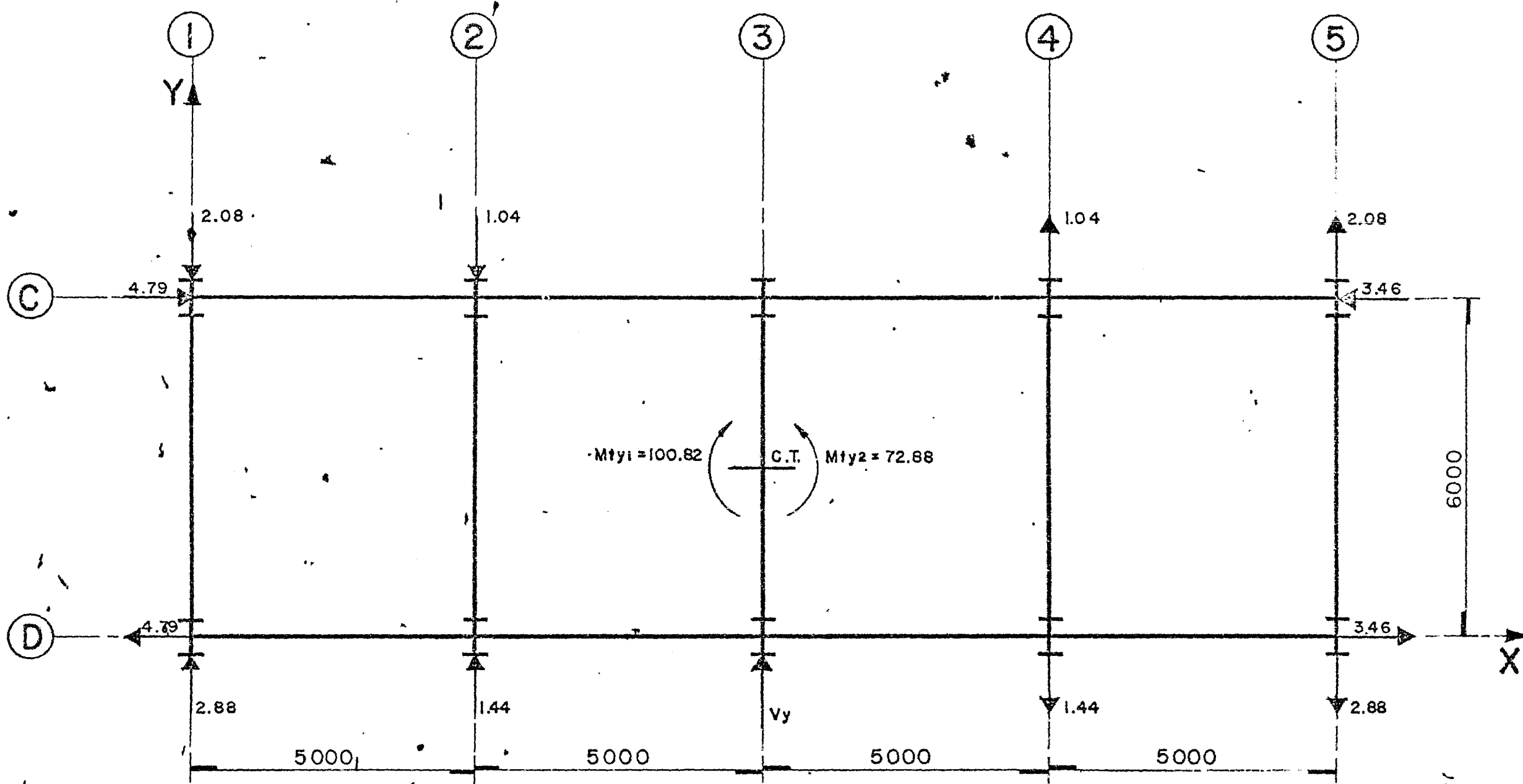
276.9

830.7

$$Y_T = \frac{276.9}{92.3} = 3$$



PLANTA NIVEL + 4



PLANTA NIVEL + 4

# DISTRIBUCION DE CORTANTES ENTRE LOS ELEMENTOS RESISTENTES EN CADA PISO

ENTREPISO: 4

$V_x = V_y = 28.53 \text{ TON.}$

$Y_v = 2.6 \text{ MTS.}$

$X_v = 8.32 \text{ MTS.}$

$e_x \text{ CAL} = 9.89 - 8.32 = 1.57 \text{ MTS.}$

$e_y \text{ CAL} = 3.0 - 2.6 = 0.40 \text{ MTS.}$

EJE	Kiy	Xi	Kiy-Xi	Xit	Kiy Xit	Kiy Xit <sup>2</sup>	EFECTO DE Vy			EFECTO DE Vx
							DIRECTO	TORSION	TOTAL	TORSION
1	7.41	0	0	-9.89	-73.29	724.79	5.83	3.63	9.46	-1.0
2	7.41	5	37.05	-4.89	-36.24	177.19	5.83	1.80	7.63	-0.49
3	7.14	10	71.40	0.11	0.79	0.09	5.62	0.02	5.64	0.01
4	7.14	15	107.10	5.11	36.49	186.44	5.62	0.67	6.29	0.50
5	7.14	20	142.80	10.11	72.19	729.79	5.62	1.32	6.94	0.98

SUMAS 36.24

358.35

1818.30

$$X_T = \frac{358.35}{36.24} = 9.89$$

$$M_{tx1} = 28.53 (0.4 - 0.1 \times 6); \quad M_{tx2} = 5.71 \text{ T.M.}$$

$$M_{tx2} = 28.53 (1.5 \times 0.4 + 0.1 \times 6); \quad M_{tx2} = 34.24 \text{ T.M.}$$

$$M_{ty1} = 28.56 (1.5 \times 1.57 + 0.1 \times 20); \quad M_{ty1} = 124.38 \text{ T.M.}$$

$$M_{ty2} = 28.56 (0.4 - 0.1 \times 20); \quad M_{ty2} = 45.70 \text{ T.M.}$$

$$\sum Kiy X_{it}^2 + \sum Kix Y_{it}^2 = 2510.58$$

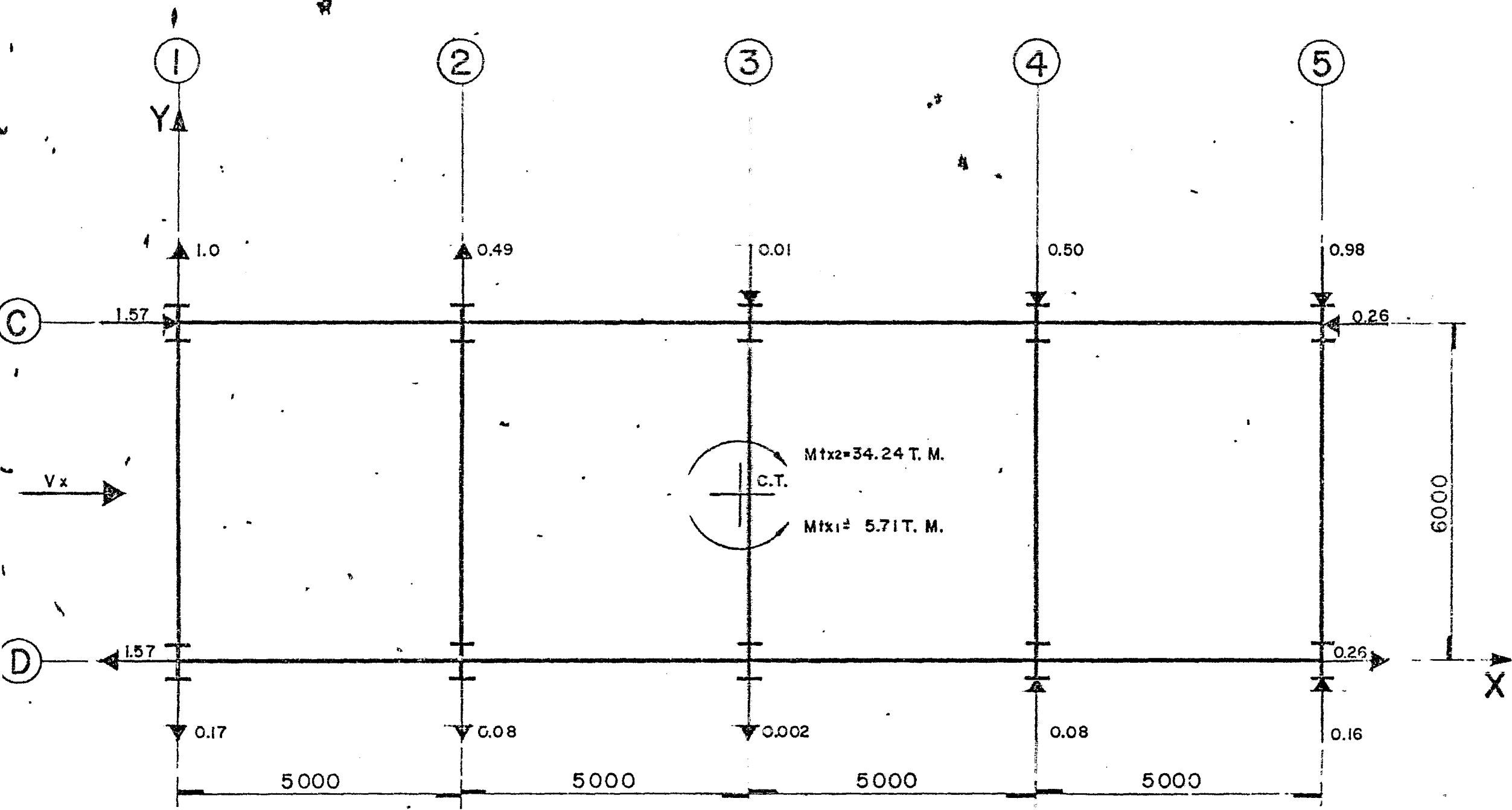
EJE	Kix	Yi	Kix Yi	Yit	Kix Yit	Kix Yit <sup>2</sup>	EFECTO DE Vx			EFECTO DE Vy
							DIRECTO	TORSION	TOTAL	TORSION
C	38.46	6	230.76	3	115.38	346.14	14.27	1.57	15.84	5.72
D	38.46	0	0	-3	-115.38	346.14	14.27	0.26	14.53	-5.72

SUMAS 76.92

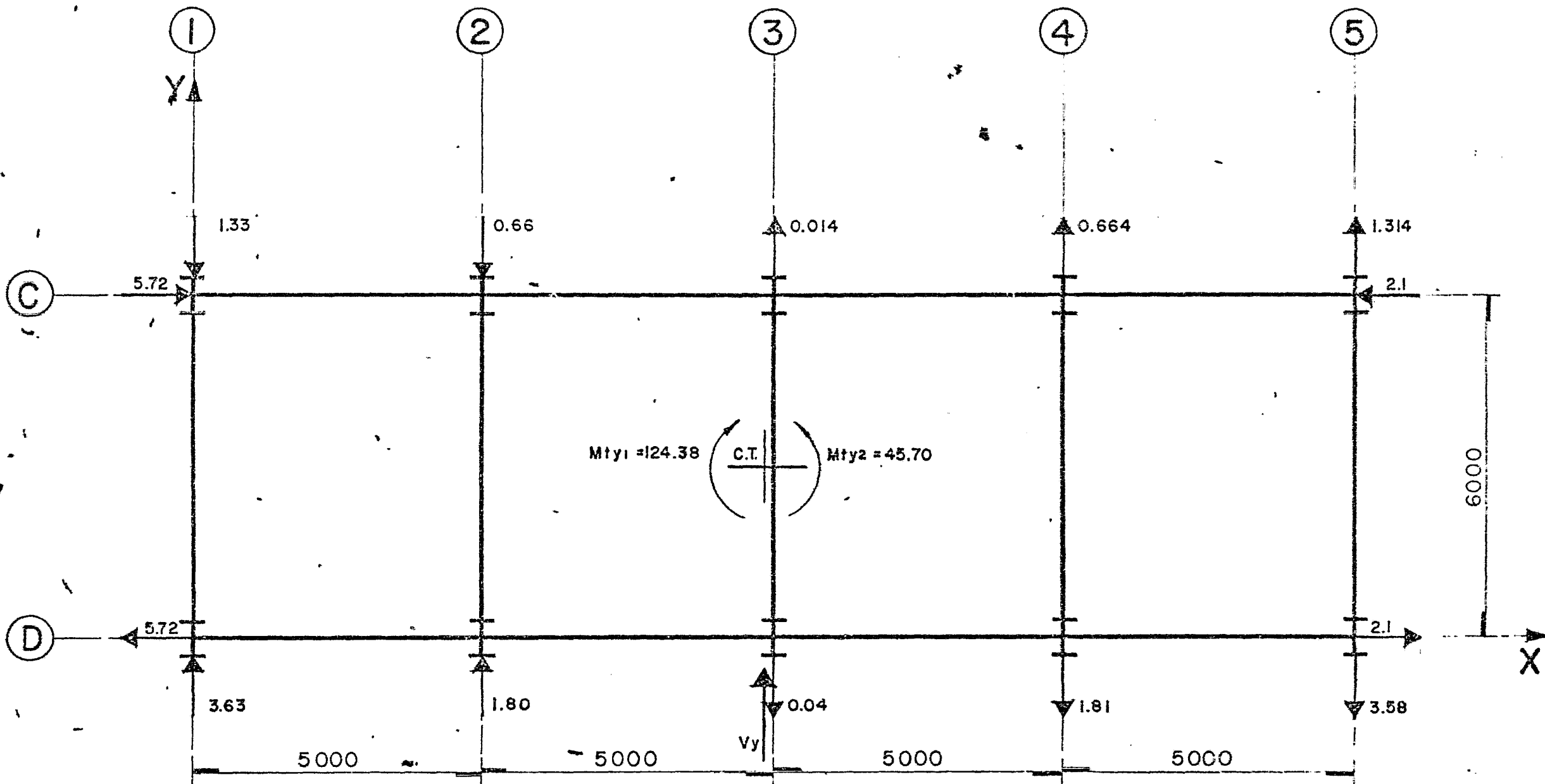
230.76

692.28

$$Y_T = \frac{230.76}{76.92} = 3.0$$



PLANTA NIVEL + 5



PLANTA NIVEL + 5



# DISTRIBUCION DE CORTANTES ENTRE LOS ELEMENTOS RESISTENTES EN CADA PISO

ENTREPISO: 5

$V_x = 5.86 \quad V_y = 3.12 \quad \text{CAL} = -2.5 + 2.33 = -0.17 \text{ MTS.}$   
 $V_y = 5.86 \quad X_v = 2.33 \quad \text{CAL} = 3.12 - 3.0 = 0.12 \text{ MTS.}$

EJE	Kiy	Xi	Kiy Xi	Xit	Kiy Xit	Kiy X <sup>2</sup> it	EFECTO DE Vy			EFECTO DE Vx
							DIRECTO	TORSION	TOTAL	TORSION
1	2.94	0	0	-2.5	-7.35	18.375	2.93	0.11	3.04	0.11
2	2.94	5	14.7	2.5	7.35	18.375	2.93	0.05	2.98	-0.11

SUMAS 5.88

14.7

36.75

$X_T = \frac{14.7}{5.88} = 2.5$

$M_{tx} = \frac{V_x}{e_y} (1.5 \times 0.12 + 0.10 \times 6); \quad M_{tx1} = 4.57 \text{ T-M}$   
 $M_{tx2} = -2.81 \text{ T-M}$

$M_{ty} = \frac{V_y}{e_x} (0.17 - 0.10 \times 5); \quad M_{ty1} = 4.42 \text{ T-M}$   
 $M_{ty2} = -1.93 \text{ T-M}$

$\sum Kiy X_{it}^2 + \sum Kix Y_{it}^2 = 36.75 + 257.22 = 293.97$

$V_{oiy} = \frac{Kiy}{\sum Kiy} V_y$

$V_{ijy} = \frac{Kiy X_{it}}{\sum Kiy X_{it}^2 + \sum Kix Y_{it}^2} M_{ty}$

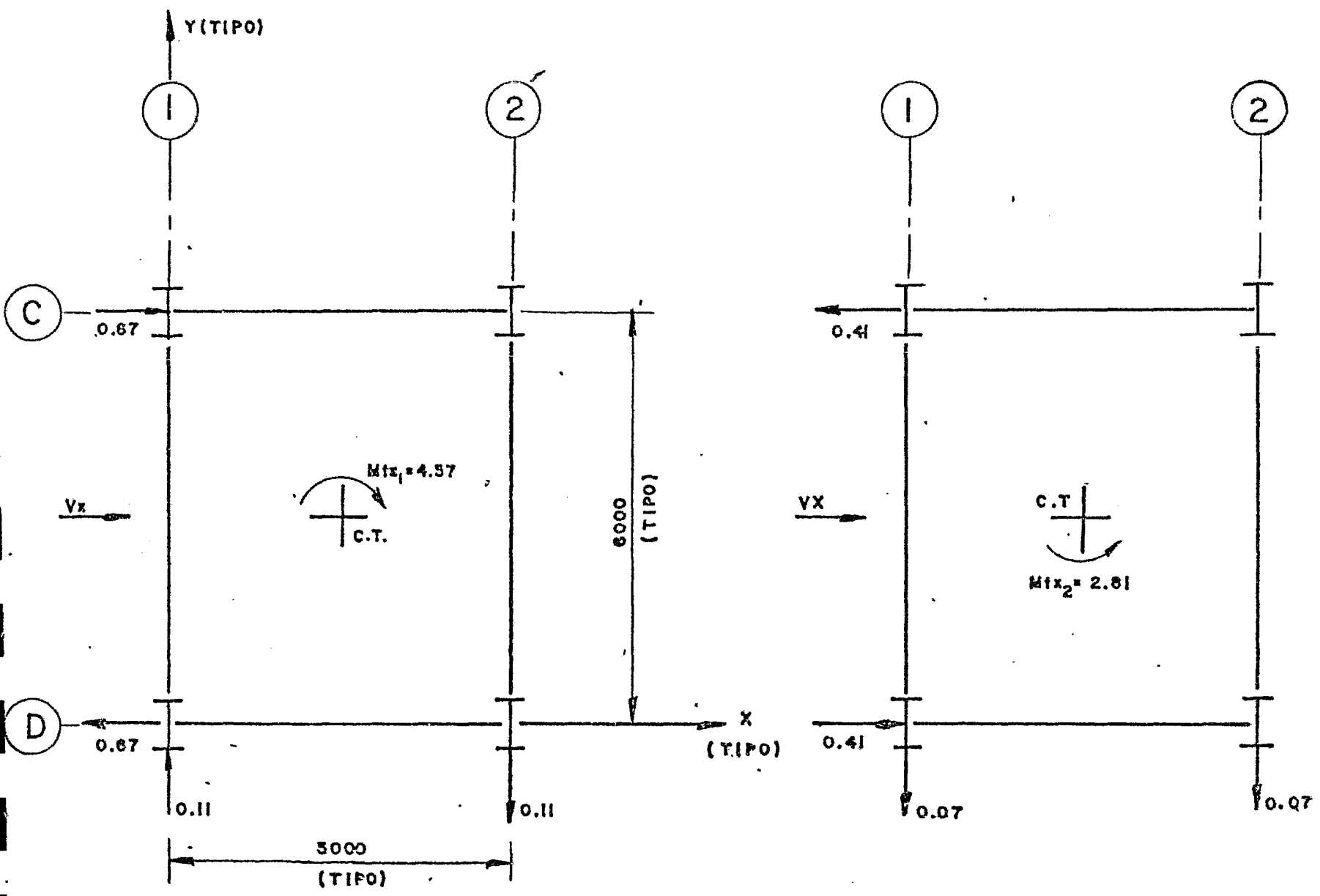
EJE	Kix	Yi	Kix Yi	Yit	Kix Yit	Kix Y <sup>2</sup> it	EFECTO DE Vx			EFECTO DE Vy
							DIRECTO	TORSION	TOTAL	TORSION
C	14.29	6	85.74	3	42.87	128.61	2.93	0.67	3.60	0.65
D	14.29	0	0	-3	-42.87	128.61	2.93	0.41	3.34	-0.65

SUMAS 28.58

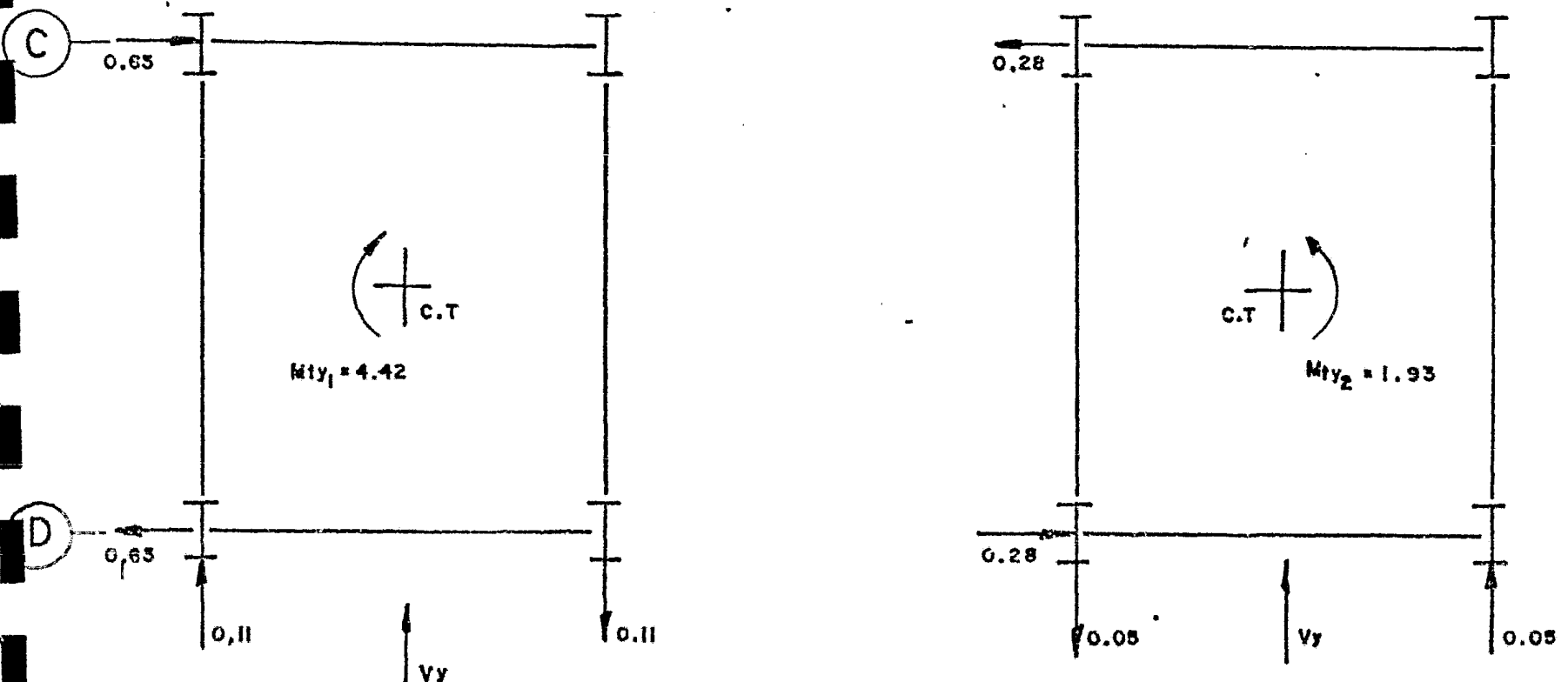
85.74

257.22

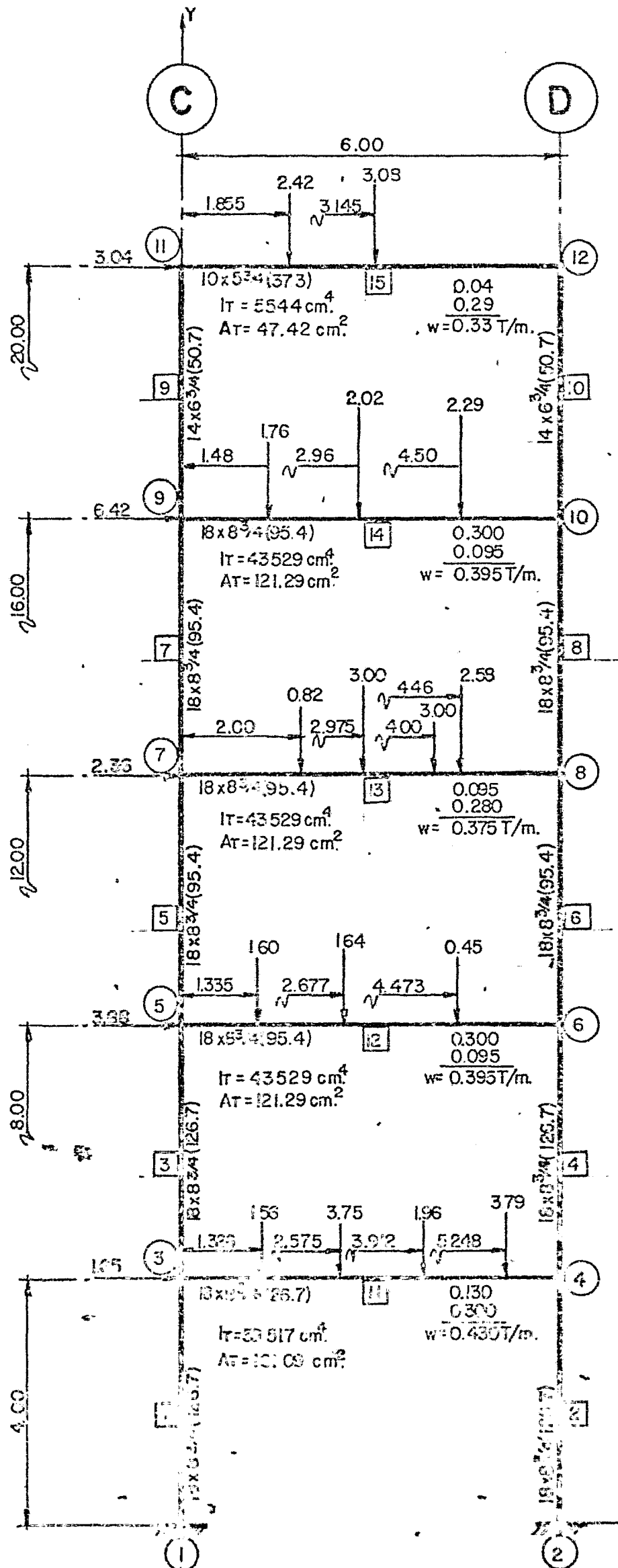
$Y_T = \frac{85.74}{28.58} = 3$



PLANTA NIVEL +6



# MARCO EJE I



$I_{col} = 14117 \text{ cm}^4$   
 $A_{col} = 64.52 \text{ cm}^2$

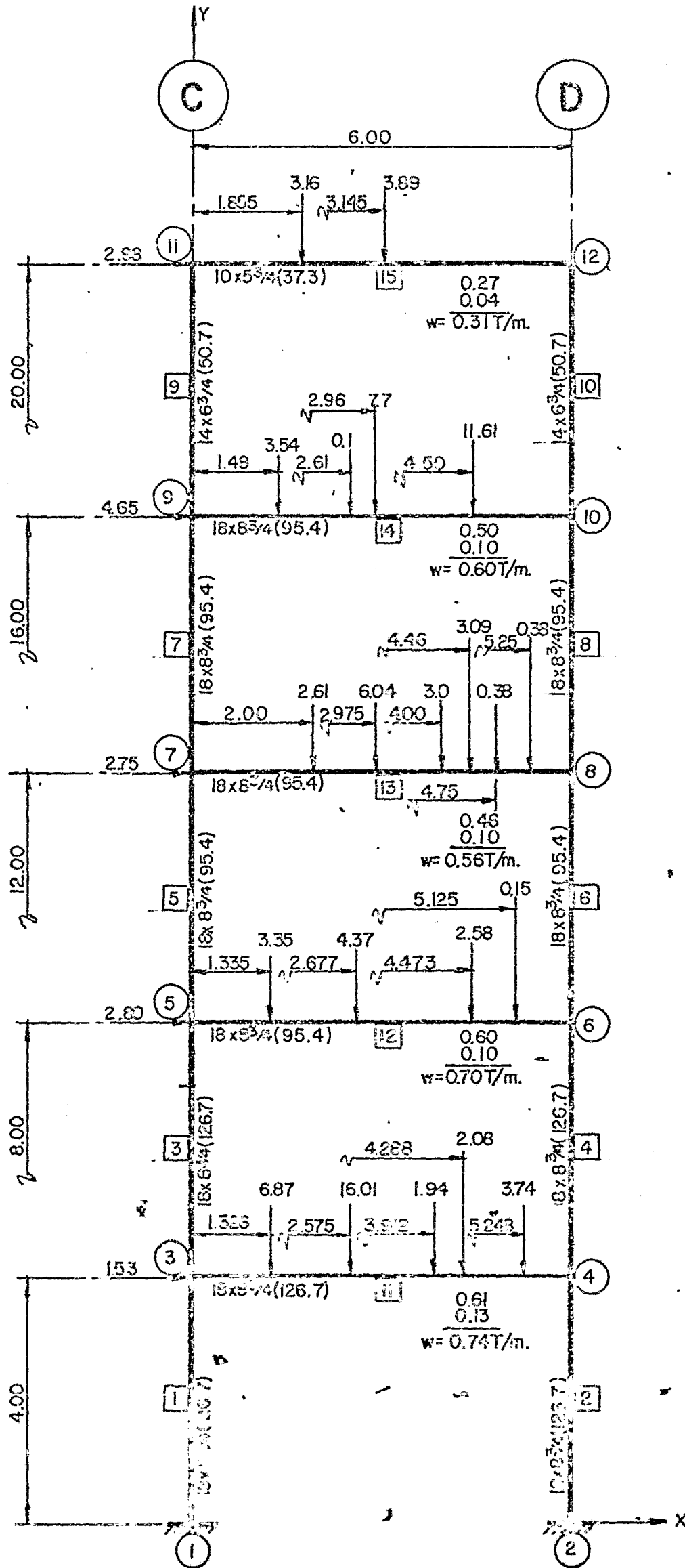
$I_{col} = 43529 \text{ cm}^4$   
 $A_{col} = 121.29 \text{ cm}^2$

$I_{col} = 43529 \text{ cm}^4$   
 $A_{col} = 121.29 \text{ cm}^2$

$I_{col} = 59517 \text{ cm}^4$   
 $A_{col} = 131.09 \text{ cm}^2$

$I_{col} = 59517 \text{ cm}^4$   
 $A_{col} = 131.09 \text{ cm}^2$

# MARCO EJE (2)









LISTADO DEL ARCHIVO DE DATOS: MARCO E1

ES 1 Y 2

6704244-8

07-11-84

JESUS PONCE S.

12 15 2 2100

1	0	0	0	0	0
2	0	0	0	6	0
3	1	1	1	0	4
4	1	1	1	6	4
5	1	1	1	0	8
6	1	1	1	6	8
7	1	1	1	0	12
8	1	1	1	6	12
9	1	1	1	0	16
10	1	1	1	6	16
11	1	1	1	0	20
12	1	1	1	6	20

1	1	3	1	1	161.09	59517
2	2	4	1	1	161.09	59517
3	3	5	1	1	161.09	59517
4	4	6	1	1	161.09	59517
5	5	7	1	1	121.29	43529
6	6	8	1	1	121.29	43529
7	7	9	1	1	121.29	43529
8	8	10	1	1	121.29	43529
9	9	11	1	1	64.52	14117
10	10	12	1	1	64.52	14117
11	3	4	1	1	161.09	59517
12	5	6	1	1	121.29	43529
13	7	8	1	1	121.29	43529
14	9	10	1	1	121.29	43529
15	11	12	1	1	47.42	5544

1  
CARGA MUERTA + EQUIPO EN OPERACION PARA  
EJE 1

0-7

11	.43	1.56	1.326	3.75	2.575	1.96	3.912
11	0	3.74	5.248	0	0	0	0
12	.4	1.6	1.335	1.64	2.671	.45	4.473
13	.38	.82	2.3	2.975	3	4	
13	0	2.58	4.46	0	0	0	0
14	.4	1.76	1.48	2.02	2.96	2.29	4.5
15	.33	2.42	1.855	3.08	3.145	0	0

2  
CARGA MUERTA + EQUIPO EN OPERACION PARA  
EJE 2

1 9

4 0 -7.7 0

11	.74	6.87	1.326	16.01	2.575	1.94	3.912
11	0	2.98	3.288	3.74	5.248	0	0
12	.6	3.35	1.335	4.37	2.677	2.50	4.473
12	0	.15	5.125	0	0	0	0
13	.5*	2.11	2.604	2.975	3	4	
13	0	3.09	4.46	.38	4.75	.38	5.25
14	.6	3.54	1.48	.1	2.61	7.7	2.96
14	0	11.61	4.5	0	0	0	0
15	.31	3.16	1.855	3.89	3.145	0	0



NUMERO DE NODOS : 12  
 NUMERO DE MIEMBROS : 15  
 NUMERO DE CARGAS : 2  
 MODULO DE ELASTICIDAD, MTONS/CM<sup>2</sup>: 2100

NODO XCOORD YCOORD XREL YREL MZREL

1	0.00	0.00000			
2	6.00	0.00000			
3	0.00	4.00111			
4	6.00	4.00111			
5	0.00	8.00111			
6	6.00	8.00111			
7	0.00	12.00111			
8	6.00	12.00111			
9	0.00	16.00111			
10	6.00	16.00111			
11	0.00	20.00111			
12	6.00	20.00111			

MIEMBRO LG. JT HI. JT LOW. FIX HI. FIX (CM)<sup>2</sup>

(CM) <sup>4</sup>	L. METER			
1	1	311	161.09	59517.00
4.00				
2	2	411	161.09	59517.00
4.00				
3	3	511	161.09	59517.00
4.00				
4	4	611	161.09	59517.00
4.00				
5	5	711	121.29	43529.00
4.00				
6	6	811	121.29	43529.00
4.00				
7	7	911	121.29	43529.00
4.00				
8	8	1011	121.29	43529.00
4.00				
9	9	1111	64.52	14117.00
4.00				
10	10	1211	64.52	14117.00
4.00				
11	3	411	161.09	59517.00
6.00				
12	5	611	121.29	43529.00
6.00				
13	7	811	121.29	43529.00
6.00				
14	9	1011	121.29	43529.00
6.00				
15	11	1211	47.42	5544.00
6.00				

GLOBAL STIFF MATRIX ROWS=31 COLS=9 ELEME  
NTS=279

CARGA MUEBTA + EQUIPO EN OPERACION PARA  
EJE-1

NODO NO.	X-DESPL CMETER	Y-DESPL CM
ETER	ROTACION GRADO	
1	0.0000	0.00

64	0.0133		
4	-0.0038		-0.02
95	-0.0115		
5	-0.0037		-0.04
81	0.0022		
6	-0.0038		-0.04
97	0.0001		
7	-0.0155		-0.06
66	0.0141		
8	-0.0149		-0.07
26	-0.0217		
9	-0.0314		-0.07
96	0.0034		
10	-0.0309		-0.08
46	-0.0057		
11	0.0239		-0.09
20	0.0259		
12	0.0139		-0.09
44	-0.0668		

MIEMBRO AXIAL CORTE COLILLA EN TO  
NS CORTE MOMENTO EN M-TON

NO.	FORCE TONS		LO.JT	HI.JT
	LO.JT	HI.JT		
1	-22.36	3.07	-1.18	1.18
2	-24.95	3.07	1.18	-1.18
3	-16.62	-3.19	-1.26	1.26
4	-17.10	1.91	1.26	-1.26
5	-13.04	-1.67	-1.18	1.18
6	-14.59	2.82	1.18	-1.18
7	-8.31	-3.27	-1.44	1.44
8	-7.64	2.52	1.44	-1.44
9	-4.19	-2.19	-1.66	1.66
10	-3.29	4.48	1.66	-1.66
11	0.08	-4.10	5.74	7.85
12	-0.08	6.55	3.58	2.51
13	0.26	3.10	4.73	6.95
14	0.22	6.32	4.11	4.36
15	-1.66	4.71	4.19	3.29
	-4.48	4.10		

FIN DE CASO

CARRA MUERTA + EQUIPO EN OPERACION PARA

EJE 2

NO.	X=DESPLE-DE FER	Y=DESPLE-EN
SECTOR	ROTACION GRADO	
1	0.0000	0.00
00	0.0000	
2	0.0000	0.00

4	0.0241	-0.07
14	-0.0343	
5	0.0642	-0.09
80	0.0089	
6	0.0593	-0.11
43	-0.0008	
7	0.0600	-0.13
66	0.0173	
8	0.0657	-0.16
10	-0.0247	
9	0.0244	-0.16
21	0.0399	
10	0.0207	-0.19
12	-0.0535	
11	0.0636	-0.17
71	0.0976	
12	0.0489	-0.20
25	-0.0627	

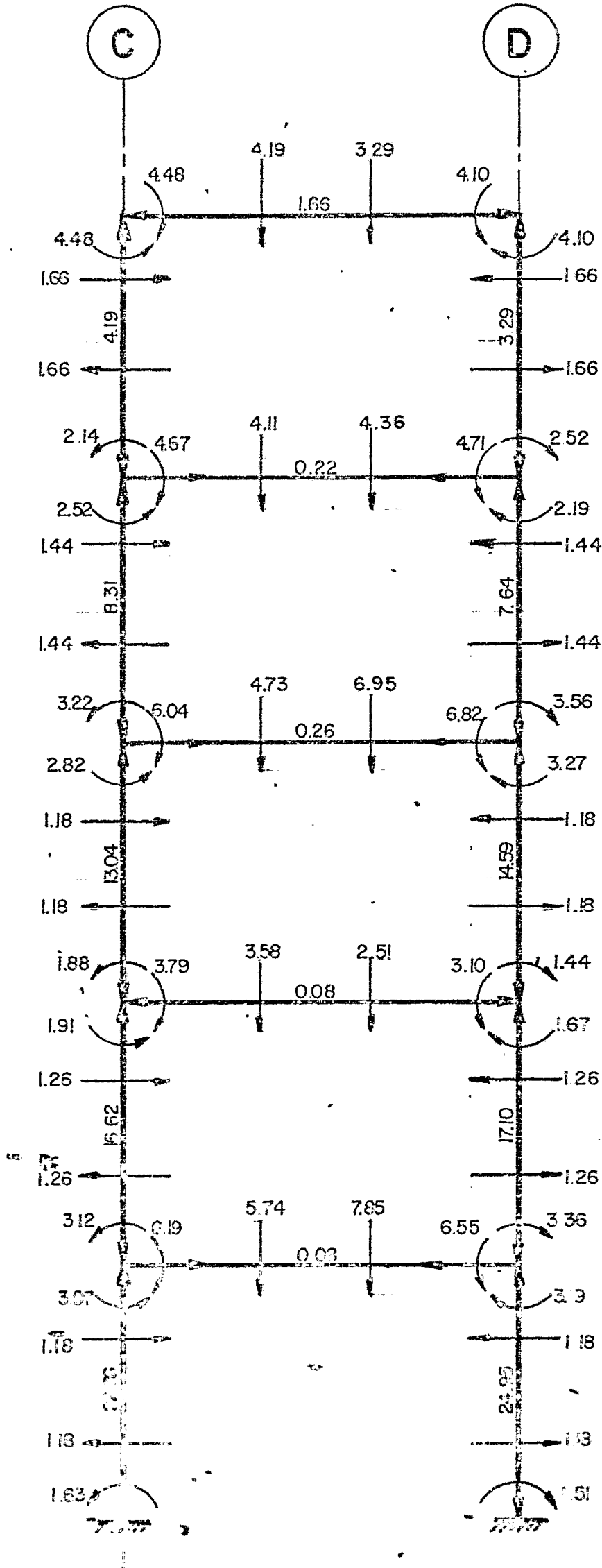
MIEMBRO AXIAL CORTE COLILLA EN TO  
NS CORTE MOMENTO EN M-TON

NO.	FORCE TONS		LO. JT	HI. JT
	LO. JT	HI. JT		
1	-50.75		-3.37	3.37
	4.13	9.36		
2	-60.40		3.37	-3.37
	-4.87	-8.62		
3	-32.13		-3.70	3.70
	9.53	5.26		
4	-36.24		3.70	-3.70
	-9.22	-5.57		
5	-24.57		-1.64	1.64
	2.94	3.61		
6	-29.75		1.64	-1.64
	-2.32	-4.23		
7	-16.24		-4.03	4.03
	7.16	8.97		
8	-19.22		4.03	-4.03
	-6.83	-9.29		
9	-5.07		-2.45	2.45
	4.16	5.65		
10	-3.84		2.45	-2.45
	-4.81	-5.00		
11	0.33		18.62	-16.46
	-18.89	17.84		
12	-2.06		7.56	6.49
	3.20	7.69		
13	2.40		8.34	10.52
	-10.77	11.06		
14	1.59		11.16	-15.09
	-13.12	14.10		
15	-2.45		5.07	3.84
	5.65	5.00		

FIN DE CASO

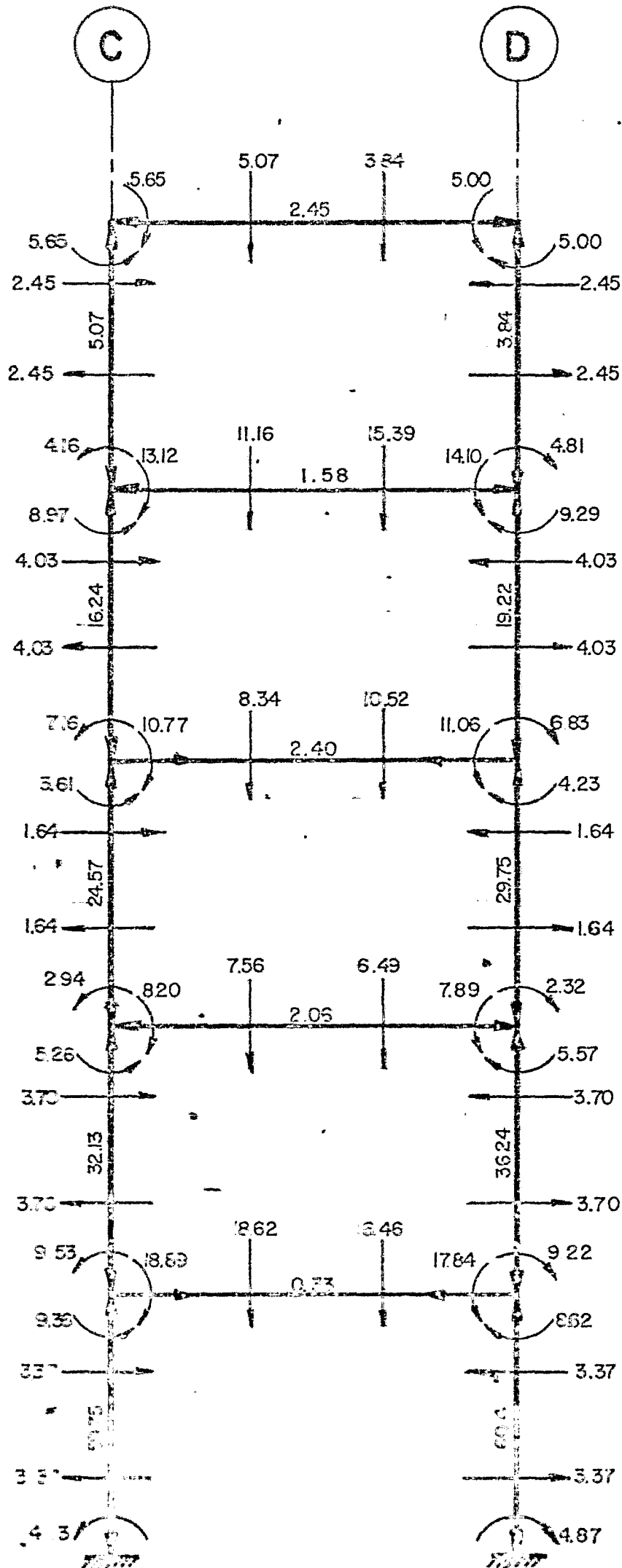
# MARCO EJE I

CARGA MUERTA + EQUIPO EN OPERACION

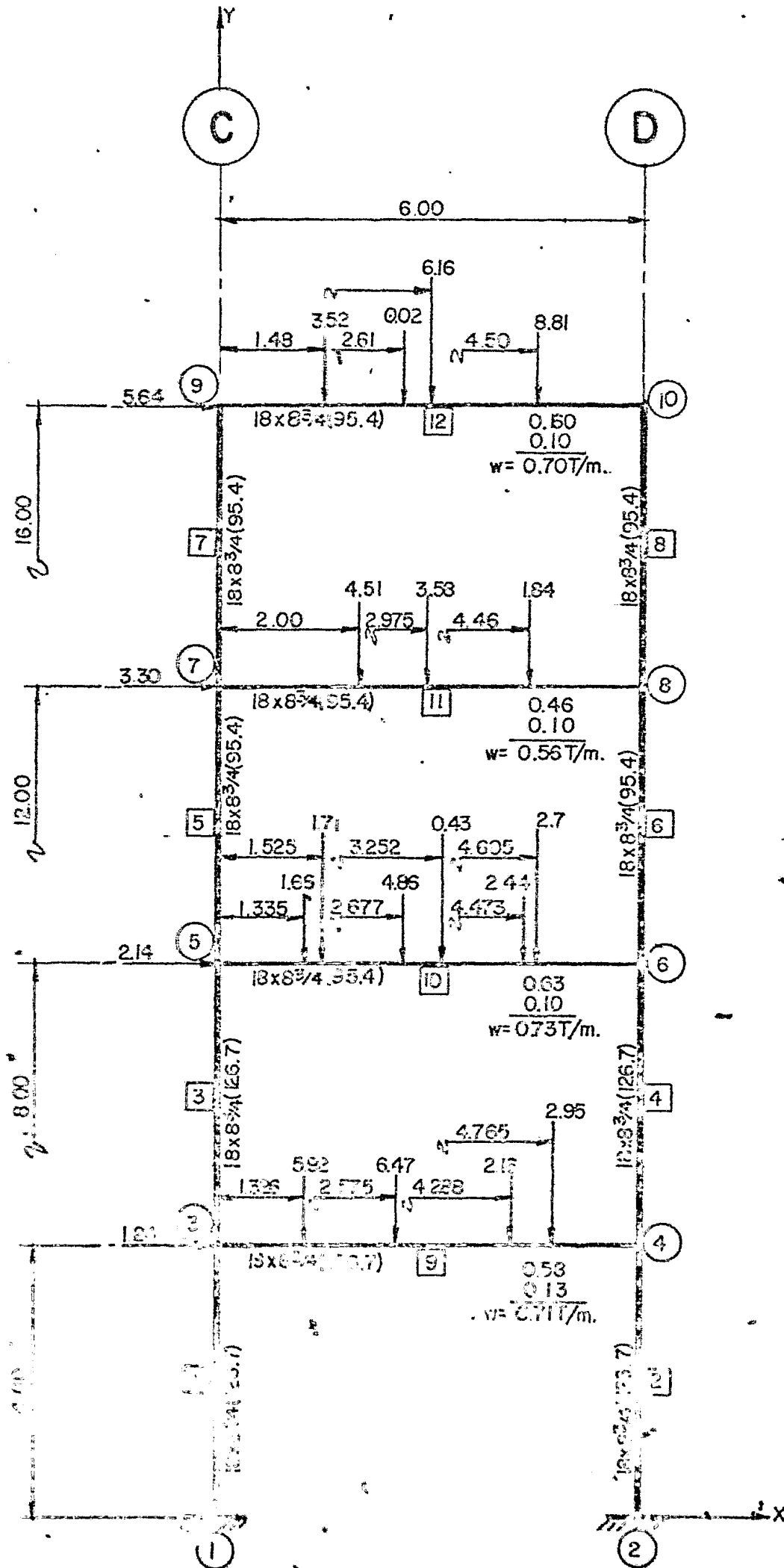


# MARCO EJE (2)

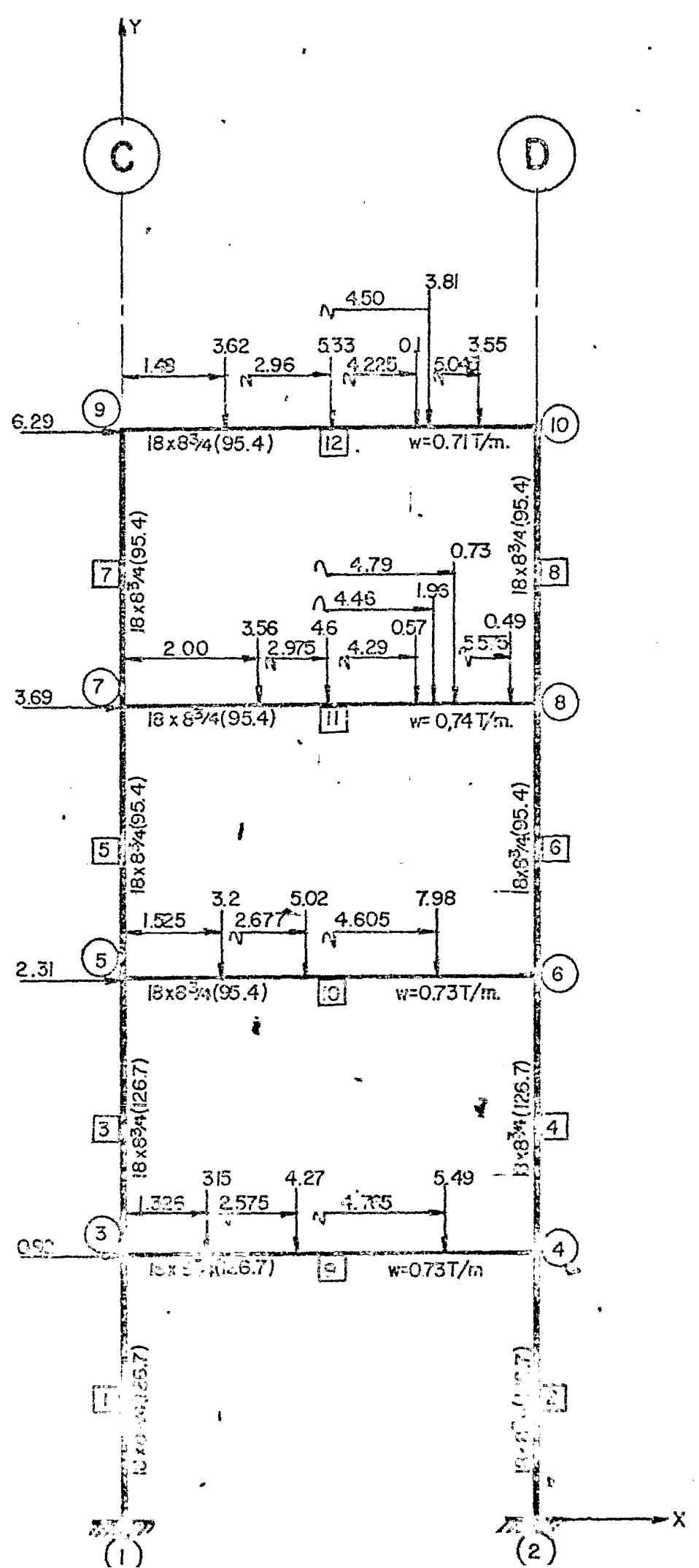
CARGA MUERTA + EQUIPO EN OPERACION



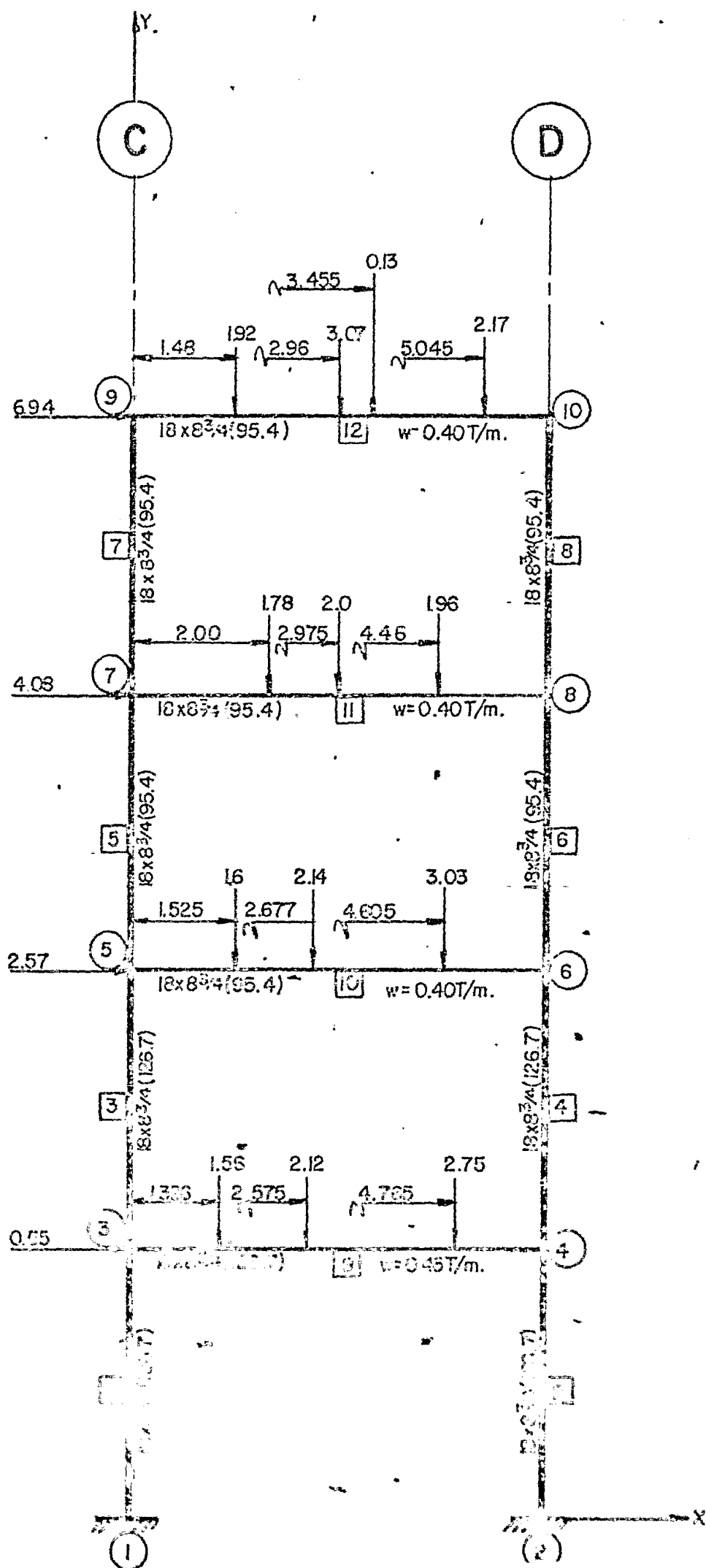
# MARCO EJE ③



# MARCO EJE ④



# MARCO EJE ⑤













~~LISTADO DEL ARCHIVO DE DATOS - MARCO EJ~~

ES 3,4 Y 5

6704904-8

09-11-84

JESUS PONCE S.

10 12 3 2100

1	0	0	0	0	0		
2	0	0	0	6	0		
3	1	1	1	0	4		
4	1	1	1	6	4		
5	1	1	1	0	8		
6	1	1	1	6	8		
7	1	1	1	0	12		
8	1	1	1	6	12		
9	1	1	1	0	16		
10	1	1	1	6	16		
1	1	3	1	1	161.09	59517	
2	2	4	1	1	161.09	59517	
3	3	5	1	1	161.09	59517	
4	4	6	1	1	161.09	59517	
5	5	7	1	1	121.29	43529	
6	6	8	1	1	121.29	43529	
7	7	9	1	1	121.29	43529	
8	8	10	1	1	121.29	43529	
9	3	4	1	1	161.09	59517	
10	5	6	1	1	121.29	43529	
11	7	8	1	1	121.2	43529	
12	9	10	1	1	121.29	43529	

1  
CARGA MUERTA + EQUIPO EN OPERACION PARA

EJE 3

0 7							
9	.71	5.92	1.326	6.47	2.575	2.16	4.288
9 0	2.95	4.765	0	0	0	0	0
10	.73	1.66	1.335	1.71	1.525	4.86	2.677
10 0	.43	3.252	2.44	4.473	2.7	4.605	
11	.56	4.51	2	3.56	2.975	1.84	4.46
12	.7	3.52	1.48	.02	2.61	6.16	2.96
12 0	8.81	4.5	0	0	0	0	0

2  
CARGA MUERTA + EQUIPO EN OPERACION PARA

EJE 4

0 6							
9	.73	3.15	1.326	4.27	2.575	5.49	4.765
10	.73	3.2	1.525	5.02	2.677	7.98	4.605
11	.74	3.56	2	4.6	2.975	.57	4.29
11 0	1.96	4.46	.73	4.79	.49	5.575	
12	.71	3.62	1.48	5.33	2.96	.1	4.225
12 0	3.81	4.5	3.55	5.045	0	0	0

3  
CARGA MUERTA + EQUIPO EN OPERACION PARA

EJE 5

0 5							
9	.43	1.56	1.336	2.12	2.575	2.75	4.765
10	.4	1.6	1.525	2.14	2.677	3.03	4.605
11	.4	1.78	2	2.975	1.86	4.46	
12	.4	1.92	1.48	3.07	2.96	.13	3.485
12 0	2.17	5.045	0	0	0	0	0

PROBLEMA DE: JESUS PONCE S.

NUMERO DE NODOS : 10  
 NUMERO DE MIEMBROS : 12  
 NUMERO DE CARGAS : 3  
 MODULO DE ELASTICIDAD, MTONS/CM2: 3100  
 NODO XCOORD YCOORD XREL YREL NZREL

1	0.00	0.00000
2	6.00	0.00000
3	0.00	4.00111
4	6.00	4.00111
5	0.00	8.00111
6	6.00	8.00111
7	0.00	12.00111
8	6.00	12.00111
9	0.00	16.00111
10	6.00	16.00111

MIEMBRO LO.JT HI.JT LOW.FIX HI.FIX (CM)2  
 (CM)4 L.METER

1	1	311	161.09	59517.00
2	2	411	161.09	59517.00
3	3	511	161.09	59517.00
4	4	611	161.09	59517.00
5	5	711	121.29	43529.00
6	6	811	121.29	43529.00
7	7	911	121.29	43529.00
8	8	1011	121.29	43529.00
9	3	411	161.09	59517.00
10	5	611	121.29	43529.00
11	7	811	121.20	43529.00
12	9	1011	121.29	43529.00

GLOBAL STIFF MATRIX ROWS=25 COLS=9 ELEME  
 NTS=225

CARGA MUERTA + EQUIPO EN OPERACION PARA  
 EJE 3

NODO NO.	X-DESPL CMETER	Y-DESPL CM
ETER	ROTACION GRADO	
1	0.0000	0.00
00	0.0000	
2	0.0000	0.00
00	0.0000	
3	0.0000	-0.04
45	0.0000	
4	0.0150	0.04
53	-0.0101	
5	0.0250	-0.07
62	0.0100	
6	0.0210	-0.07

7	0.0255	-0.10
19	0.0070	
8	0.0344	-0.10
84	0.0036	
9	0.0211	-0.11
76	0.0574	
10	0.0117	-0.12
84	-0.0694	

MIEMBRO AXIAL CORTE COLILLA EN TON  
 NS CORTE MOMENTO EN M-TON

NO.	FORCE TONS		LO.JT	HI.JT
	LO.JT	HI.JT		
1	-37.66	4.84	-1.76	1.76
2	-38.28	4.84	1.76	-1.76
3	-25.90	6.09	-3.18	3.18
4	-28.28	6.09	3.18	-3.18
5	-17.06	2.54	-1.52	1.52
6	-18.94	2.54	1.52	-1.52
7	-9.99	9.98	-3.98	3.98
8	-12.72	9.98	3.98	-3.98
9	1.42	10.58	11.76	10.00
10	-1.66	10.87	8.83	9.35
11	2.46	7.80	7.03	6.21
12	-3.98	10.58	9.99	12.72

FIN DE CASO

CARGA MUERTA + EQUIPO EN OPERACION PARA EJE 4

NODO NO.	X-DESPL CMETER	Y=DESPL CM
ITER	ROTACION GRADOS	
1	0.0000	0.00
00	0.0000	
2	0.0000	0.00
00	0.0000	
3	-0.0043	-0.04
05	0.0135	
4	-0.0012	-0.04
80	-0.0151	
5	0.0136	0.07
13	0.0189	
6	-0.0181	-0.08
54	0.0255	
7	-0.0327	-0.09
78	0.0105	
8	-0.0288	-0.11
69	-0.0117	
9	-0.0401	-0.11
81	0.0189	
10	-0.0438	-0.13
58	-0.0505	

MIEMBRO AXIAL CORTE COLILLA EN TON

NO.	FORCE TONS		LO.JT	HI.JT
	LO.JT	HI.JT		
1	-34.27		-1.20	1.20
	1.67	3.15		
2	-40.62		1.20	-1.20
	-1.59	-3.23		
3	-20.03		-2.93	2.93
	5.56	6.15		
4	-31.57		2.93	-2.93
	-5.28	-6.42		
5	-16.90		-2.05	2.05
	4.43	3.76		
6	-20.12		2.05	-2.05
	-4.64	-3.54		
7	-9.07		-3.68	3.68
	5.83	8.89		
8	-11.60		3.68	-3.68
	-5.61	-9.11		
9	1.72		8.24	-9.05
	-8.70	8.51		
10	-0.88		9.13	11.45
	-10.57	11.07		
11	1.63		7.83	8.52
	-9.59	9.15		
12	-3.68		9.07	11.60
	-8.89	9.11		

FIN DE CASO

CARGA MUERTA + EQUIPO EN OPERACION PARA EJE 5

NO.	X-DESPL CM	Y-DESPL CM
1	0.0000	0.00
00	0.0000	
2	0.0000	0.00
00	0.0000	
3	-0.0020	-0.02
02	0.0075	
4	-0.0007	-0.02
24	-0.0083	
5	-0.0053	-0.03
53	0.0083	
6	-0.0062	-0.03
92	-0.0100	
7	-0.0110	-0.04
86	0.0061	
8	-0.0091	-0.05
39	-0.0068	
9	0.0085	-0.05
59	0.0243	
10	-0.0127	-0.05
18	0.0239	

MIEMBRO AXIAL CORTE COLLILLA EN TON

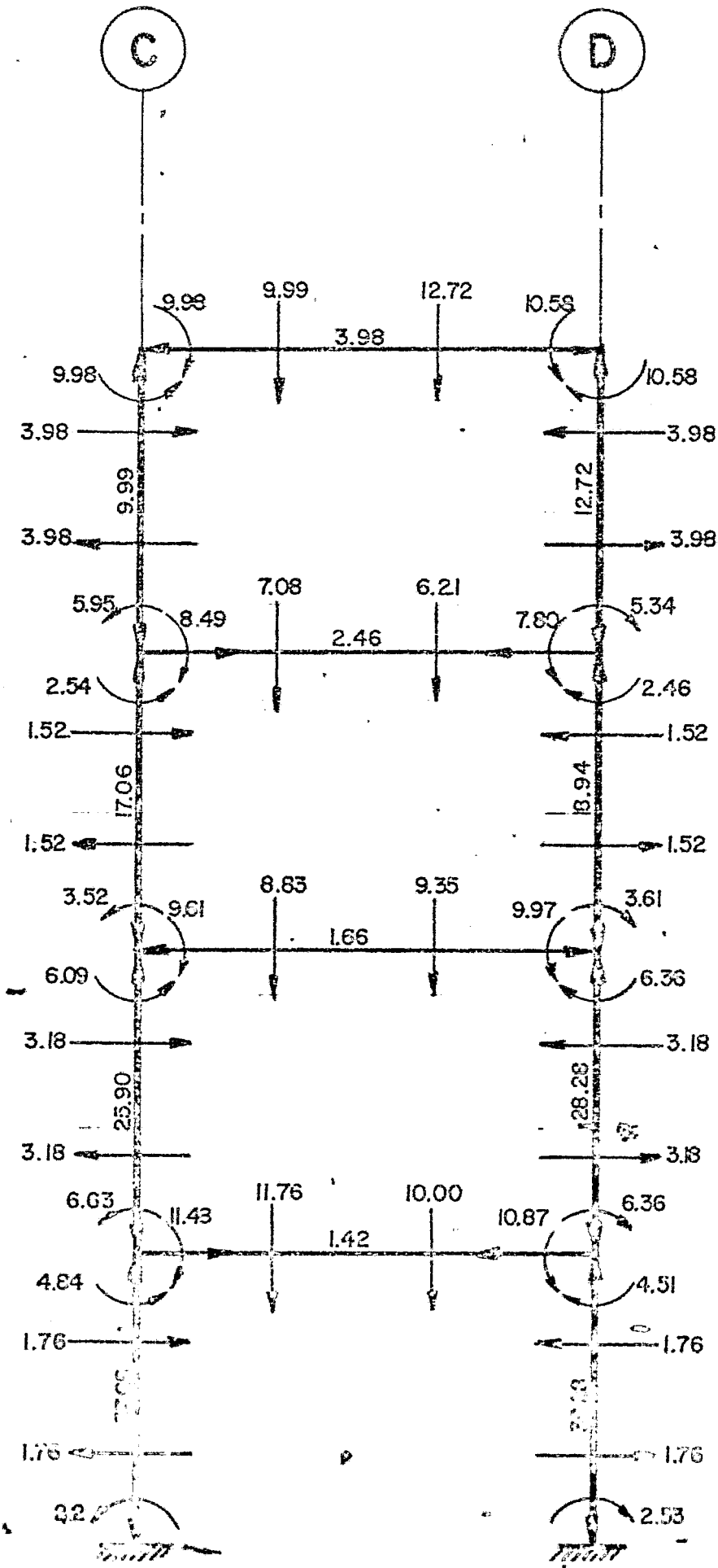
NO.	FORCE TONS		LO.JT	HI.JT
	LO.JT	HI.JT		
1	-17.06		-0.66	0.66
	0.91	1.73		
2	-18.95		0.66	-0.66
	-0.87	-1.73		
3	-12.77		-1.37	1.37
	2.70	2.78		
4	-14.23		1.37	-1.37



	2.06	1.83		
6	-9.32		0.96	-0.96
	-2.05	-1.79		
7	4.61		1.78	1.78
	2.83	4.33		
8	-5.08		1.78	-1.78
	-2.87	-4.33		
9	0.71		4.28	4.73
	-4.43	4.32		
10	-0.41		4.27	4.90
	-4.78	4.33		
11	0.82		3.90	4.24
	-4.66	-4.37		
12	-1.78		4.61	5.08
	-4.28	4.33		
FIN DE CASO				

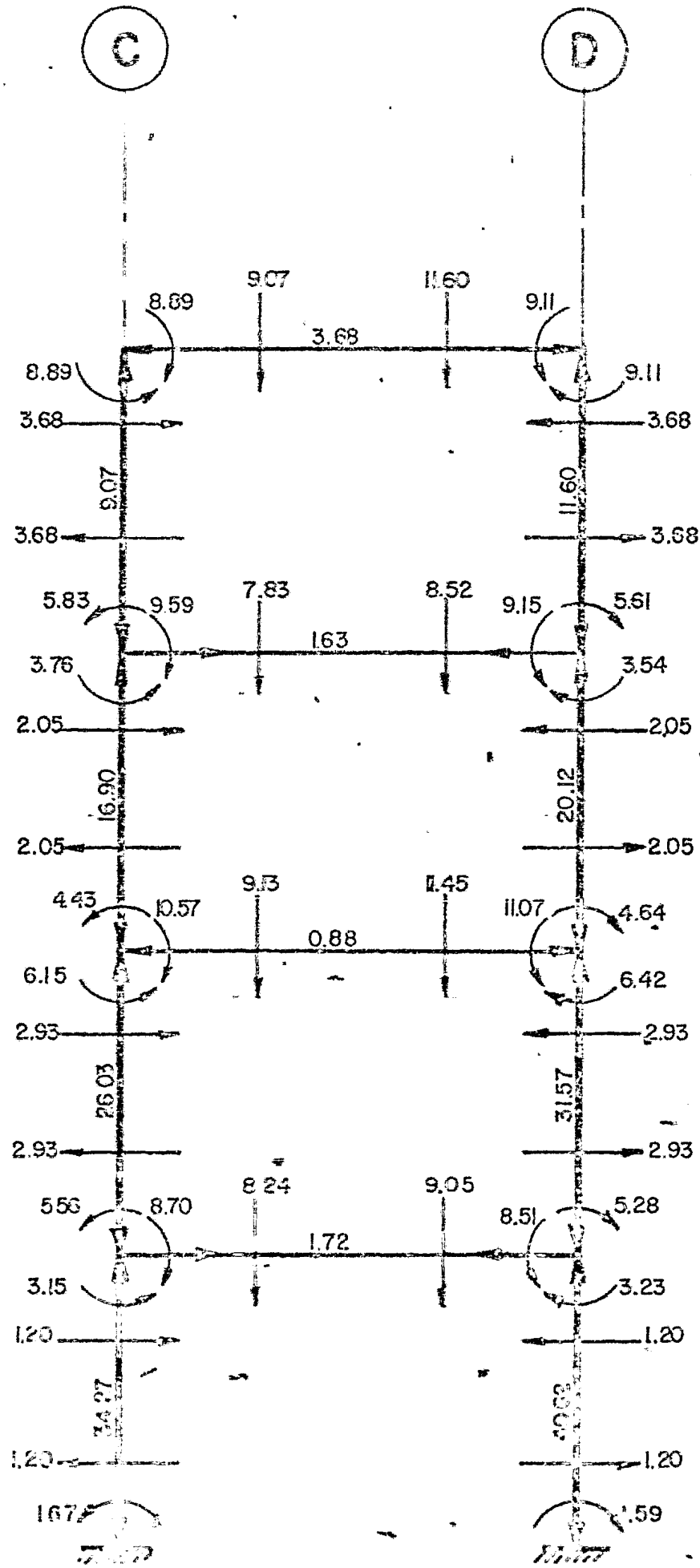
# MARCO EJE ③

CARGA MUERTA + EQUIPO EN OPERACION



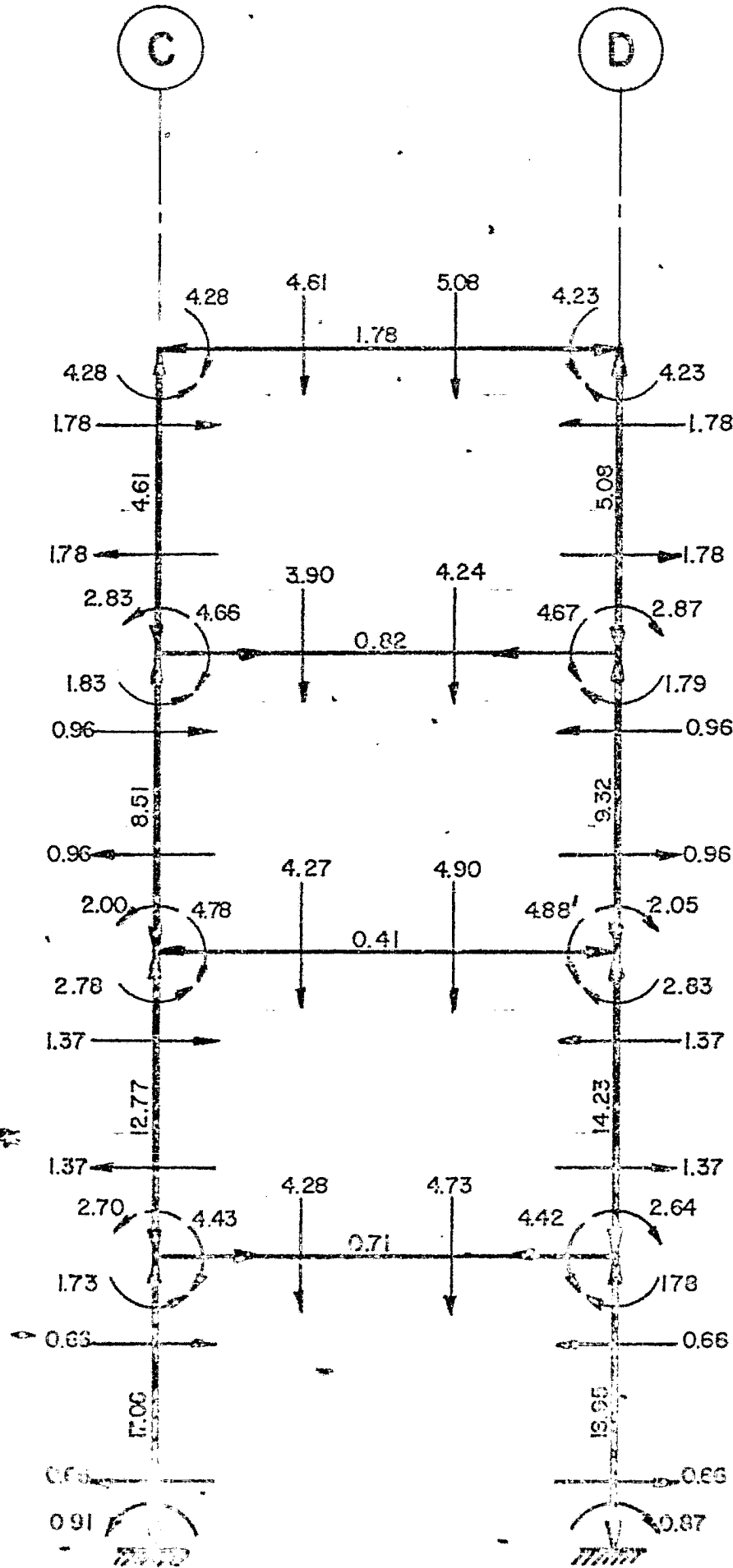
# MARCO EJE ④

CARGA MUERTA + EQUIPO EN OPERACION



# MARCO EJE (5)

CARGA MUERTA + EQUIPO EN OPERACION



LISTADO DEL ARCHIVO DE DATOS : MNMOS.EJ

E 1

6704944-8

28-10-83

PROYECTO DE: JESUS PONCE S

12	15	1	2100			
1	0	0	0	0	0	
2	0	0	0	0	0	
3	1	1	1	0	4	
4	1	1	1	0	4	
5	1	1	1	0	8	
6	1	1	1	0	8	
7	1	1	1	0	12	
8	1	1	1	0	12	
9	1	1	1	0	16	
10	1	1	1	0	16	
11	1	1	1	0	20	
12	1	1	1	0	20	
1	1	3	1	1	161.09	59517
2	2	4	1	1	161.09	59517
3	3	5	1	1	161.09	59517
4	4	6	1	1	161.09	59517
5	5	7	1	1	121.29	43529
6	6	8	1	1	121.29	43529
7	7	9	1	1	121.29	43529
8	8	10	1	1	121.29	43529
9	9	11	1	1	64.52	14117
10	10	12	1	1	64.52	14117
11	3	4	1	1	161.09	59517
12	5	6	1	1	121.29	43529
13	7	8	1	1	121.29	43529
14	9	10	1	1	121.29	43529
15	11	12	1	1	47.42	5544

1

SISMO

5	0		
3	1.65	0	0
5	3.88	0	0
7	2.36	0	0
9	6.42	0	0
11	3.04	0	0

6704944-8

28-10-83

PROYECTO DE: JESUS PONCE S

NUMERO DE NODOS	:	12			
NUMERO DE MIEMBROS	:	15			
NUMERO DE CARGAS	:	1			
MODULO DE ELASTICIDAD	:	3000000			
NODO	EXCENRO	YEXCENRO	XREL	YREL	ZREL

1	0.00	0.00000
2	0.00	0.00000
3	0.00	0.00111
4	0.00	0.00111
5	0.00	0.00111

6.11 8.20111  
 8.11 12.10111  
 10.11 16.10111  
 12.11 20.10111  
 14.11 24.10111

MEMBER	UNIT	HT	HT LCU.FIX	HT.FIX (CNR)
(CM)4		L. HE FLR		
1	1	311	161.09	59517.00
2	1	411	161.09	59517.00
3	7	511	161.09	59517.00
4	1	611	161.09	59517.00
5	3	711	121.29	43529.00
6	3	811	121.29	43529.00
7	7	911	121.29	43529.00
8	3	1011	121.29	43529.00
9	7	1111	64.52	14117.00
10	11	1211	64.52	14117.00
11	3	411	161.09	59517.00
12	3	611	121.29	43529.00
13	7	811	121.29	43529.00
14	3	1011	121.29	43529.00
15	11	1211	47.42	5544.00

GLOBAL STIFF MATRIX ROWS=31 COLS=9 ELEMENTS=279

BISMO

MEMBER NO.	X=DESPL CMETER	Y=DESPL CM
ETER	POTACION GRADE	
1	0.0000	0.00
00	0.0000	
2	0.0000	0.00
00	0.0000	
3	0.8364	0.03
55	0.1338	
4	0.8349	-0.03
55	0.1332	
5	2.2212	0.03
02	0.1370	
7	2.2137	-0.03
02	0.1369	
7	3.6526	0.03
02	0.1449	

99	0.1448	4.8131	0.08
10	0.1079	4.8056	-0.08
11	0.1078	5.9652	0.09
12	0.1439	5.9560	-0.09
13	0.1439		

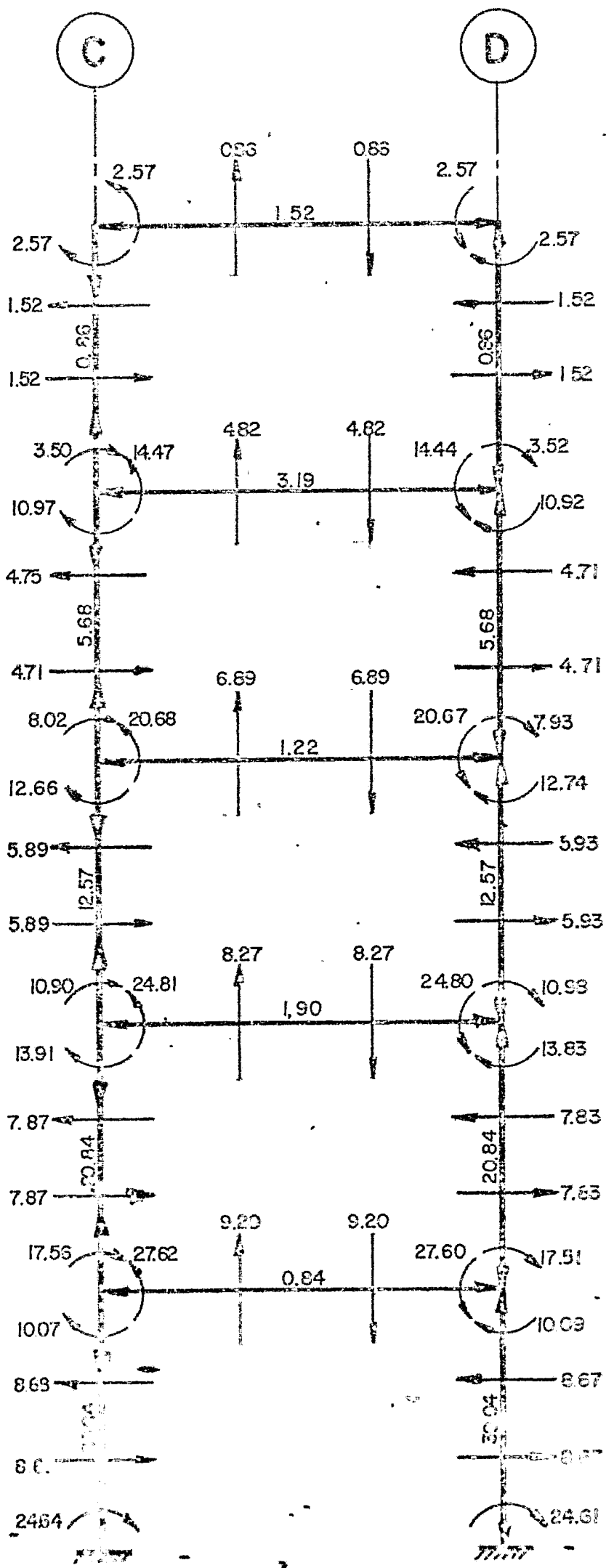
MIEMBROS FANTAL CORTE DE ILLA EN TON  
 NO. CORTE MOMENTO EN M-TON

NO.	FORCE TDMS	LD. JT	HI. JT
1	30.04	8.68	-8.68
	-24.81	-10.07	
2	30.04	8.67	-8.67
	-24.81	-10.09	
3	20.84	7.87	-7.87
	-17.81	-13.91	
4	20.84	7.88	-7.88
	-17.81	-13.83	
5	12.57	5.87	-5.87
	-10.87	-12.66	
6	12.57	5.93	-5.93
	-10.87	-12.74	
7	5.68	4.75	-4.75
	-8.81	-10.97	
8	5.68	4.71	-4.71
	-7.81	-10.92	
9	0.86	1.52	-1.52
	-3.57	-2.57	
10	-0.86	1.52	-1.52
	-3.52	-2.57	
11	-0.84	5.20	5.20
	27.62	27.60	
12	-1.90	8.27	8.27
	24.81	24.80	
13	-1.22	6.88	6.88
	20.88	20.67	
14	-3.19	4.82	4.82
	14.47	14.44	
15	-1.52	-0.86	0.86
	2.57	2.57	

FIN DE CORTA

# MARCO EJE I

SISMO





REPÚBLICA ARGENTINA DE BAYONA - MARCO 10

1  
2  
3

JESUS CRISTO S

12 1 2100

1 0 0 0 0 0

2 0 0 0 0 0

3 1 1 0 4

4 1 1 0 4

5 1 1 1 0 8

6 1 1 1 0 8

7 1 1 1 0 12

8 1 1 1 0 12

9 1 1 1 0 16

10 1 1 1 0 16

11 1 1 1 0 20

12 1 1 1 0 20

1 1 3 1 1 161.00 59517

2 2 4 1 1 161.00 59517

3 3 5 1 1 161.00 59517

4 4 6 1 1 161.00 59517

5 5 7 1 1 121.00 43529

6 6 8 1 1 121.00 43529

7 7 9 1 1 121.00 43529

8 8 10 1 1 121.00 43529

9 9 11 1 1 64.50 14117

10 10 12 1 1 64.50 14117

11 3 4 1 1 161.00 59517

12 5 6 1 1 121.00 43529

13 7 8 1 1 121.00 43529

14 9 10 1 1 161.00 43529

15 11 12 1 1 64.42 5844

1

SIGNO

5 0

3 1.53 0 0

5 2.3 0 0

7 2.79 0 0

9 4.25 0 0

11 2.56 0 0

6704944-B

23-10-83

FABRILENA DE JESUS CRISTO S

NUMERO DE NOMBRES : 15

NUMERO DE MINUTOS : 15

NUMERO DE CANTOS : 1

VELOCIDAD DE REPRODUCCION: 1.00

NOMBRE: XXXXXXX XXXXXX XXXL YRRA NIREI

1 0.00 0.00111

2 1.00 0.00111

3 2.00 0.00111

4 3.00 0.00111

5 0.00 0.00111

6	8.00	8.00111
7	8.00	12.00111
8	8.00	12.00111
9	8.00	16.00111
10	8.00	16.00111
11	8.00	20.00111
12	8.00	20.00111

MIEMBRO L2.JT HI.JT LOW.FIX HI.FIX 10002  
(CM)4 L.METER

1	1	511	161.07	59517.00
4.00				
2	2	411	161.07	59517.00
4.00				
3	3	511	161.07	59517.00
4.00				
4	4	611	161.07	59517.00
4.00				
5	5	711	121.29	43529.00
4.00				
6	6	811	121.29	43529.00
4.00				
7	7	911	121.29	43529.00
4.00				
8	8	1011	121.29	43529.00
4.00				
9	9	1111	64.52	14117.00
4.00				
10	10	1211	64.52	14117.00
4.00				
11	3	411	161.07	59517.00
6.00				
12	5	611	121.29	43529.00
6.00				
13	7	811	121.29	43529.00
6.00				
14	9	1011	121.29	43529.00
6.00				
15	11	1211	47.27	5544.00
6.00				

GLOBAL STIFF MATRIX ROWS=31 COLS=9 FRAME  
NTS=273

SISMO

NODO NO.	X-DESPL CMETER	Y-DESPL CM
EIFE	ROTACION GRADE	
1	0.0000	0.00
03	0.0000	
2	0.0000	0.00
03	0.0000	
3	0.7084	0.03
03	0.1189	
4	0.7071	-0.03
03	0.1187	
5	1.8812	0.05
14	0.1185	
6	1.8786	0.05
14	0.1183	
7	3.1117	0.03
04	0.1181	

08	0.1345	1035	-0.07
09	0.0821	082	0.07
10	0.0925	0814	-0.07
11	0.1125		0.07
12	0.1278		
13	5.1078		-0.07
14	0.1211		

MIFENRO AYIAL CORTI ECUILLA S. 70  
 DE CORRI DOMINIO EN M. FOR

NO.	PLAZA	TONS	LO. ST	HI. ST
	LB. ST	HI. ST		
1	23.43		7.33	-7.33
	-20.89	-8.53		
2	21.63		7.33	-7.33
	-20.85	-8.55		
3	17.85		6.80	-6.80
	-14.81	-11.58		
4	11.85		4.50	-4.50
	-14.77	-11.55		
5	10.77		5.18	-5.18
	-9.53	-11.14		
6	10.79		5.20	-5.20
	-9.53	-11.17		
7	4.94		3.82	-3.82
	-6.41	-3.87		
8	4.94		3.81	-3.81
	-6.38	-3.84		
9	0.81		1.49	-1.49
	-3.53	-2.43		
10	0.81		1.43	-1.43
	-3.53	-2.43		
11	-0.77		-7.78	7.78
	23.34	23.34		
12	-1.39		-7.06	7.06
	21.17	21.16		
13	-1.39		-5.85	5.85
	17.53	17.53		
14	-2.32		-4.13	4.13
	12.80	12.80		
15	-1.49		-0.81	0.81
	0.43	0.43		

FIN DE CARGA



8 3

24-NOV-82

JFS

10	1	1	1	0	16	
1	1	3	1	1	181.09	59517
2	2	4	1	1	181.09	59517
3	3	5	1	1	181.09	59517
4	4	6	1	1	181.09	59517
5	5	7	1	1	121.29	43529
6	6	8	1	1	121.29	43529
7	7	9	1	1	121.29	43529
8	8	10	1	1	121.29	43529
9	9	11	1	1	181.09	59517
10	10	12	1	1	121.29	43529
11	11	13	1	1	121.29	43529
12	12	14	1	1	121.29	43529

1

59517

3 1.24 0 0

5 2.14 0 0

7 1.2 0 0

9 3.24 0 0

24-NOV-82  
PROBLEMA DE ANEL

NUMERO DE NODOS : 10  
 NUMERO DE MEMBRAS : 12  
 NUMERO DE ANELAS : 1  
 CODIGO DE ACTIVIDAD: NODOS: 0 R: 2100  
 CODIGO DE ACTIVIDAD: ANEL: 0 R: 0 R: 0 R: 0

1	0.00	0.00000
2	0.00	0.00000
3	0.00	4.00000
4	0.00	4.00000
5	0.00	8.00000
6	0.00	8.00000
7	0.00	12.00000
8	0.00	12.00000
9	0.00	16.00000
10	0.00	16.00000

311 121.02 58317.00

1	2	311	121.02	58317.00
2	3	311	121.09	58317.00
3	4	311	121.19	58317.00
4	5	711	121.29	58317.00
5	6	811	121.29	58317.00
6	7	311	121.29	58317.00
7	8	1011	121.29	58317.00
8	9	411	121.29	58317.00
9	10	311	121.29	58317.00
10	11	311	121.29	58317.00
11	12	1011	121.29	58317.00

STIFF MATRIX ROWS=25 COLS=9 ELEME  
NTS=15

SPINE

NODE NO.	X=DESP. CM	Y=DESP. CM
ETFE	ROTATION	ROTATION
1	0.0000	0.00
00	0.0000	
2	0.0000	0.00
00	0.0000	
3	0.0943	0.02
31	0.0943	0.02
4	0.0943	0.02
31	0.0943	0.02
5	0.4189	0.03
31	0.4189	0.03
6	0.1187	-0.03
31	0.1187	-0.03
7	0.0964	0.04
04	0.0964	0.04
8	0.0962	-0.04
04	0.0962	-0.04
9	0.0558	0.05
31	0.0558	0.05
07	0.0558	-0.05
31	0.0558	-0.05

SPINE COORDINATE SYSTEM

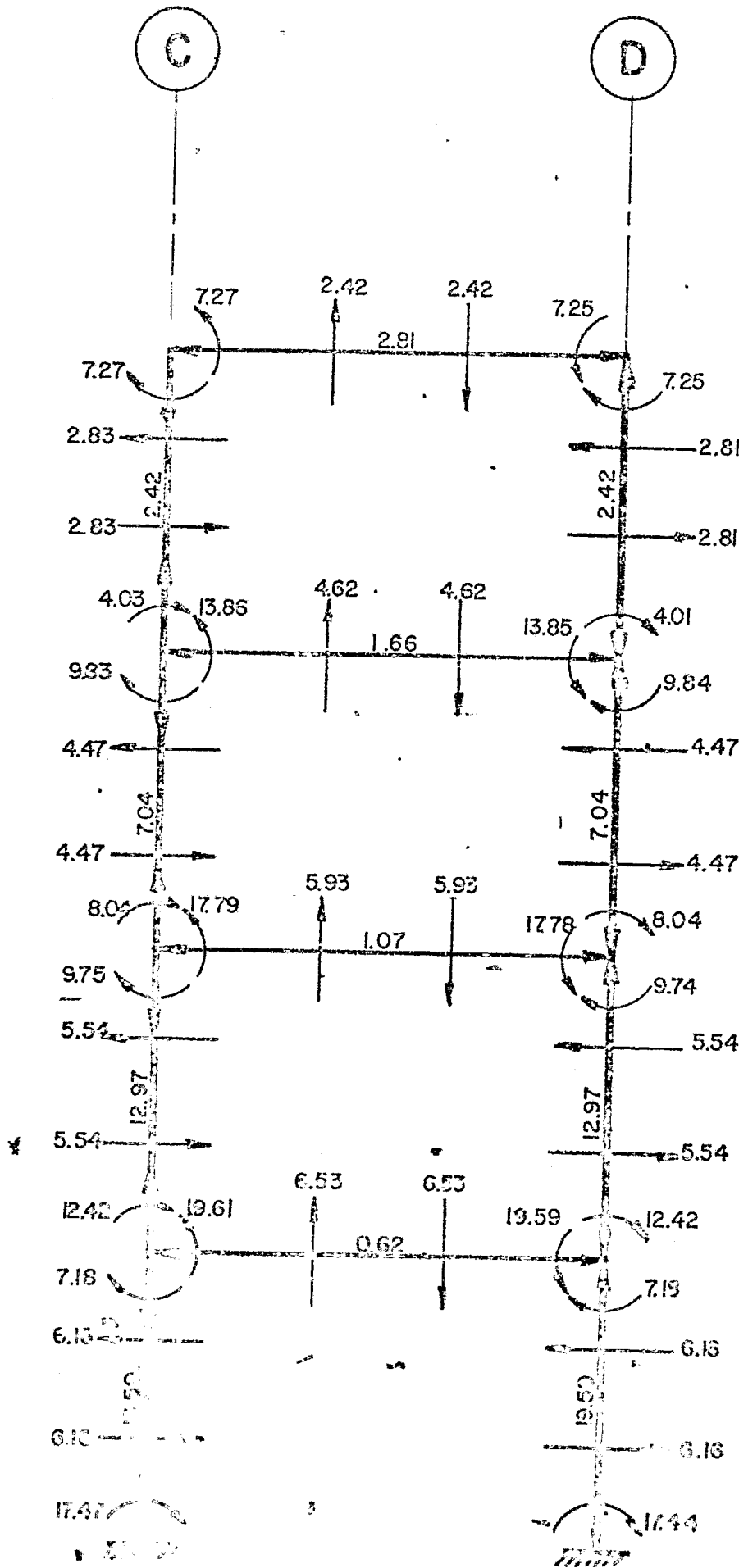
NODE NO.	X=DESP. CM	Y=DESP. CM
1	0.00	0.00
2	0.16	0.16
3	0.16	0.16
4	0.51	0.51
5	0.51	0.51
6	4.47	4.47
7	4.47	4.47

	8.04	2.84		
7	2.21	2.13		-2.83
	4.00	7.27		
	2.21	3.61		-2.81
8	4.01	2.25		2.03
	17.79	17.78		
10	4.77	3.93		5.93
11	17.79	17.78		
	4.62	4.62		
12	13.84	13.85		
	2.42	2.42		
	7.95	7.95		

FIN DE CASO

# MARCO EJE 3

SISMO





ESTADO DE CUENTAS DE CARGOS - MARCO E.C.

4

20-NOV-83

0.1

12 2100

1	0	0	0	0	0	
2	0	0	0	6	1	
3	1	1	1	0	4	
4	1	1	1	6	1	
5	1	1	1	0	5	
6	1	1	1	6	1	
7	1	1	1	0	12	
8	1	1	1	6	1	
9	1	1	1	0	13	
10	1	1	1	6	13	
1	1	3	1	1	121.09	59517
2	2	4	1	1	121.09	59517
3	3	5	1	1	121.09	59517
4	4	6	1	1	121.09	59517
5	5	7	1	1	121.29	48529
6	6	8	1	1	121.29	48529
7	7	9	1	1	121.29	48529
8	8	10	1	1	121.29	48529
9	9	4	1	1	121.09	59517
10	5	8	1	1	121.29	48529
11	7	8	1	1	121.29	48529
12	9	10	1	1	121.29	48529

61500  
4 0

3 2.00 0 0  
5 2.31 0 0  
7 3.69 0 0  
9 3.29 0 0

24-NOV-83  
PROBLEMA DEL JPE

NUMERO DE CUENTAS : 10  
NUMERO DE NUMEROS : 12  
NUMERO DE CARGAS : 1  
NUMERO DE BENEFICIARIOS/CMR: 2100  
NOMBRE ABOGADO YUBERA ABEL NOMBRE

1	0.00	0.00111
2	6.00	0.90111
3	0.00	4.00111
4	3.00	3.30111
5	0.00	3.00111
6	3.00	3.00111
7	0.00	3.00111
8	6.00	12.00111
9	0.00	12.00111
10	3.00	12.00111

NUMERO DE CUENTAS HIJOS HIJOS HIJOS (CMR)

1	4.00	2	411	161.07	39517.00
2	4.00	3	511	161.07	39517.00
3	4.00	4	611	161.07	39517.00
4	4.00	5	711	121.29	43529.00
5	4.00	6	811	121.29	43529.00
6	4.00	7	911	121.29	43529.00
7	4.00	8	1011	121.29	43529.00
8	4.00	9	411	161.07	39517.00
9	6.00	10	511	121.29	43529.00
10	6.00	11	611	121.29	43529.00
11	6.00	12	1011	121.29	43529.00
12	6.00				

GLOBAL STIFF MATRIX ROWS=25 COLS=9 ELEMENTS=225

ELENO	ROTA NO.	X-DESPL	Y-DESPL
MEMB	ROTACION	GRADO	CM
1		0.0000	0.00
00		0.0000	
2		0.0000	0.00
00		0.0000	
3		0.6423	0.02
05		0.1032	
4		0.6415	-0.02
55		0.1030	
5		1.7255	0.04
22		0.1317	
6		1.7238	-0.04
38		0.1317	
7		2.8520	0.05
49		0.1074	
8		2.8477	-0.05
47		0.1071	
9		3.6280	0.05
91		0.0318	
10		3.6206	-0.05
91		0.0318	

MIEMBRO	ANIL	CONTE	DELLA	EN TO
NO.	FORTE	TOUS	LO.UT	MI.UT
	LO.UT	MI.UT		
1	11.87	8.61	-8.61	
	-18.04	-7.55		
2	-11.37	8.60	-8.60	
	-10.00	8.00		
3	12.41	8.61	-8.61	
	-13.65	-10.71		
4	-14.27	8.61	-8.61	
	-12.11	-10.71		
5	7.80	4.90	-4.90	
	-9.01	-15.15		
6	-7.80	4.90	-4.90	

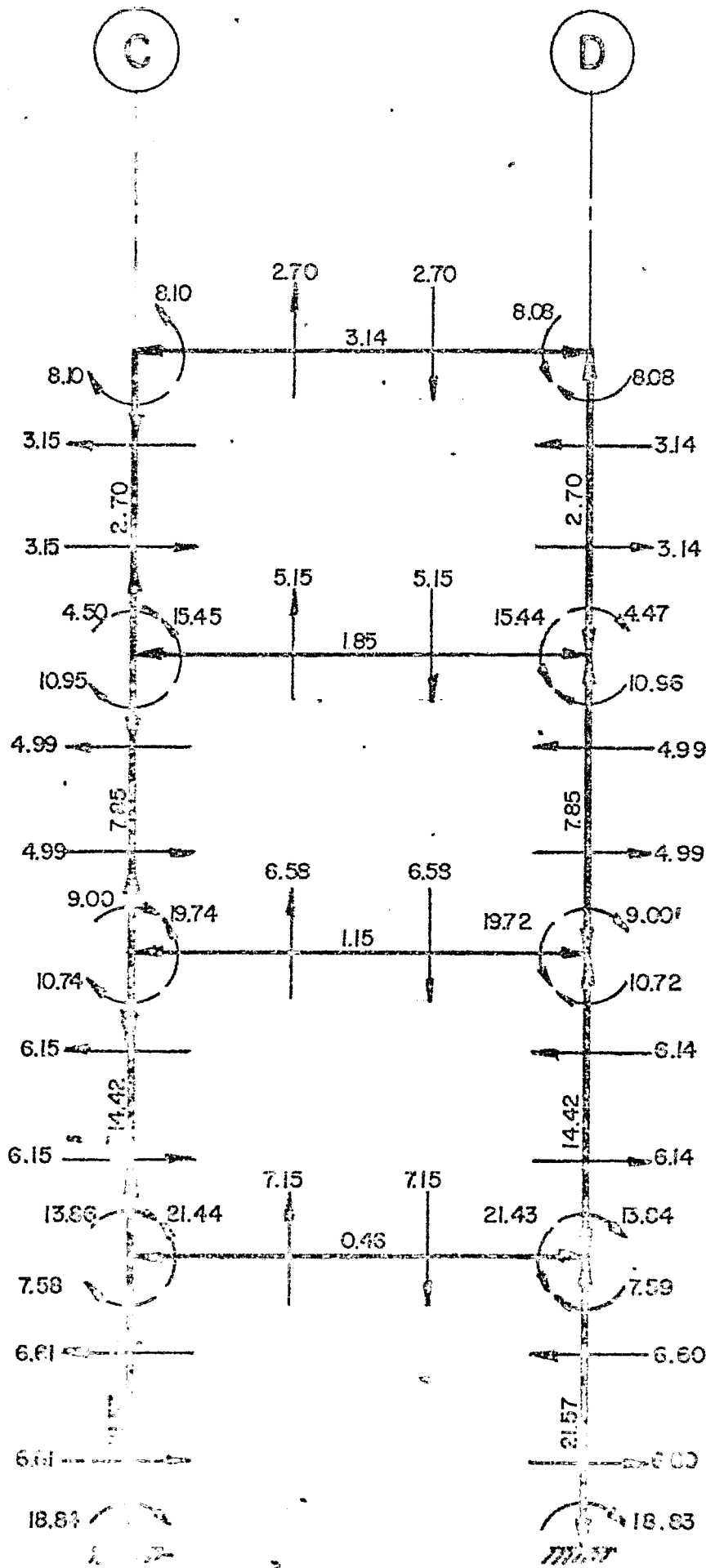
	3.70	3.15	-3.15
	4.10	3.14	-3.14
	4.40	7.18	7.18
	21.43	6.50	6.50
	18.71	5.15	5.15
	15.41	2.70	2.70
	8.10	8.05	

APR 1967

# MARCO EJE

4

## SISMO



LISTADO DEL ARCHIVO DE DATOS : HANCO BI

5

21-NOV-87

CPS						
10	10	1	2100			
1	0	0	0	0	0	
2	0	0	0	0	0	
3	1	1	1	0	4	
4	1	1	1	0	4	
5	1	1	1	0	8	
6	1	1	1	0	8	
7	1	1	1	0	12	
8	1	1	1	0	12	
9	1	1	1	0	16	
10	1	1	1	0	16	
1	1	1	1	1	161.09	59517
2	2	1	1	1	161.09	59517
3	3	1	1	1	161.09	59517
4	4	1	1	1	161.09	59517
5	5	7	1	1	121.29	43529
6	6	8	1	1	121.29	43529
7	7	1	1	1	121.29	43529
8	8	10	1	1	121.29	43529
9	8	4	1	1	161.09	59517
10	5	1	1	1	121.29	43529
11	7	8	1	1	121.29	43529
12	9	10	1	1	121.29	43529

SISMO

4	0
5	2.57 0 0
7	4.08 0 0
8	3.94 0 0

24-NOV-83

PROBLEMA Inter-CPS

NUMERO DE NOMBRES : 10  
 NUMERO DE MIEMBROS : 12  
 NUMERO DE CARGAS : 1  
 NOMBRE DE ELASTICIDAD, MITOS Y DIFER: 1000  
 NOMBRE DE ELASTICIDAD, MITOS Y DIFER: 1000

1	0.00	0.00000
2	0.00	0.00000
3	0.00	4.00000
4	0.00	4.00000
5	0.00	8.00000
6	0.00	8.00000
7	0.00	12.00000
8	0.00	12.00000
9	0.00	16.00000
10	0.00	16.00000

4.00

4.00

4.00

4.00

4.00

4.00

4.00

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4.00

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4.00

4.00

4.00

4.00

4.00

4.00

4.00

411 131.09 13517.00

511 131.09 13519.00

611 131.09 13517.00

711 131.29 13529.00

811 131.29 13529.00

911 131.29 13529.00

1011 131.29 13529.00

411 131.09 13517.00

611 131.29 13529.00

811 131.29 13529.00

1011 131.29 13529.00

GLOBAL STIFF MATRIX ROWS=25 COLS=9 ELEME NTS=825

RESULTS

MEMB NO.	X-DESPL	Y-DESPL	Z-DESPL	ROTATION
1	0.0000	0.0000	0.0000	0.00
2	0.0000	0.0000	0.0000	0.00
3	0.6987	0.1126	0.0000	0.02
4	0.6742	0.1124	0.0000	-0.02
5	1.8958	0.1454	0.0000	0.04
6	1.8958	0.1451	0.0000	-0.04
7	3.1735	0.1185	0.0000	0.06
8	3.1737	0.1182	0.0000	-0.06
9	3.5734	0.0682	0.0000	0.08
10	3.5734	0.0682	0.0000	-0.08

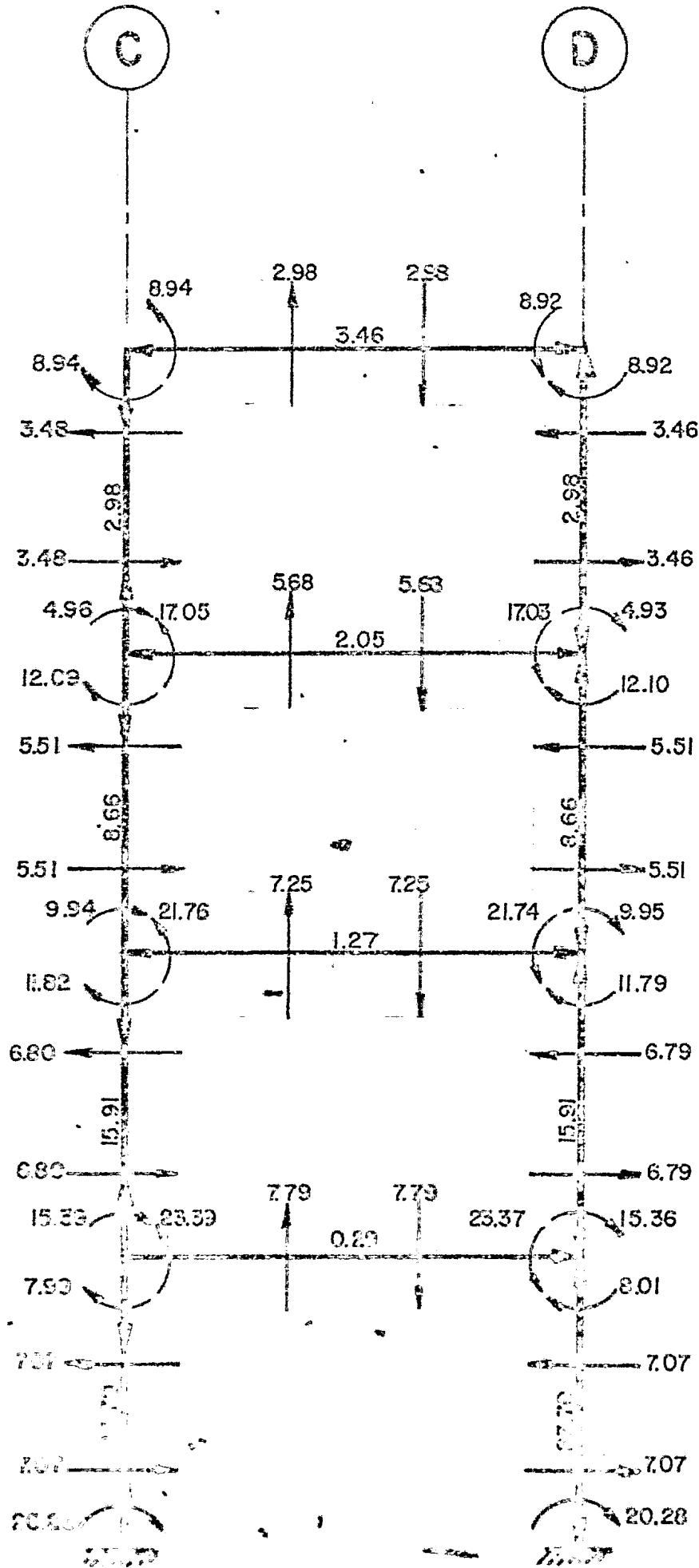
MEMBER COORDINATE SYSTEMS

MEMB NO.	COOR. SYSTEM	ORIGIN	AXES
1	1	0,0,0	X,Y,Z
2	1	0,0,0	X,Y,Z
3	1	0,0,0	X,Y,Z
4	1	0,0,0	X,Y,Z
5	1	0,0,0	X,Y,Z
6	1	0,0,0	X,Y,Z
7	1	0,0,0	X,Y,Z
8	1	0,0,0	X,Y,Z
9	1	0,0,0	X,Y,Z
10	1	0,0,0	X,Y,Z

7	2.98	3.48	-3.48
8	-2.98	3.48	-3.48
9	25.77	25.37	25.37
10	-1.37	-7.35	7.35
11	-2.03	-3.33	3.33
12	8.94	8.92	8.92
FIN DE C/BO			

# MARCO EJE ⑤

SISMO

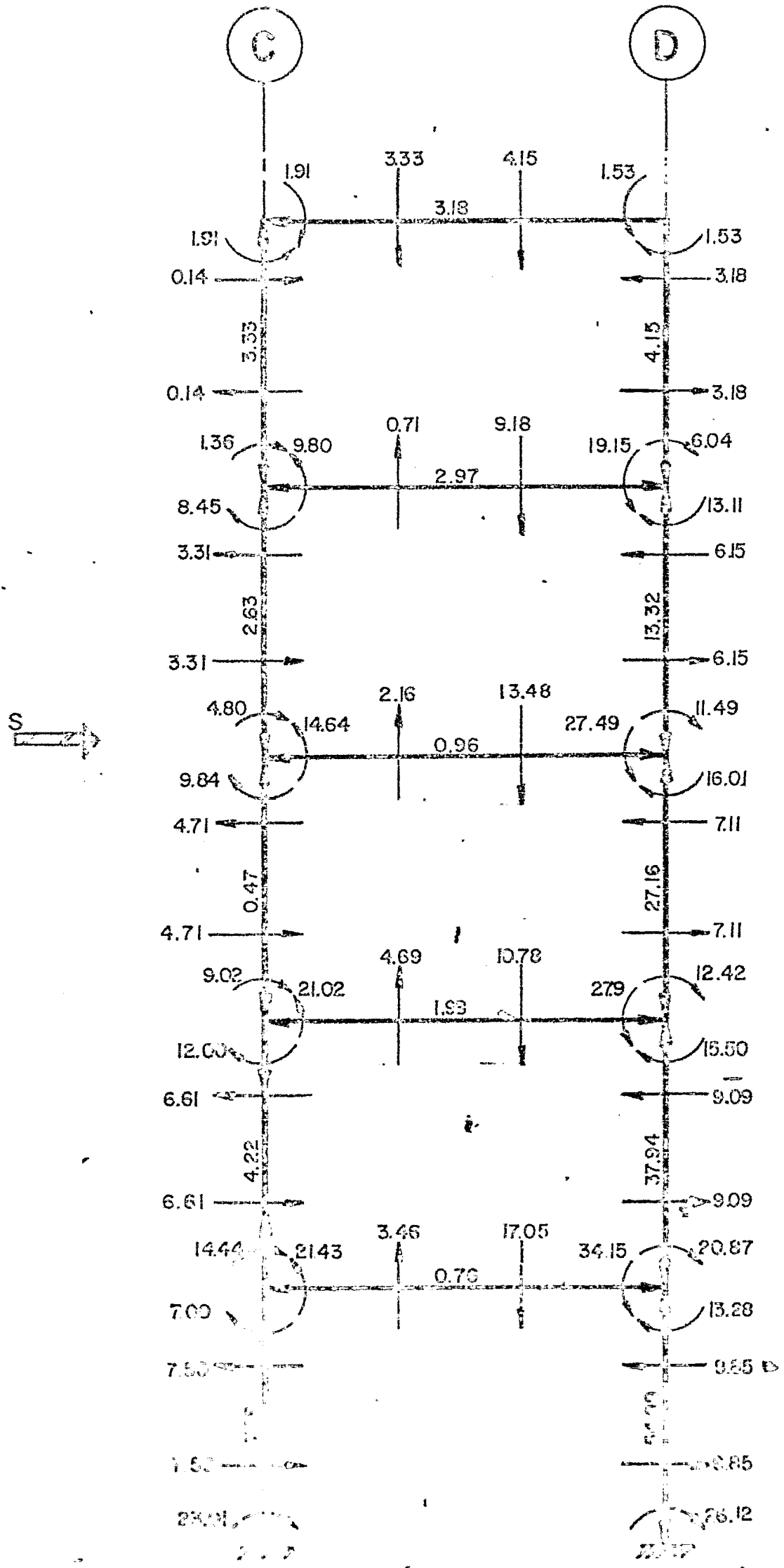




# MARCO EJE I

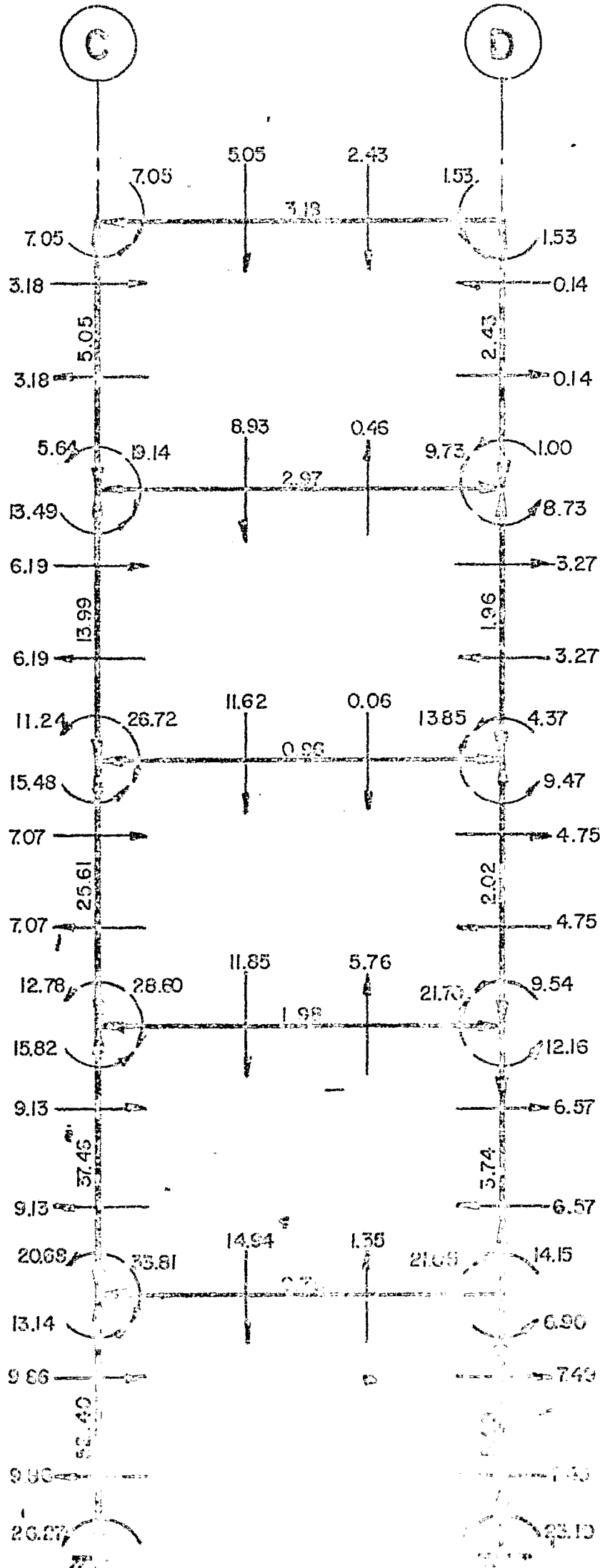
CM. + E.O. + S

C.M. = CARGA MUERTA  
 E.O. = EQUIPO EN OPERACION  
 S = SISMO



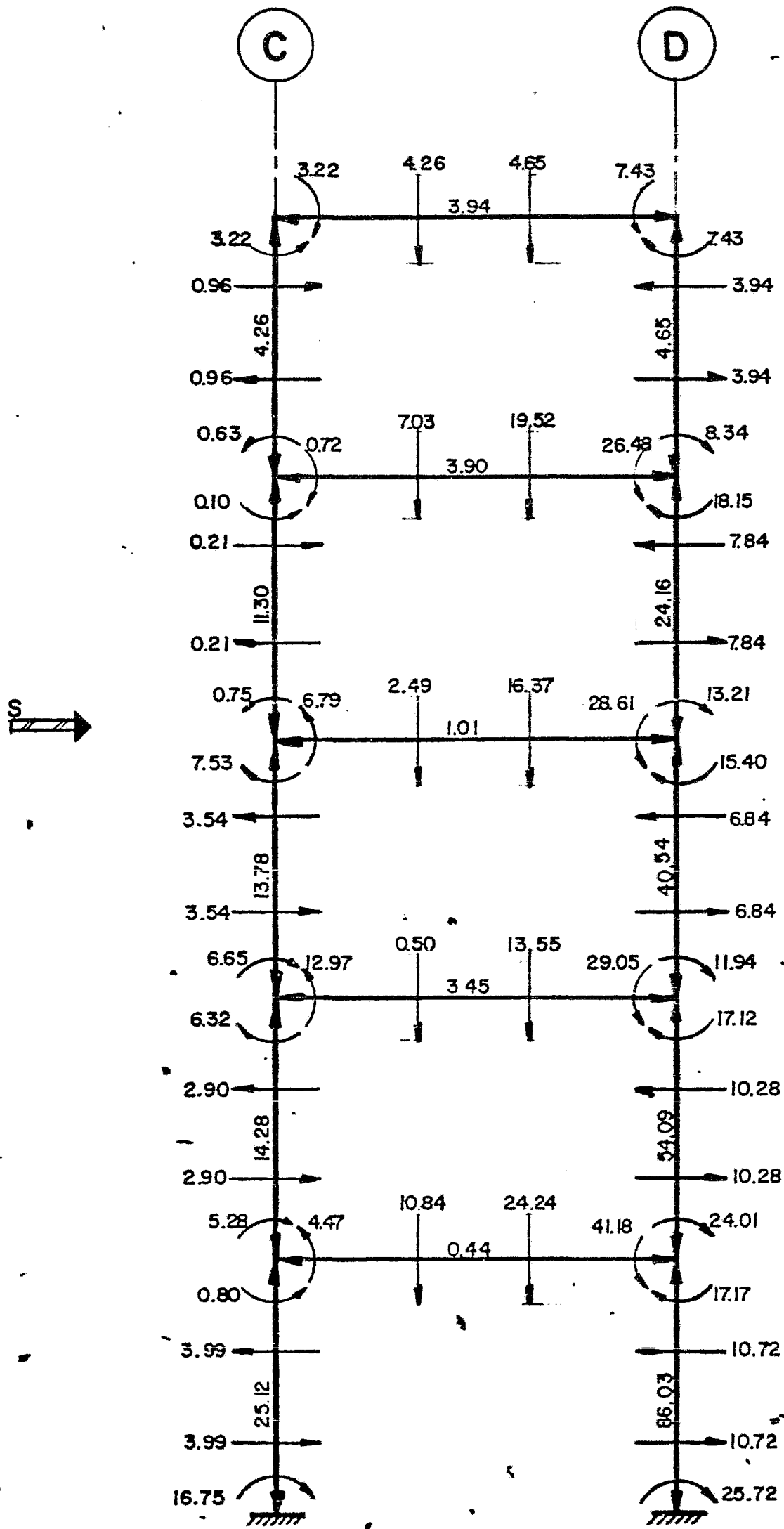
# MARCO EJE I

C.M. + E.O. + S



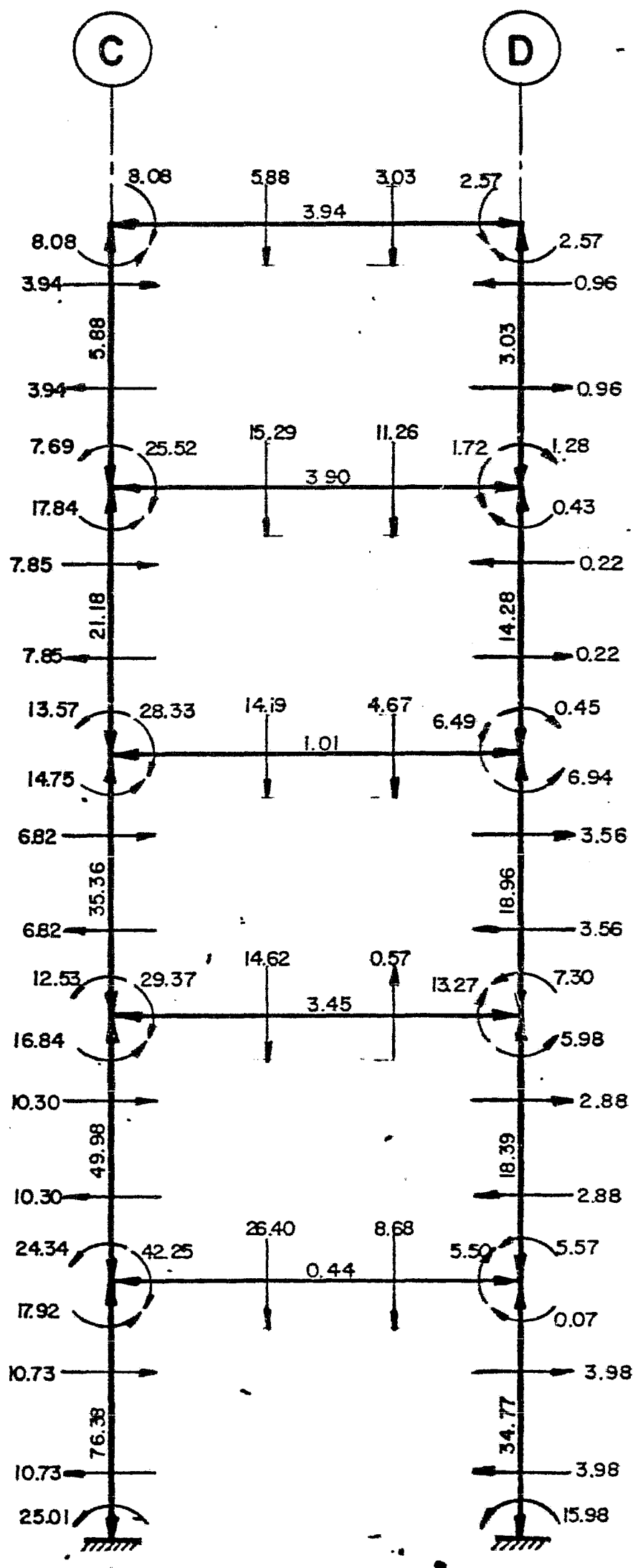
# MARCO EJE ②

C. M. + E. O. + S



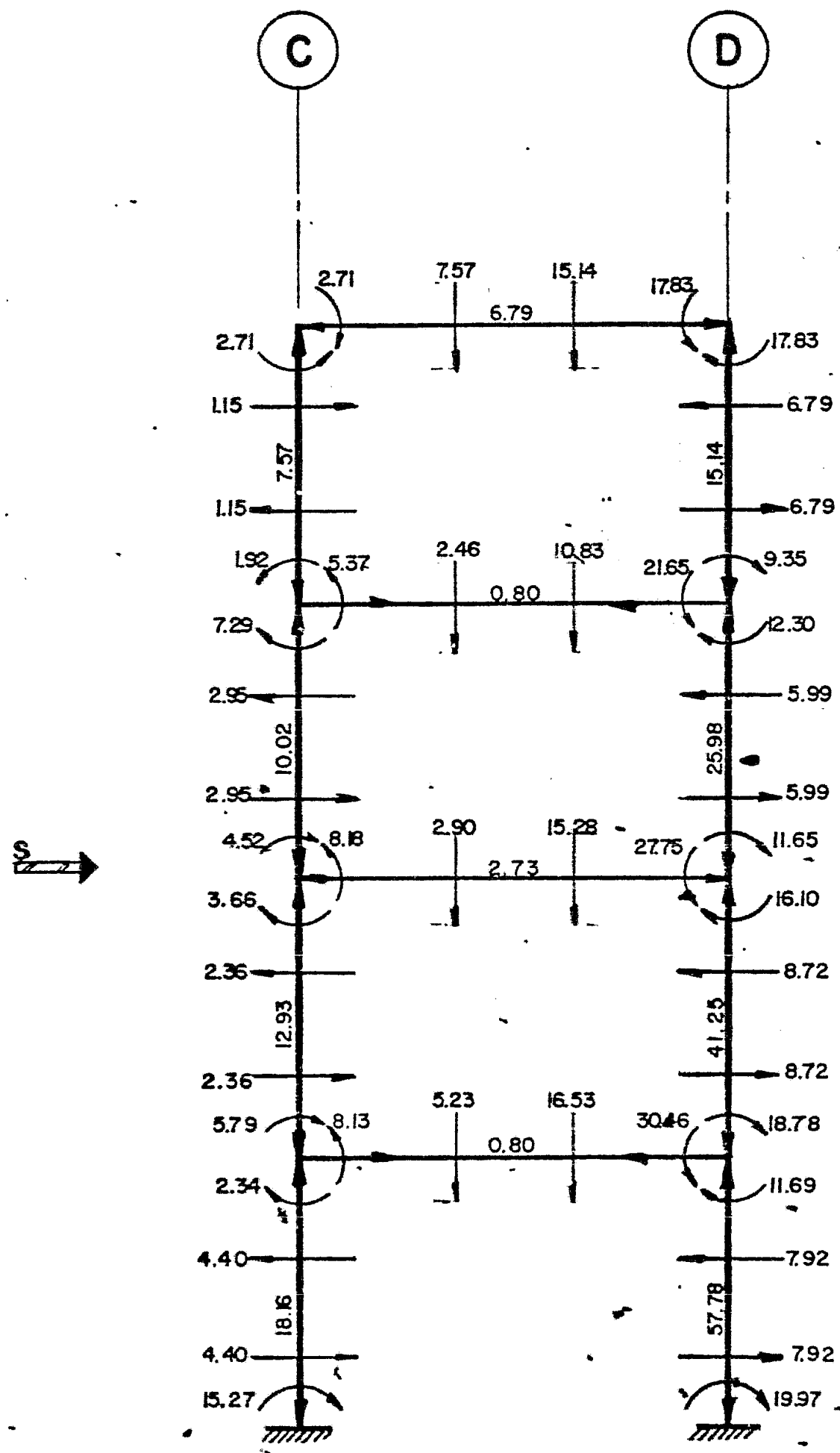
# MARCO EJE ②

C. M. + E. O. + S



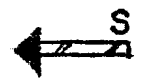
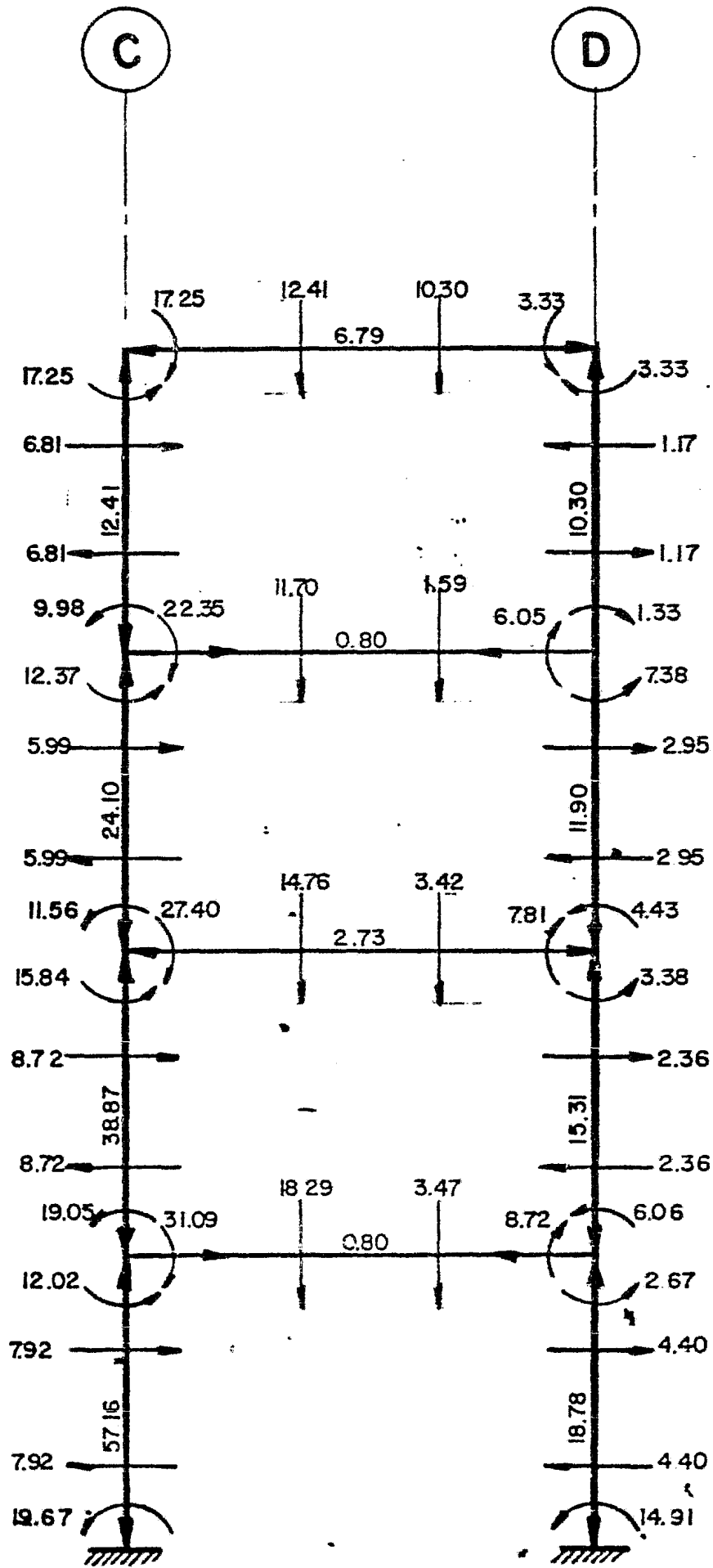
# MARCO EJE ③

C.M. + E.O. + S



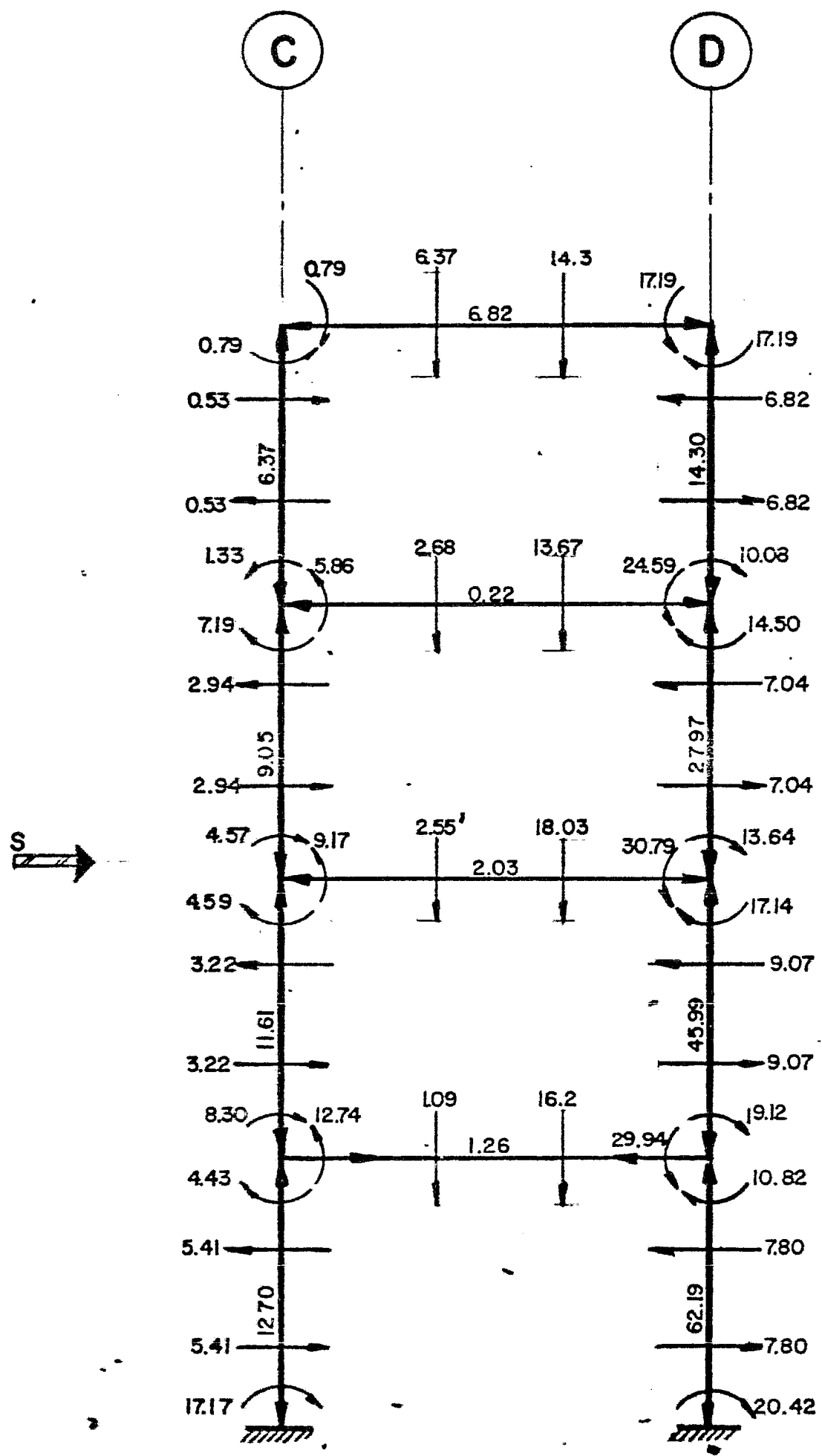
# MARCO EJE ③

C.M. + E.O. + S



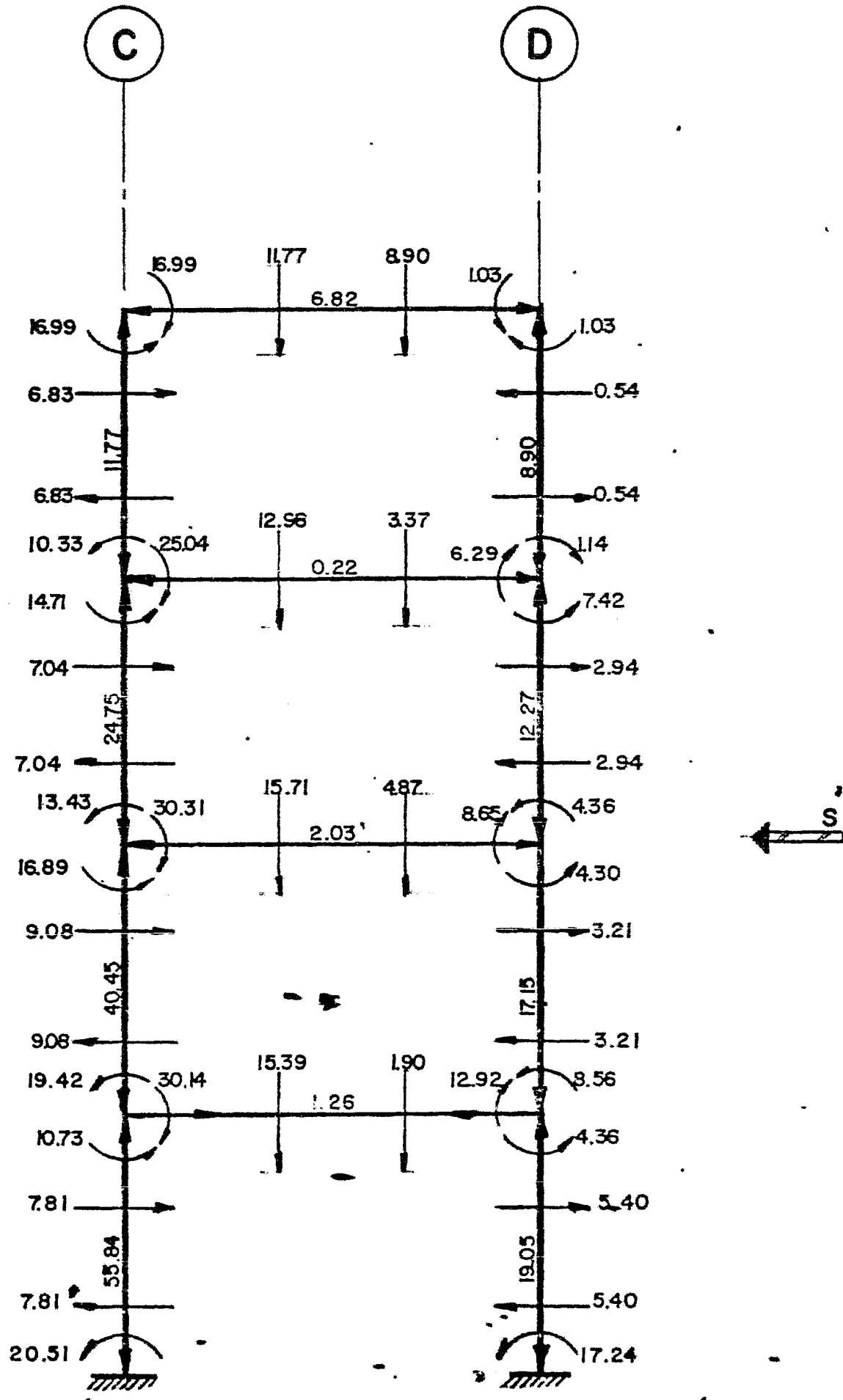
# MARCO EJE ④

C.M. + E.O. + S



# MARCO EJE ④

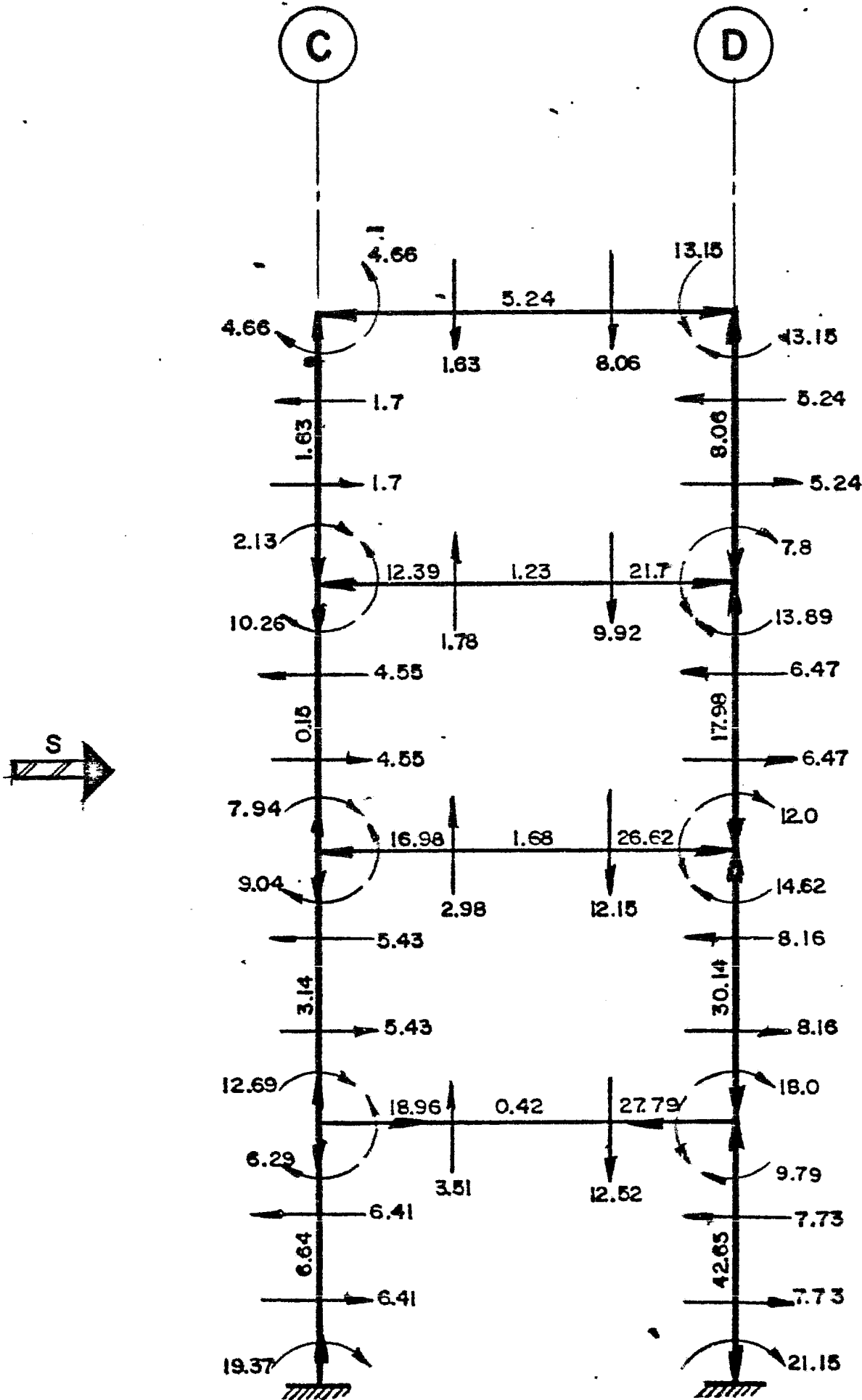
C.M. + E.O. + S





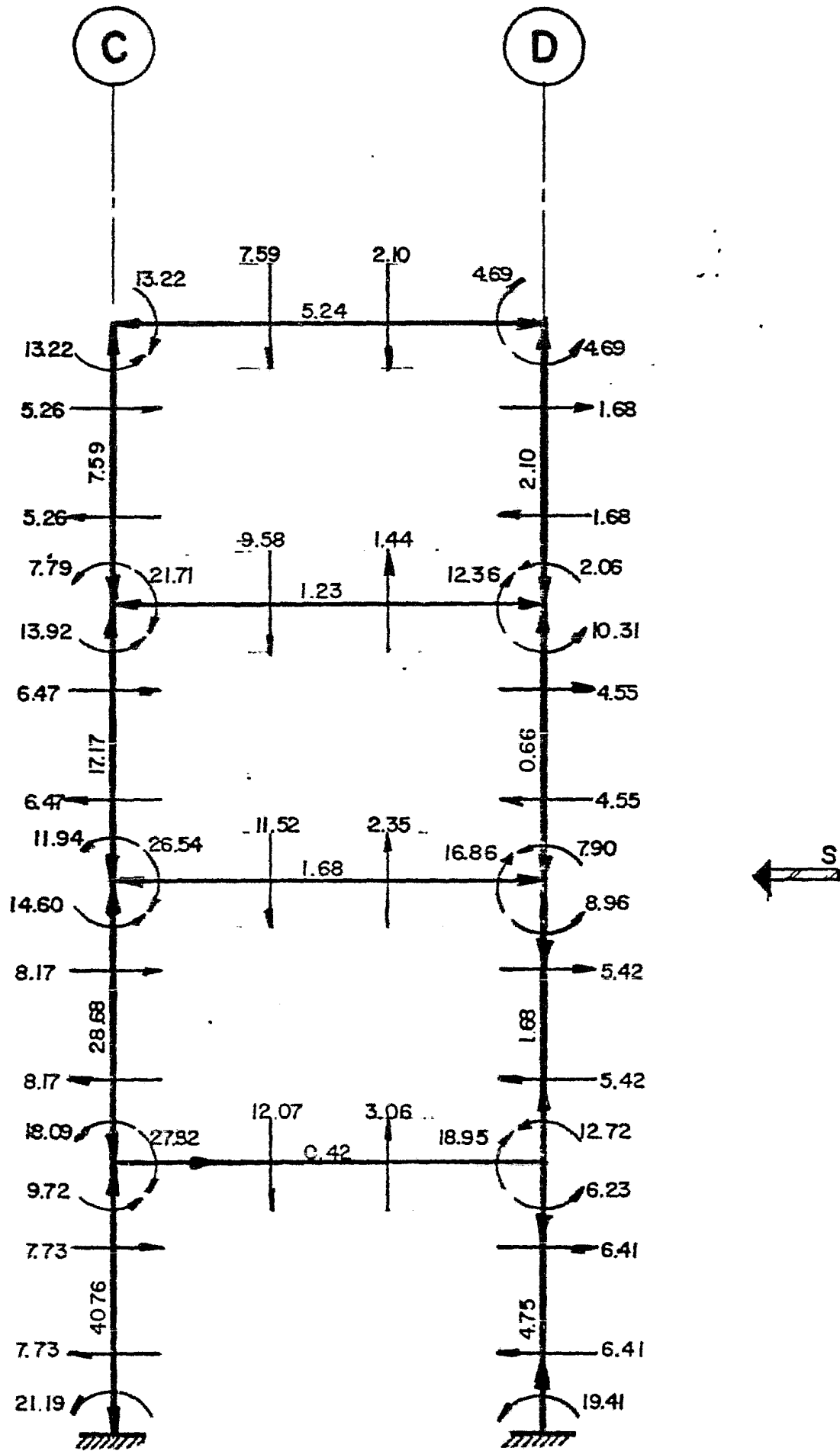
# MARCO EJE ⑤

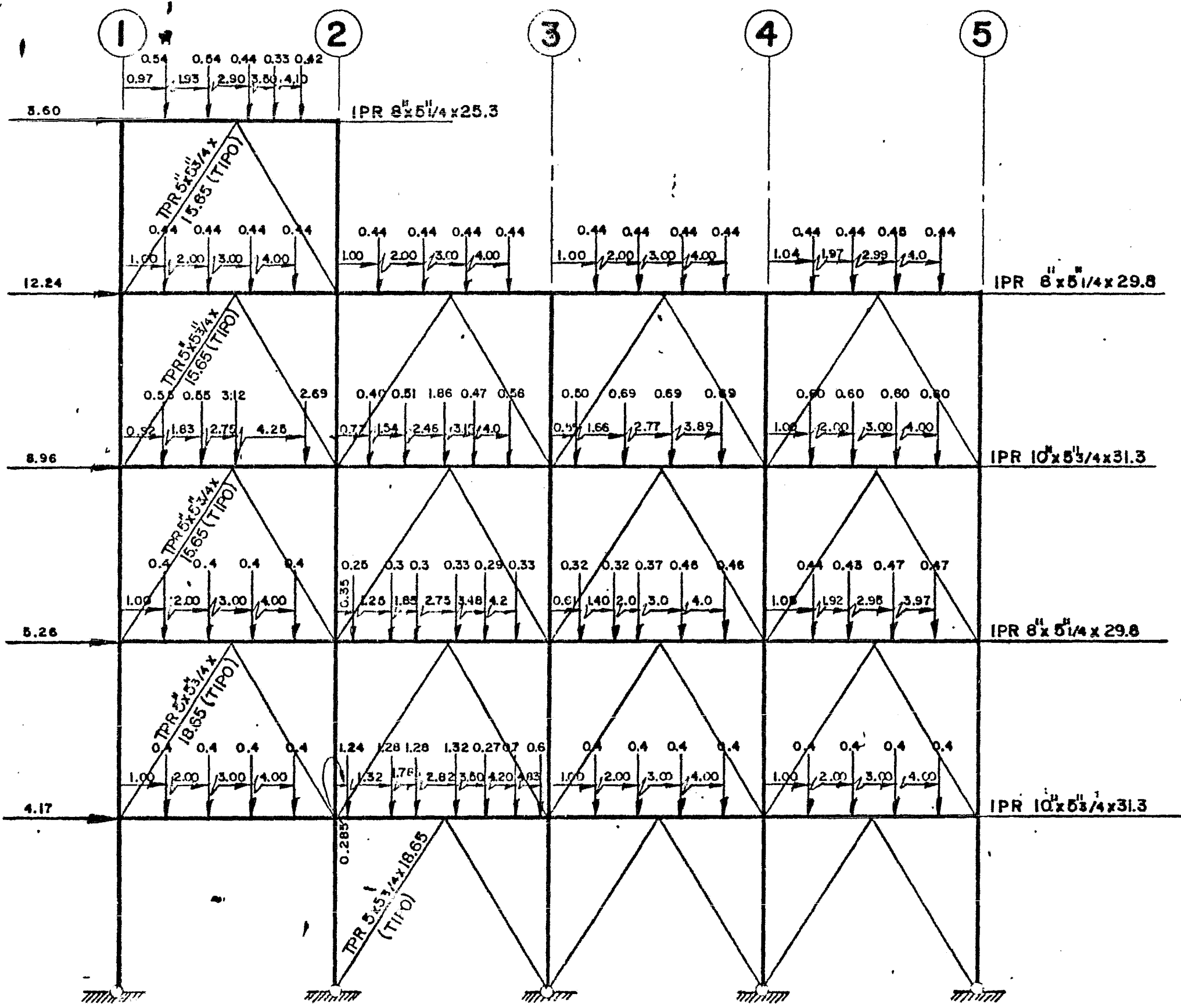
C.M + E.O. + S



# MARCO EJE 5

C. M. + E. O. + S





1

2

3

4

5

3.60

12.24

8.96

5.26

4.17

IPR 8<sup>11</sup> x 5<sup>11</sup> / 4 x 25.3

IPR 8<sup>11</sup> x 5<sup>11</sup> / 4 x 29.8

IPR 10<sup>11</sup> x 5<sup>11</sup> / 4 x 31.3

IPR 8<sup>11</sup> x 5<sup>11</sup> / 4 x 29.8

IPR 10<sup>11</sup> x 5<sup>11</sup> / 4 x 31.3

0.54 0.64 0.44 0.33 0.42  
0.97 1.93 2.90 3.88 4.10

0.44 0.44 0.44 0.44  
1.00 2.00 3.00 4.00

0.44 0.44 0.44 0.44  
1.00 2.00 3.00 4.00

0.44 0.44 0.44 0.44  
1.00 2.00 3.00 4.00

0.44 0.44 0.45 0.44  
1.04 1.97 2.99 4.00

0.55 0.55 3.12 2.69  
0.92 1.83 2.75 4.25

0.40 0.51 1.86 0.47 0.56  
0.77 1.54 2.46 3.15 4.00

0.50 0.69 0.69 0.69  
0.97 1.66 2.77 3.89

0.50 0.60 0.60 0.50  
1.05 2.00 3.00 4.00

0.4 0.4 0.4 0.4  
1.00 2.00 3.00 4.00

0.25 0.3 0.3 0.33 0.29 0.33  
0.35 1.25 1.85 2.73 3.48 4.2

0.32 0.32 0.37 0.46 0.46  
0.61 1.40 2.0 3.0 4.0

0.44 0.43 0.47 0.47  
1.05 1.92 2.95 3.97

0.4 0.4 0.4  
1.00 2.00 3.00 4.00

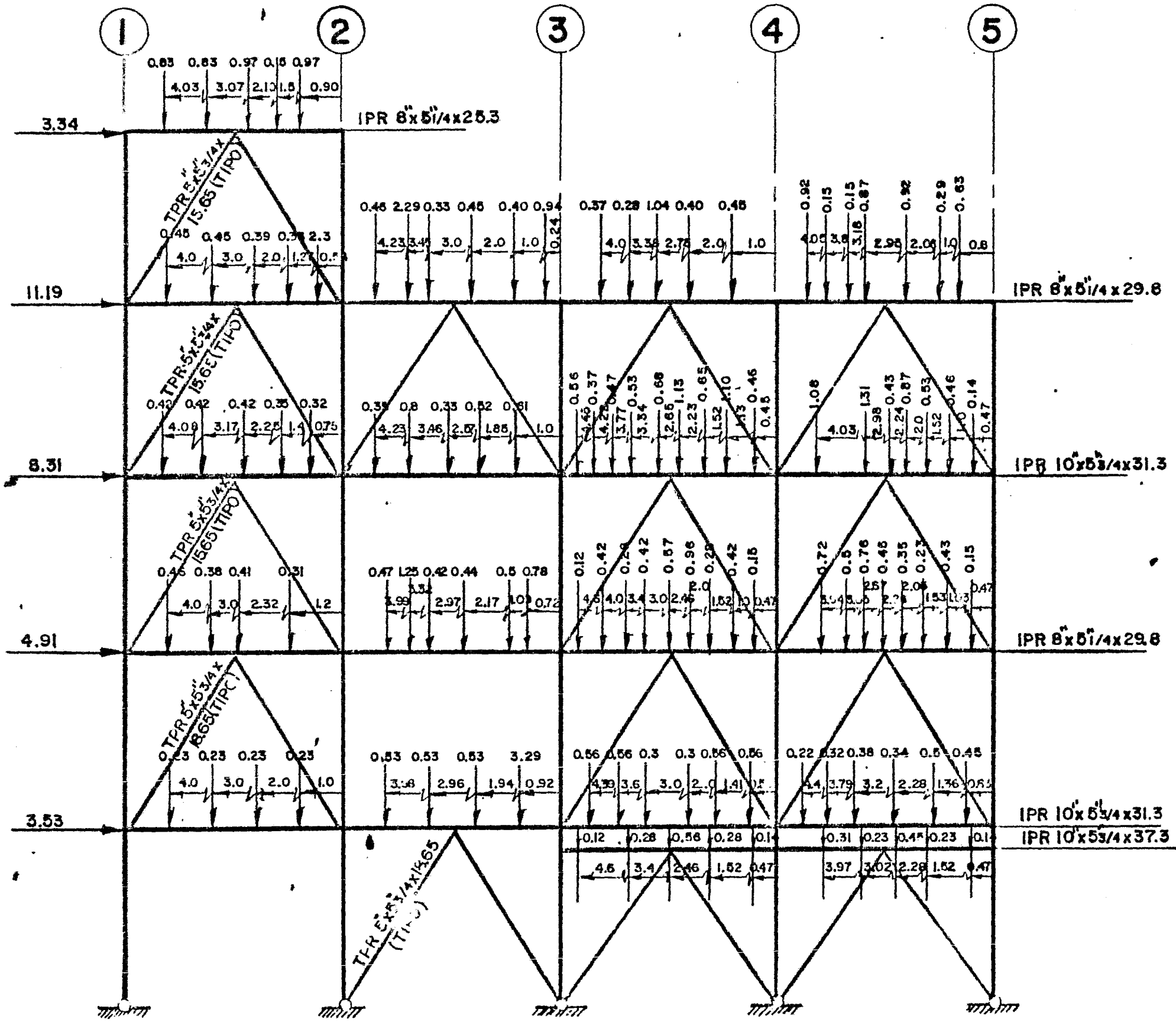
1.24 1.28 1.28 1.32 0.27 0.7 0.6  
1.32 1.75 2.82 3.60 4.20 4.93

0.4 0.4 0.4 0.4  
1.00 2.00 3.00 4.00

0.4 0.4 0.4 0.4  
1.00 2.00 3.00 4.00

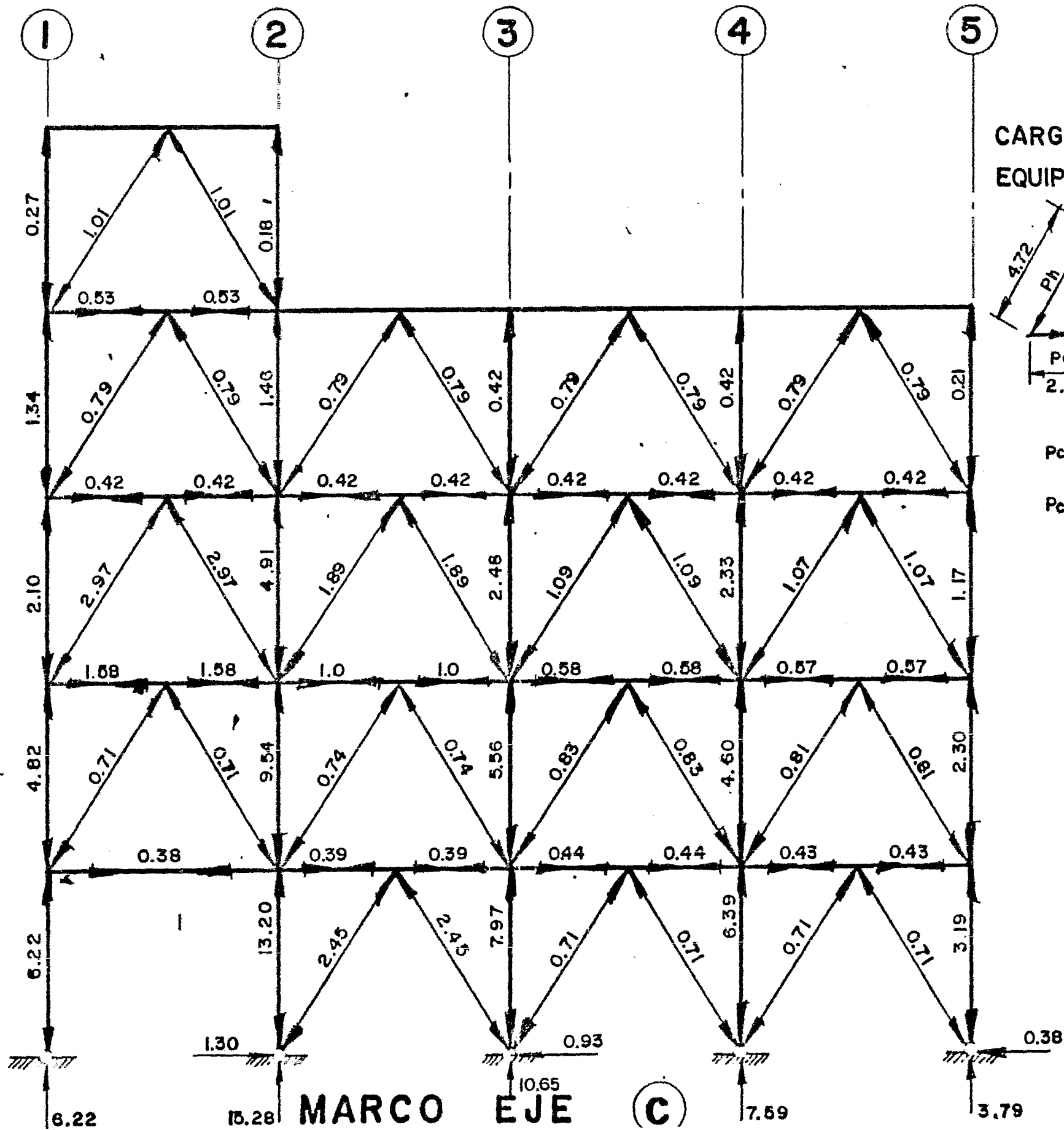
0.285  
IPR 5<sup>11</sup> x 5<sup>11</sup> / 4 x 18.65 (TIPO)

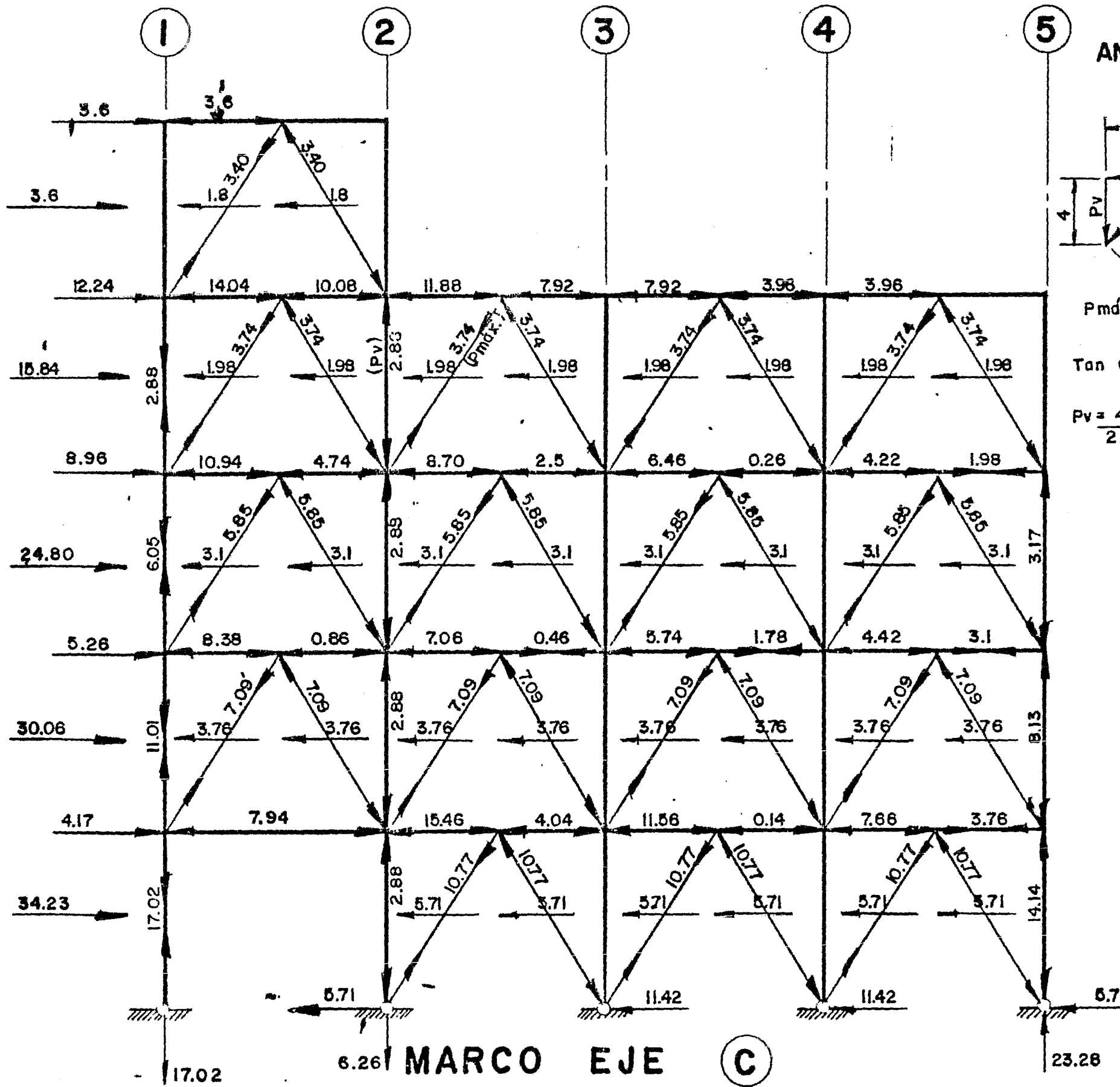
MARCO EJE (C)



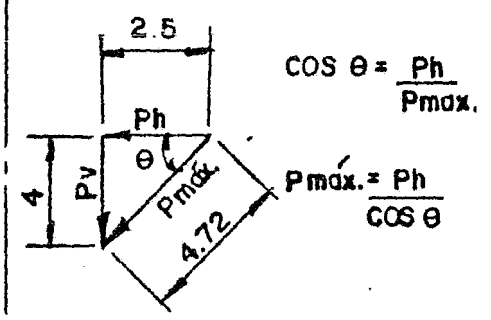
MARCO EJE

D





### ANALISIS SISMICO DEFINITIVO



$$\cos \theta = \frac{Ph}{P_{max}}$$

$$P_{max} = \frac{Ph}{\cos \theta}$$

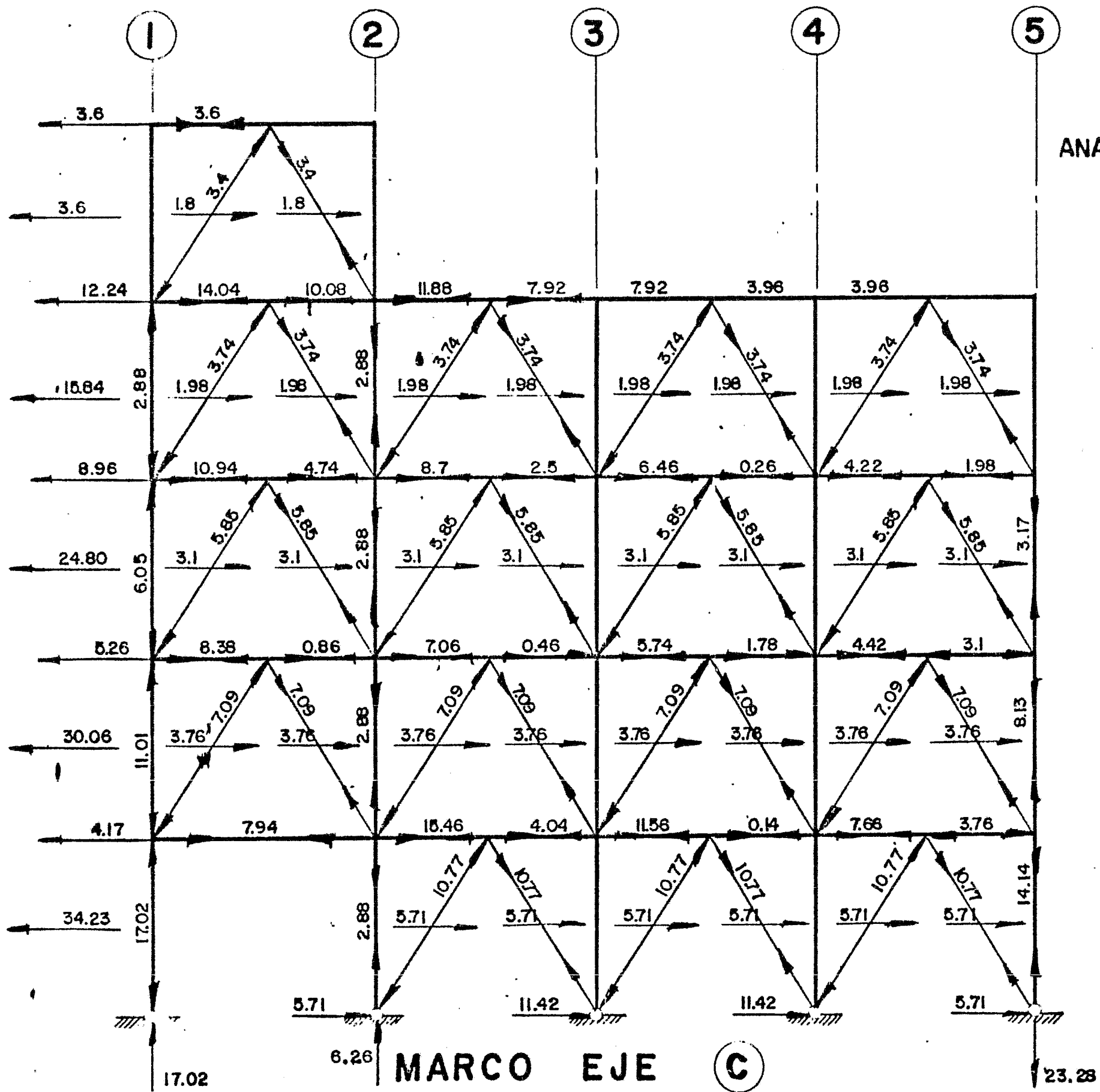
$$P_{max} = \frac{4.72 Ph}{2.5} = 1.89 Ph$$

$$\tan \theta = \frac{Pv}{Ph}; Pv = Ph \tan \theta$$

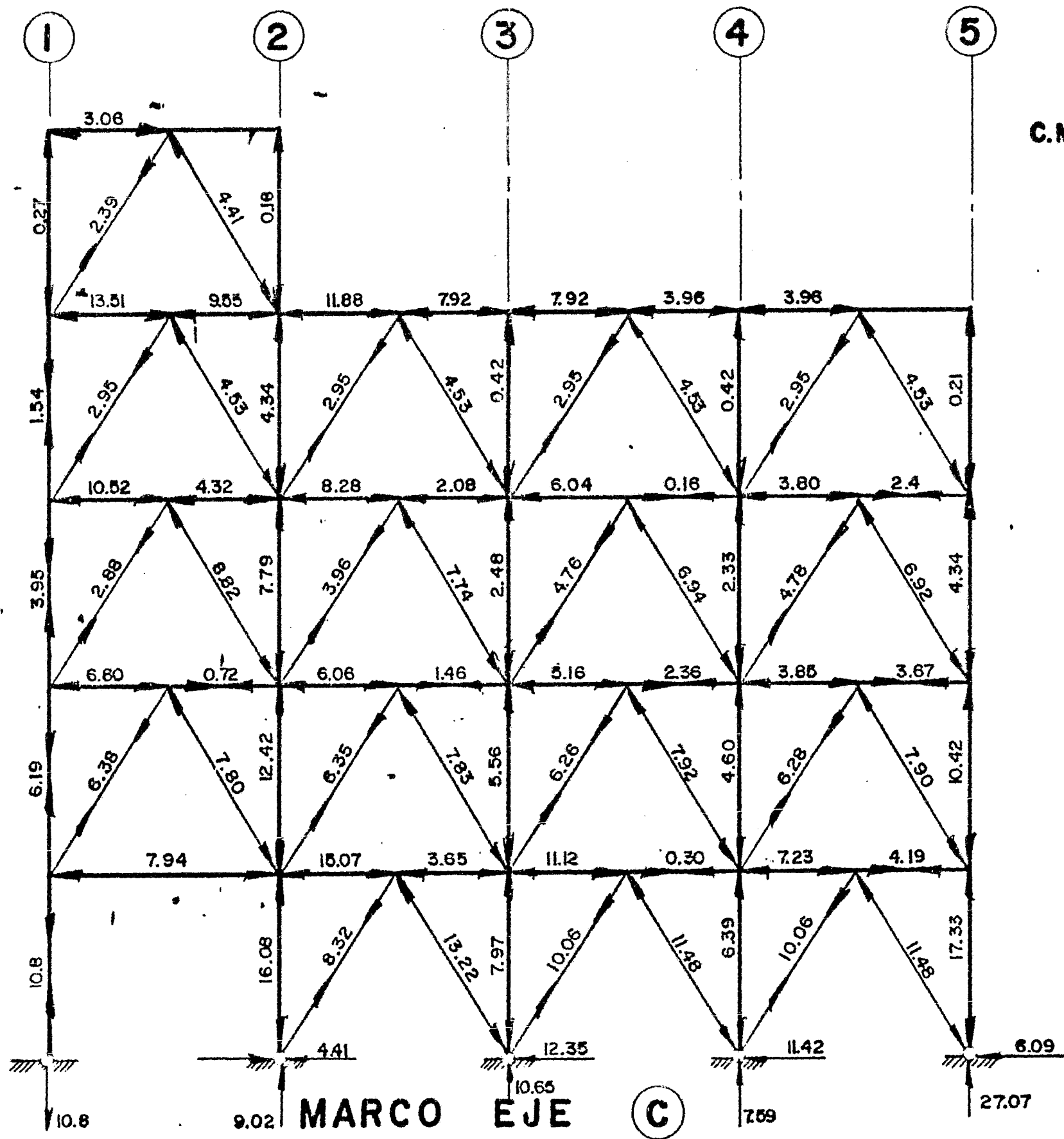
$$Pv = \frac{4}{2.5} Ph \quad Pv = 1.6 Ph$$

MARCO EJE C

ANALISIS SISMICO  
DEFINITIVO



MARCO EJE C

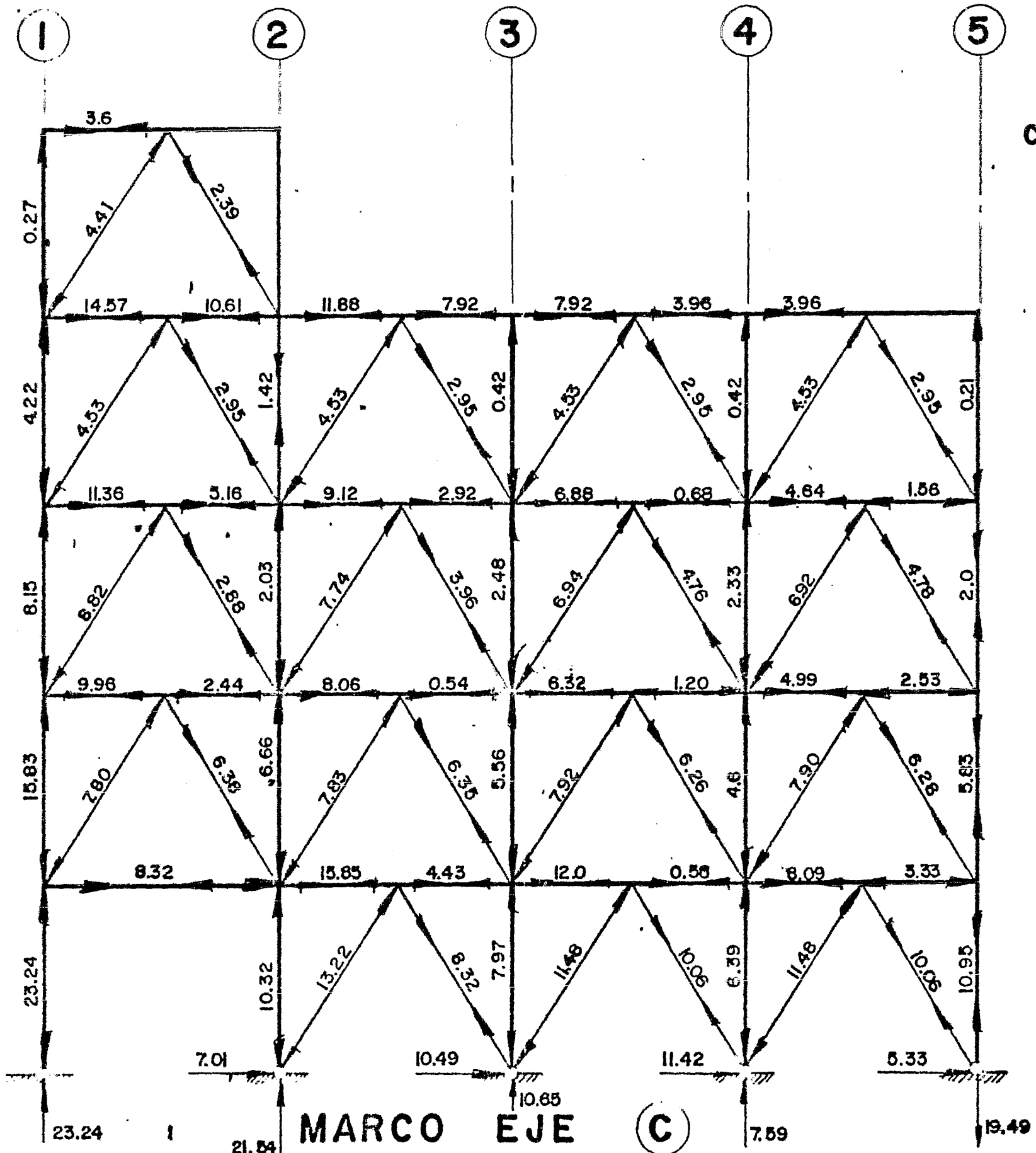


C.M+E.O+S



MARCO EJE C





CM+E.O-S



MARCO EJE

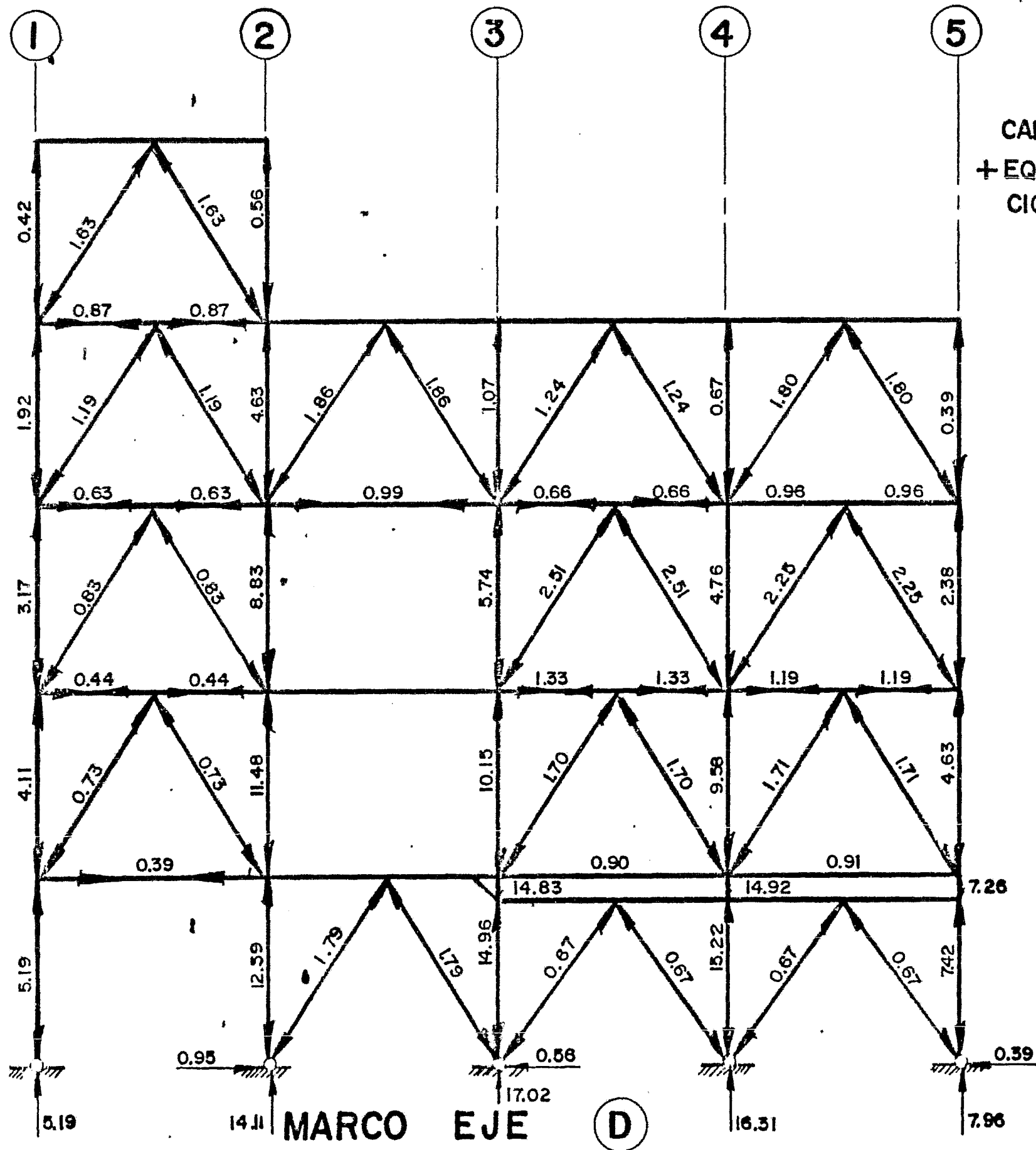
(C)

23.24

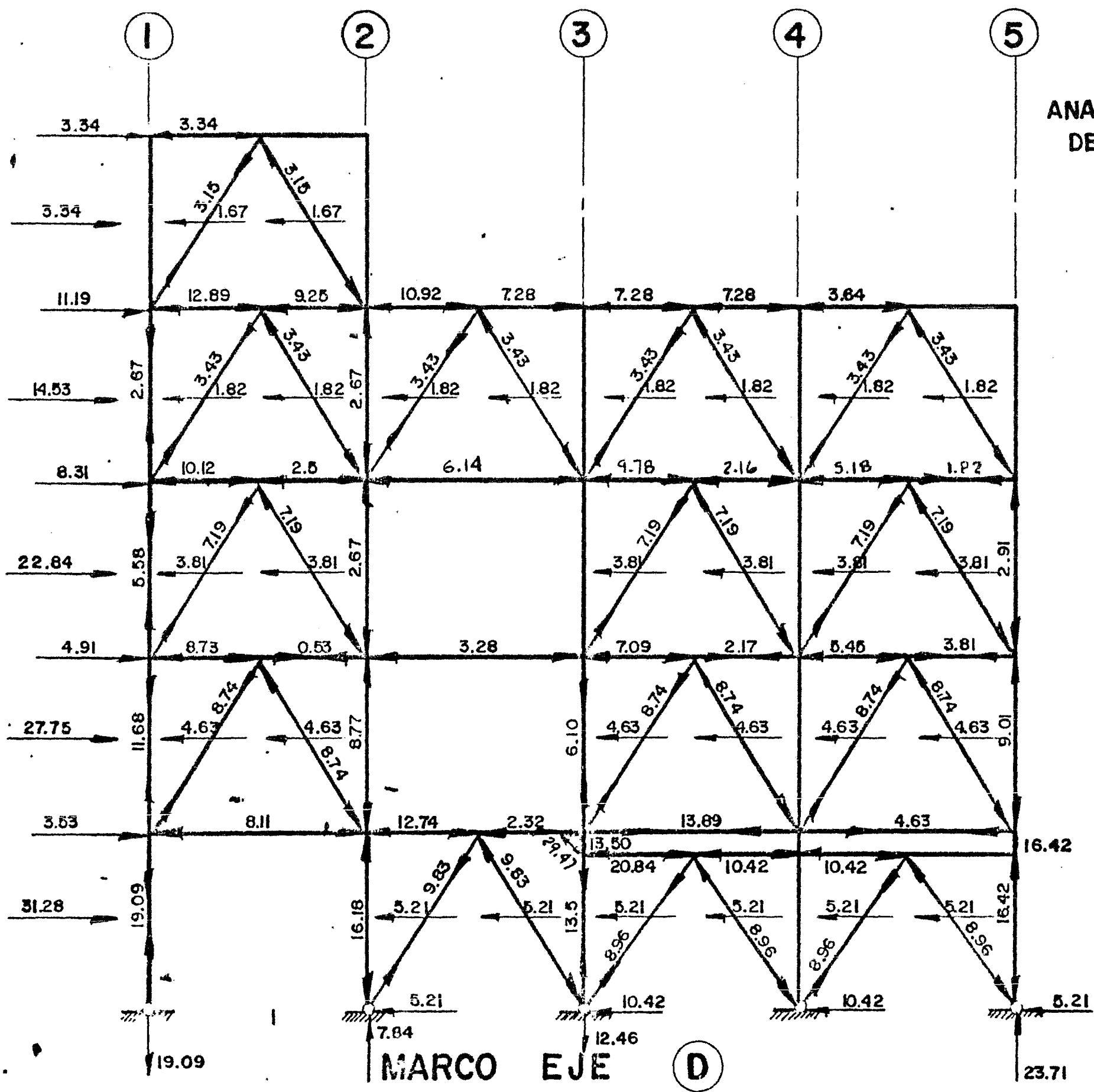
21.54

7.59

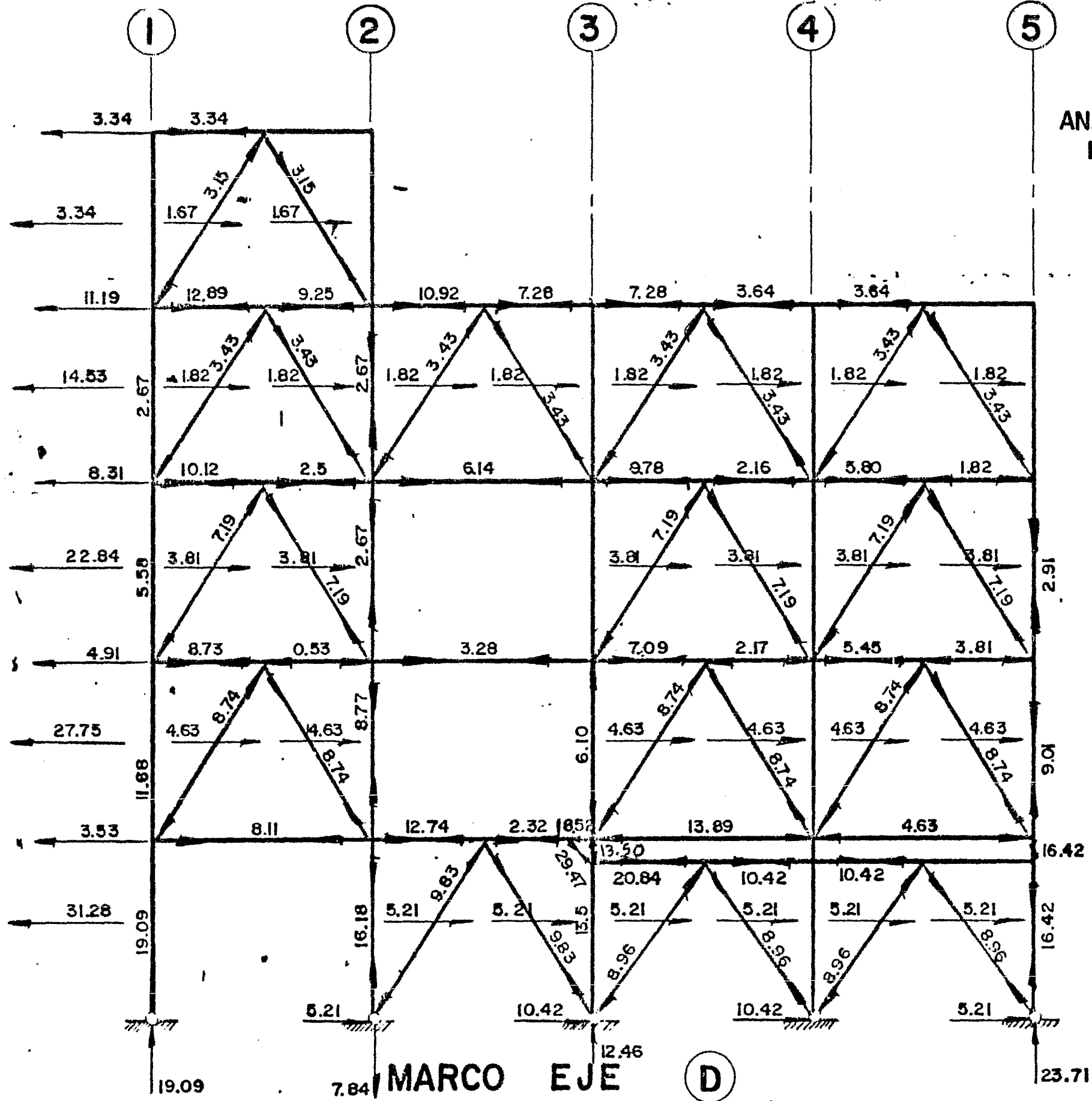
19.49

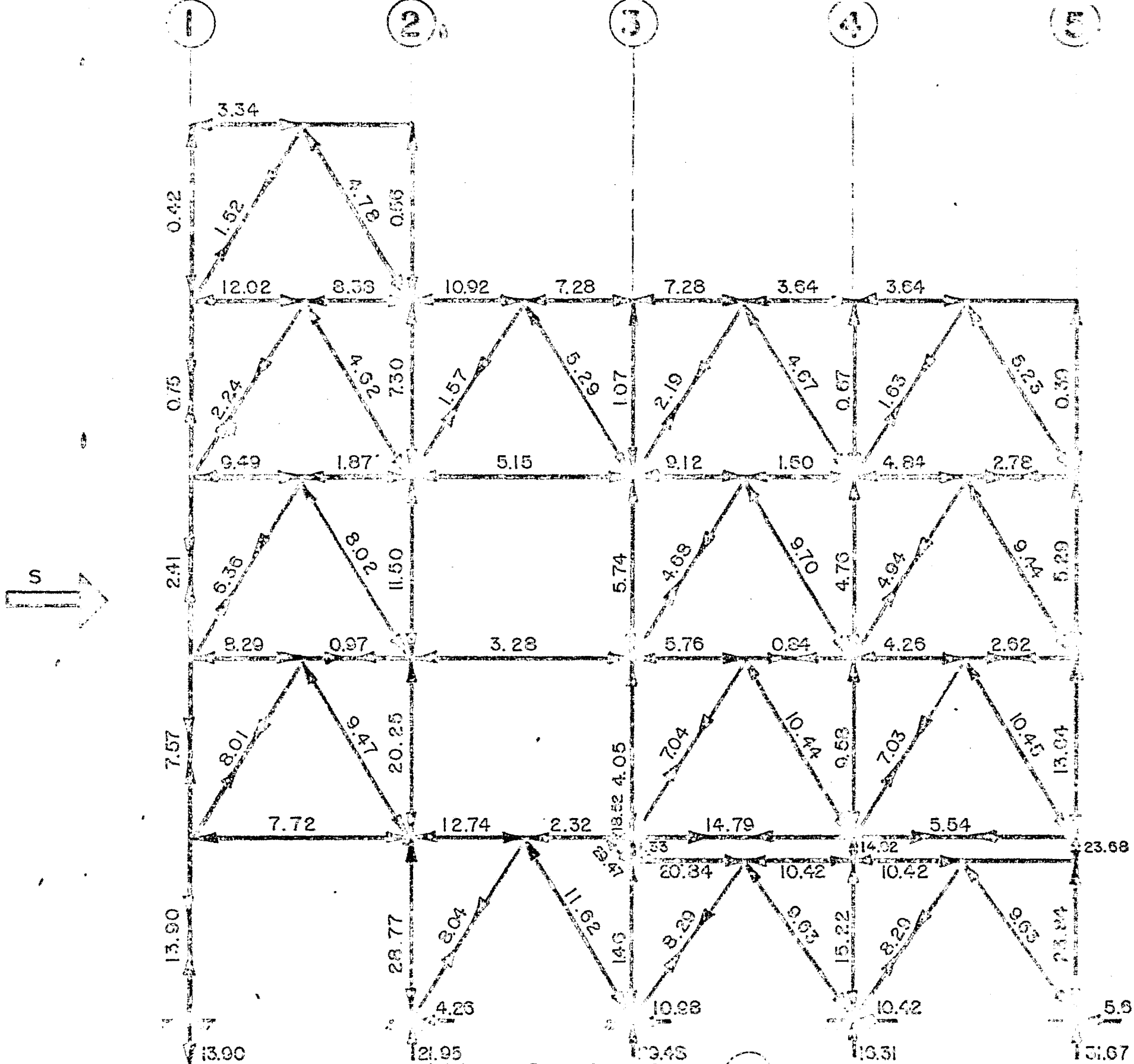


ANALISIS SISMICO  
DEFINITIVO



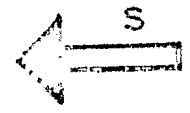
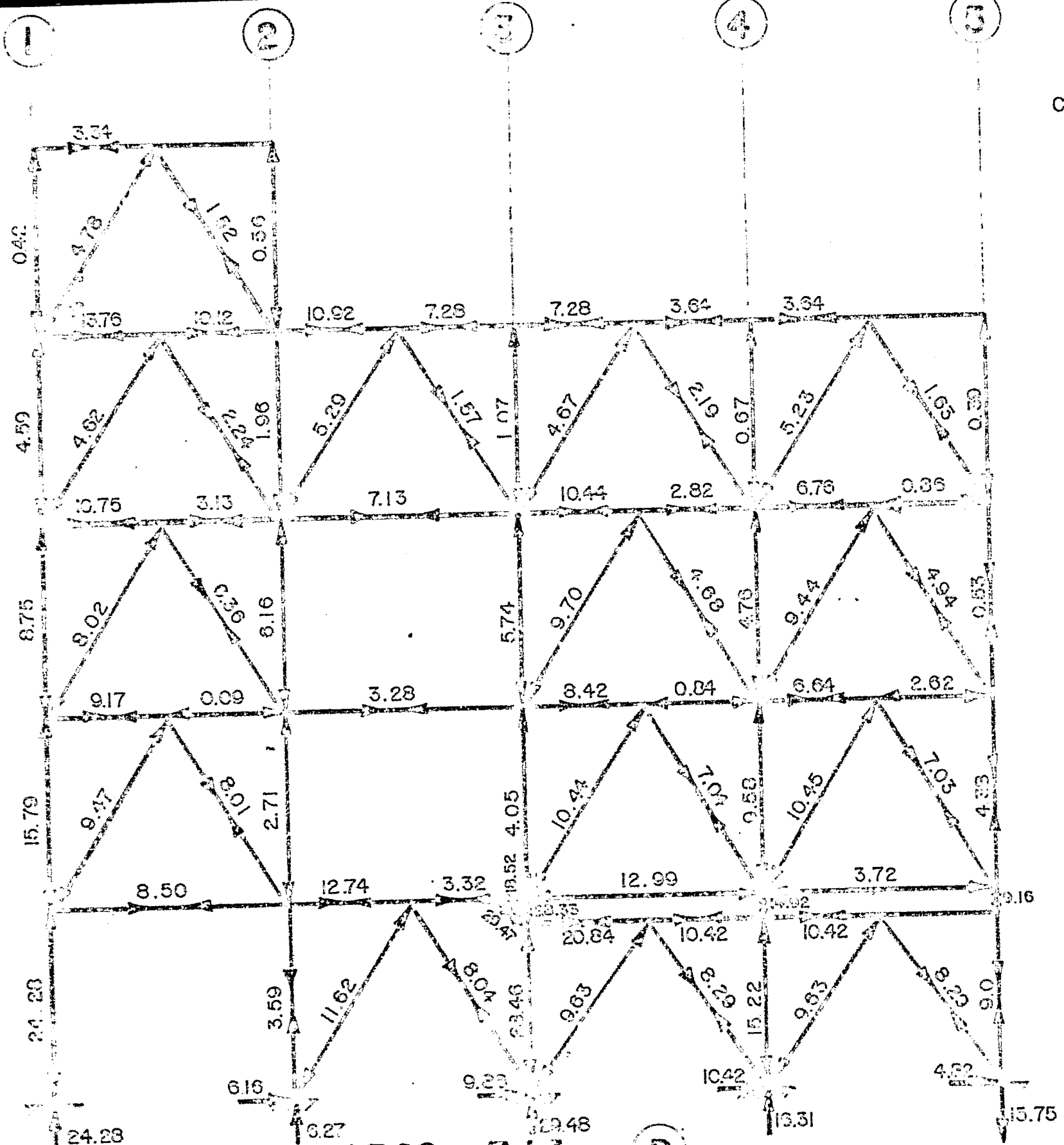
ANALISIS SISMICO  
DEFINITIVO





MARCO E. P. **D**

CM + E.O - S



MARCO EJE (D)

## CAPITULO 4 Y 5

### DISEÑO ESTRUCTURAL Y REVISION

En este capítulo se diseñaron y revisaron las secciones propuestas en el análisis y diseño preliminar.

Una vez conocidos los elementos mecánicos tanto por cargas gravitacionales como por cargas accidentales, se procedió al diseño definitivo de los diferentes miembros de la estructura. Siguiendo los criterios de diseño que marca el A.I.S.C. y los cuales se aplicaron para las condiciones más severas de trabajo, es decir, cada elemento se diseñó para la envolvente máxima.

Este diseño nos sirvió para elaborar el capítulo 5 ya que se verificó si los perfiles propuestos eran los adecuados para las cargas impuestas.

# DISEÑO DEFINITIVO

## DISEÑO TRABES SECUNDARIAS

(NO SE DISEÑAN TODAS LAS TRABES SE ELIGE UNA SOLA TRABE PARA MOSTRAR EL PROCEDIMIENTO DE DISEÑO).

T-56

ACERO A-36

$$L = 500 \text{ cm}$$

$$V = 3.79 \text{ TON}$$

$$M_{\text{MAX}} = 8.2 \text{ TON}\cdot\text{M}$$

$$F_y = 2530 \text{ kg/cm}^2$$

SI SUPONEMOS  $F_{bx} = 1520 \text{ kg/cm}^2$

$$S_x = \frac{M}{F_{bx}} = \frac{8.2 \times 10^5}{1520} = 539 \text{ cm}^3$$

DEL MANUAL AHMSA.

SI PROBAMOS CON UNA I.P.R.  $12 \times 6\frac{1}{2} \times 40.3$  ( $S_x = 558 \text{ cm}^3$ )

$$A = 51.42 \text{ cm}^2$$

$$d = 30.4 \text{ cm}$$

$$b_f = 16.5 \text{ cm}$$

$$t_f = 1.02 \text{ cm}$$

$$t_w = 0.61 \text{ cm}$$

$$a = 1.81 \text{ cm}^{-1}$$

$$A_f$$

$$I_x = 8495 \text{ cm}^4$$

$$S_x = 558 \text{ cm}^3$$

$$r = 2.1 \text{ cm}$$

$$L_b = 100 \text{ cm}$$

REVISION SI LA SECCION ES "COMPACTA"

$$L_b \leq \frac{20.1416 b_f}{\sqrt{F_y}} = \frac{20.1416 \times 16.5}{\sqrt{2.53}} = 209 \quad F_y [\text{EN TON/cm}^2]$$

$$100 < 209 \quad \text{SI CUMPLE}$$

$$L_b \leq \frac{1405.56}{\left(\frac{d}{A_f}\right) F_y} = \frac{1405.56}{(1.81) 2.53} = 307$$

$$100 < 307 \quad \text{SI CUMPLE}$$

$$\frac{b_f}{2t_f} \leq \frac{25.185}{\sqrt{F_y}}$$

$$\frac{16.5}{2 \times 1.02} = 8.09 \leq \frac{25.185}{\sqrt{2.53}} = 15.83 \quad \text{SI CUMPLE}$$



$$\frac{bf}{2tf} \leq \frac{17.2315}{\sqrt{F_y}}$$

$$\frac{17.2315}{\sqrt{2.53}} = 10.83$$

$$8.09 < 10.83 \quad \text{SI CUMPLE}$$

$$\frac{d}{t_w} \leq \begin{cases} \frac{169.6637}{\sqrt{F_y}} \left(1 - 3.74 \frac{f_a}{F_y}\right) & \text{CUANDO } \frac{f_a}{F_y} \leq 0.16 \\ \frac{68.1306}{\sqrt{F_y}} & \text{CUANDO } \frac{f_a}{F_y} > 0.16 \end{cases}$$

$$f_a = 0$$

$$\frac{f_a}{F_y} = 0$$

$$\frac{d}{t_w} = \frac{30.4 - 2 \times 1.02}{0.61} = 46.5 < \frac{169.6637}{\sqrt{2.53}} = 106.7$$

∴ LA SECCION ES COMPACTA.

$$F_{bx} = 0.66 \times 2530 = 1670 \text{ kg/cm}^2$$

$$M_R = 1670 \times 558 = 9.32 \times 10^5 \text{ kg-cm} > 8.2 \times 10^5 \text{ kg-cm} \quad \eta = 88\%$$

REVISION DEL CORTANTE.

$$F_v = 0.4 F_y = 1012 \text{ kg/cm}^2 \quad (\text{PERMISIBLE})$$

$$f_{v, \text{ACT.}} = \frac{3790}{0.61 \times 2836} = 219 \text{ kg/cm}^2 < 1012 \text{ kg/cm}^2 \quad \text{ES CORRECTO.}$$

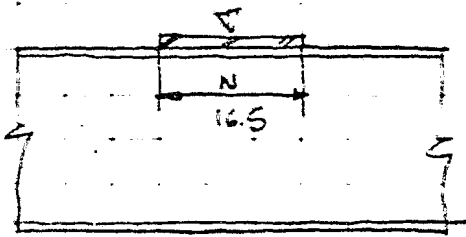
DEFLEXION

$$\delta_{\text{MAX}} = 1.0 \text{ cm}$$

$$\delta_{\text{PERM}} = \frac{l}{360} = \frac{500}{360} = 1.39 > 1.06 \quad \text{ES CORRECTO}$$

REVISION SI SE REQUIEREN ATIESADORES DE CARGA

$$R = 5200 \text{ Kg}$$



$$\frac{R}{t_w(N + 2k)} \leq 0.75 F_y \quad (\text{INTERIORES})$$

$$R = 5200 \text{ Kg} \quad k = 2.1 \text{ cm}$$

$$N = 16.5 \text{ cm} \quad t_w = 0.61 \text{ cm}$$

$$\frac{5200}{0.61(16.5 + 2 \times 2.1)} = 412 \text{ Kg/cm}^2 < 0.75 \times 2530 = 1890 \text{ Kg/cm}^2$$

$$R = 630 \text{ Kg}$$

$$N = 0$$

$$\frac{630}{0.61(0 + 2 \times 2.1)} = 246 \text{ Kg/cm}^2 < 1890 \text{ Kg/cm}^2$$

∴ NO SE REQUIEREN ATIESADORES DE CARGA.

REVISION SI SE REQUIEREN ATIESADORES DE RIGIDEZ

$$a = 500 \text{ cm}$$

$$h = 30.5 - 2 \times 1.02 = 28.46 \text{ cm}$$

$$t = 0.61 \text{ cm}$$

$$\frac{h}{t} = 47 < 260$$

$$\frac{a}{h} = 18.73$$

$$\frac{a}{h} > 1 \Rightarrow k = 5.34 + \frac{c}{(1.2)^2} = 5.34$$

$$C_v = \frac{1990}{47} \sqrt{\frac{5.34}{2530}} = 1.55$$

$$F_v = \frac{2530}{2.59} \times 1.55 = 1357 \text{ Kg/cm}^2$$

$$\therefore F_v = 0.4 F_y = 1012 \text{ Kg/cm}^2 > f_{cr} = 219$$

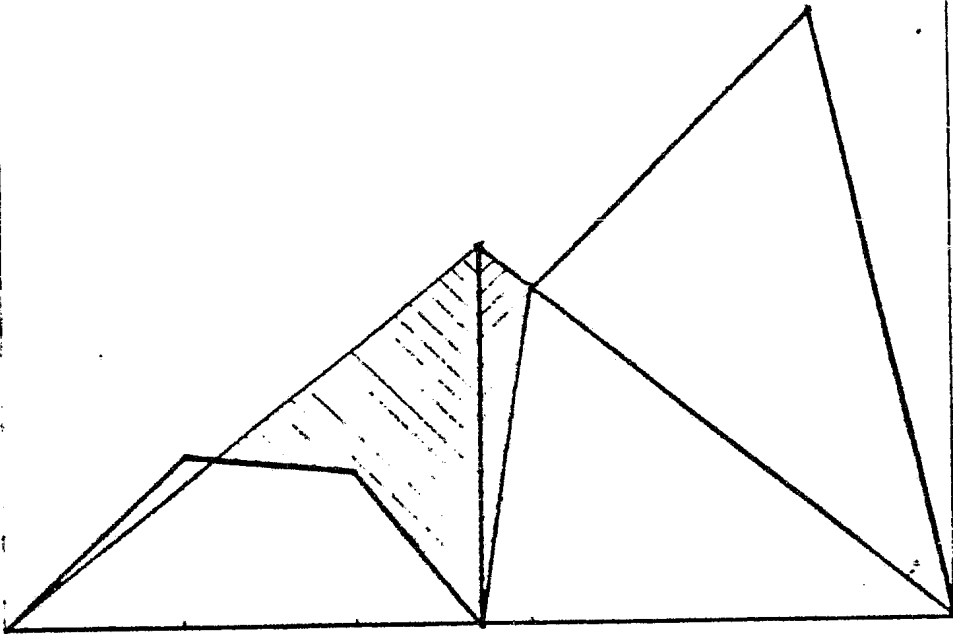
NO SON NECESARIOS LOS ATIESADORES.

# DISEÑO DEFINITIVO TRABES LONGITUDINALES

①

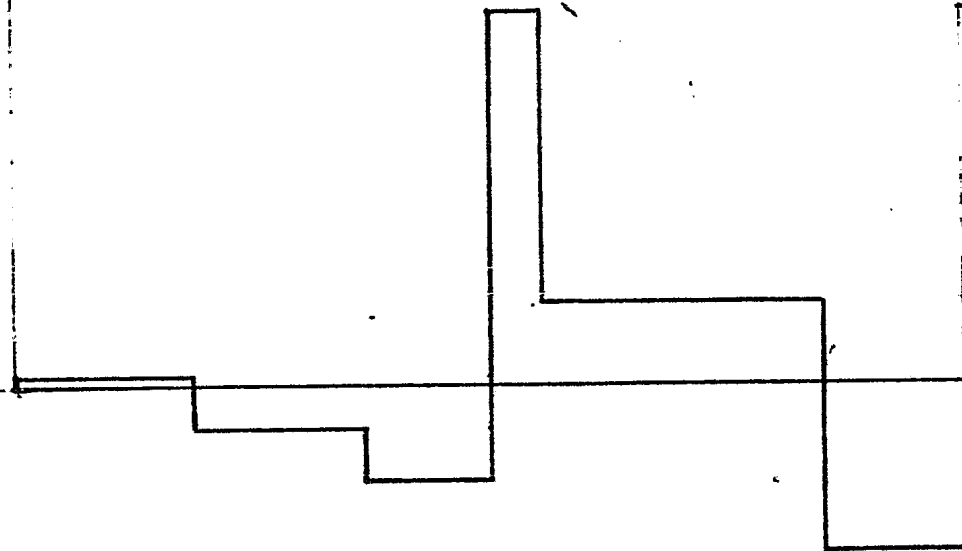
TRABE EJE (C) NIVEL T/S+4

②



ESC. HOR 1:50  
ESC. VER 1:25

DIAGRAMA DE MOMENTOS



ESC. VER 1:100

DIAGRAMA DE CORTANTES

## DISEÑO DEFINITIVO TRABES LONGITUDINALES

TRABE EJE (C) NIVEL T/S+4  
ENTRE EJES (1) Y (2)

$$\begin{array}{l} \text{CM+CV} \\ M=1.34 \text{ TON-M} \\ V=4.03 \text{ TON} \end{array} \left\{ \begin{array}{l} \text{CM+CV+SISMO} \\ M=1.34 \text{ TON-M} \\ V=4.03 \text{ TON} \\ C=10.52 \text{ TON} \end{array} \right.$$

$$L = 500 \text{ cm}$$

$$L_x = 250 \text{ cm}$$

$$L_y = 150 \text{ cm}$$

$$L_b = 150 \text{ cm}$$

SI PROBAMOS CON UNA I.P.B. 10x5<sup>3</sup>/<sub>4</sub>x31.3

$$\begin{array}{l} A = 39.93 \text{ cm}^2 \\ d = 25.1 \text{ cm} \\ b_f = 14.6 \text{ cm} \\ t_f = 0.86 \text{ cm} \\ t_w = 0.61 \text{ cm} \\ d = 1.99 \text{ cm}^{-1} \\ A_f \end{array} \left. \begin{array}{l} I_x = 4424 \text{ cm}^4 \\ S_x = 352 \text{ cm}^3 \\ r_x = 10.52 \text{ cm} \\ I_y = 403.7 \text{ cm}^4 \\ r_y = 3.17 \text{ cm} \\ r = 1.6 \text{ cm} \end{array} \right\} \text{PROPIEDADES.}$$

REVISION SI ES "COMPACTA"

$$150 < \frac{20 \cdot 14.6 \cdot 14.6}{\sqrt{2.53}} = 189 \quad \text{ES CORRECTO}$$

$$150 < \frac{1475.56}{1.99 \cdot 2.53} = 279 \quad \text{ES CORRECTO}$$

$$\frac{b_f}{2t_f} = \frac{14.6}{2 \cdot 0.86} = 8.5 < \frac{25.185}{\sqrt{2.53}} = 15.8 \quad \text{ES CORRECTO.}$$

$$8.5 < \frac{17.2315}{\sqrt{2.53}} = 10.8 \quad \text{ES CORRECTO}$$

$$f_a = \frac{10.52 \times 10^3}{39.93} = 263 \text{ kg/cm}^2$$

$$C_x = C_y = 1.0$$

$$\frac{F_{crx}}{F_y} = \frac{1.0 \cdot 252}{252} = 1 \Rightarrow F_{bx} = 18,226 \text{ kg/cm}^2$$

$$\frac{F_{cry}}{F_y} = \frac{1.0 \cdot 150}{3.17} = 47 \Rightarrow F_{by} = 1311 \text{ kg/cm}^2$$

$$\left. \begin{array}{l} F_{bx} = 18,226 \text{ kg/cm}^2 \\ F_{by} = 1311 \text{ kg/cm}^2 \end{array} \right\} \frac{f_a}{F_y} = 0.10 < 0.16$$

$$\frac{r}{t_w} = \frac{25.1}{0.61} = 41 < \frac{169.4639}{\sqrt{2.53}} \left( 1 - 3.74 \cdot \frac{0.263}{2.53} \right) = 65$$

∴ LA SECCION ES COMPACTA ⇒  $F_{Bx} = 1670 \text{ kg/cm}^2$

$$\frac{f_a}{F_a} = 0.20 > 0.15$$

$$f_{bx} = \frac{1.34 \times 10^5}{352} = 381 \text{ Kg/cm}^2$$

$$FB_x = \left(1 - \frac{f_a}{F_{ex}}\right) = \left(1 - \frac{363}{18226}\right) 1670 = 1646 \text{ Kg/cm}^2$$

$$C_{mx} = 1.0$$

$$\frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F_{ex}}\right) FB_x} = 0.23$$

$$0.17 + 0.23 = 0.44 < 1.33$$

Adicionalmente

$$\frac{263}{1520} + \frac{\frac{1.02 \times 10^5}{352}}{1670} = 0.35 < 1.33$$

CORTANTE

$$f_v = \frac{4.31 \times 10^5}{25.1 \times 0.61} = 281 \text{ Kg/cm}^2 < F_v = 0.4 \times 2530 = 1012 \text{ Kg/cm}^2$$

Revisión si se requieren atiesadores de carga.

$$N = 0$$

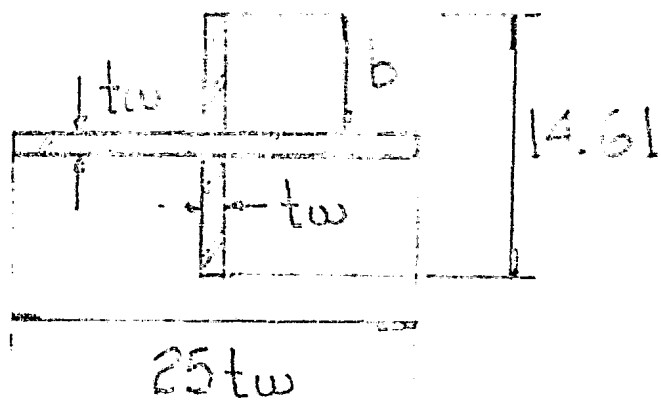
$$R = 5200 \text{ Kg}$$

$$t_w = 0.61 \text{ cm}$$

$$k = 1.6 \text{ cm}$$

$$\frac{5200}{0.61(0.7 + 2 \times 1.6)} = 2664 \text{ Kg/cm}^2 > 1890 \text{ Kg/cm}^2$$

∴ Si se requieren atiesadores de carga.



$$\begin{aligned}
 tw &= 0.61 \text{ cm} \\
 25tw &= 15.25 \text{ cm} \\
 b &= 7.0 \text{ cm}
 \end{aligned}$$

$$A = 15.25 \times 0.61 + 2 \times 7 \times 0.61 = 17.84 \text{ cm}^2$$

$$I = \frac{15.25 \times 0.61^3}{12} + 2(7 \times 0.61 \times 3.91^2) + 2\left(\frac{0.61 \times 7^3}{12}\right) = 158.8 \text{ cm}^4$$

$$r = \sqrt{\frac{158.8}{17.84}} = 2.98 \text{ cm.}$$

$$K = 0.75$$

$$\lambda = h = 25.1 \text{ cm.}$$

$$\left. \begin{aligned}
 f_a &= \frac{5200}{17.84} = 291 \text{ Kg/cm}^2 \\
 \frac{K\lambda}{r} &= 6 \Rightarrow F_a = 1501 \text{ Kg/cm}^2
 \end{aligned} \right\} \frac{f_a}{F_a} = 0.19 < 1.0$$

$$\frac{b}{t} \leq \frac{800}{\sqrt{F_y}} \left\{ \begin{aligned}
 \frac{b}{t} &= \frac{7}{0.61} = 11.5 \\
 \frac{800}{\sqrt{F_y}} &= 16
 \end{aligned} \right.$$

$$I > \left(\frac{h^4}{50}\right) = \left(\frac{25.1}{50}\right)^4 = 0.06 \text{ cm}^4$$

Revisión si se requieren atiesadores intermedios o rigidez

$$Q = 250 \text{ cm}$$

$$h = 25.1 - 2 \times 0.86 = 23.38 \text{ cm.} \left. \begin{aligned}
 & \\
 t &= 0.61 \text{ cm}
 \end{aligned} \right\} \frac{h}{t} = 38 < 260 ; \frac{Q}{h} = 11 > 3$$

$$\text{Si } \frac{Q}{h} > 1 \Rightarrow k = 5.34 ; C_v = 1.55 \Rightarrow F_{cr} = 1357 \text{ Kg/cm}^2$$

$\therefore F_c = 1012 \text{ Kg/cm}^2 > 281 \text{ Kg/cm}^2$ . Y no se requieren atiesadores.

## DISEÑO DEFINITIVO CONTRAVENTEO VERTICAL.

Eje ① del nivel T/S + 0 al T/S + 2 entre ejes ② y ③

$$\begin{array}{l|l|l} \text{C.M.+E.O} & \text{Sismo} & \text{C.M.+E.O+S} \\ \hline \text{C}=2.45 \text{ ton.} & \text{C}=10.77 \text{ ton.} & \text{C}=13.22 \text{ ton.} \end{array}$$

Si revisamos el perfil propuesto

$$\text{T.P.R } 5 \times 5^{3/4} \times 18.65$$

Propiedades :

$$A = 23.71 \text{ cm}^2$$

$$r_y = 3.32 \text{ cm}$$

$$K = 1.0$$

$$l = (2.5^2 + 4^2)^{1/2} = 4.72 \text{ mts.}$$

$$\frac{Kl}{r} = \frac{1.0 \times 472}{3.32} = 142 \Rightarrow F_a = 531 \times 1.33 = 706 \text{ Kg/cm}^2$$

$$C_{resist.} = 706 \times 23.71 = 16,739 \text{ Kg} > 13,220 \text{ Kg}$$

$$\eta = 79\%$$

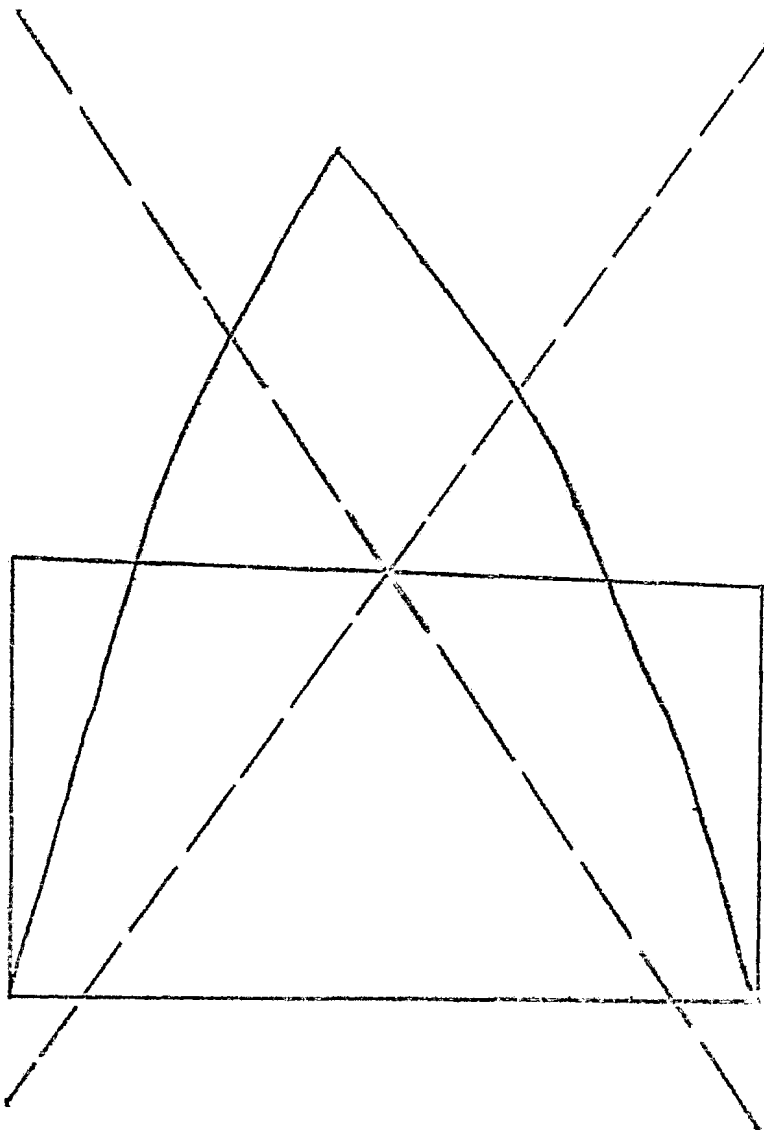
∴ La sección propuesta es correcta.

TRASE EJE (2) DEL NIVEL T<sub>3</sub>+2

SISMO

ESC. HOR. 1:75 LONGTS.  
ESC. VER. 1:300 110 Ton.

DIAGRAMA DE CORTANTES



ESC. VER 1:400 10 T-M

DIAGRAMA DE MOMENTOS

(C)

(D)



## DISEÑO DEFINITIVO TRABES TRANSVERSALES PRINCIPALES

Trabe eje ② del nivel T/S+2 entre ejes ③ y ④

Elementos mecánicos máximos:

C.M+E.O	C.M+E.O+S
$M = 18.89 \text{ t-m}$	$M = 42.25 \text{ t-m}$
$V = 18.62 \text{ ton}$	$V = 26.40 \text{ ton}$
$T = 0.33 \text{ ton}$	$C = 0.44 \text{ ton}$

Revisemos la sección propuesta: I.P.R  $18 \times 8^{3/4} \times 126.7$

Propiedades:

$A = 161.09 \text{ cm}^2$	$I_x = 59517 \text{ cm}^4$	$t_f = 2.31 \text{ cm}$
$d = 46.5 \text{ cm}$	$S_x = 2558 \text{ cm}^3$	$b_f = 22.4 \text{ cm}$
$t_w = 1.34 \text{ cm}$	$r_x = 19.23 \text{ cm}$	$k = 3.8 \text{ cm}$
$\frac{d}{A_f} = 0.9 \text{ cm}^{-1}$	$r_y = 5.01 \text{ cm}$	

$$L_b = 134 \text{ cm}$$

Revisión si la sección es compacta.

$$L_b \leq \frac{20.1416 \times 22.4}{\sqrt{2.53}} = 284 \text{ si cumple.}$$

$$L_b \leq \frac{1405.56}{0.9 \times 2.53} = 617 \text{ si cumple.}$$

$$\frac{b_f}{2t_f} = \frac{22.4}{2 \times 2.31} = 4.85 \leq \frac{25.185}{\sqrt{2.53}} = 15.83 \text{ si cumple.}$$

$$4.85 \leq \frac{17.2315}{\sqrt{2.53}} = 10.83 \text{ si cumple.}$$

Si observamos los elementos mecánicos la compresión tiende a cero. Por lo que  $f_a = 0$

$$\frac{f_a}{f_a} = 0 < 0.16$$

$$\frac{d}{t_w} = \frac{46.5 - 2 \times 2.31}{1.34} = 31.25 \leq \frac{169.4437}{\sqrt{2.55}} = 106.67 \text{ si cumple}$$

∴ La sección es compacta.

$$F_{br} = 0.66 \cdot 2530 = 1670 \text{ Kg/cm}^2$$

$$M_R = 1670 \cdot 2558 = 1.33 = 56.82 \text{ ton-m} > 42.25 \text{ ton-m} \quad \eta = 74\%$$

CORTANTE.

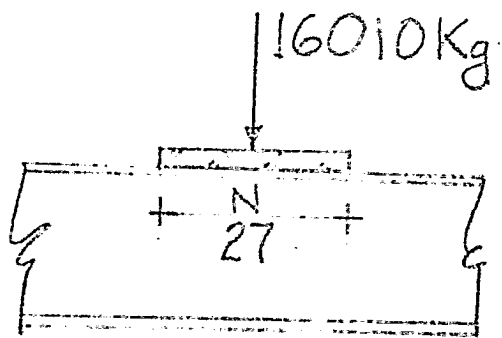
$$F_v = 1012 \text{ Kg/cm}^2$$

$$f_v = \left( \frac{264 \times 10^3}{0.61 \times 46.5} \right) 0.75 = 698 \text{ Kg/cm}^2 < 1012 \text{ Kg/cm}^2 \text{ es correcto.}$$

DEFLEXION.

$$\delta_{max.} = 0.21 < \delta_{perm.} = \frac{600}{360} = 1.67 \text{ cm es correcto.}$$

Revisión si se requieren atiesadores de carga interiores



$$\frac{16010}{1.34(27 + 2 \times 3.8)} = 345 \text{ Kg/cm}^2 < 1890 \text{ Kg/cm}^2$$

∴ No se requieren atiesadores de carga.

Revisión si se requieren atiesadores de rigidez.

$$Q = 600 \text{ cm}$$

$$h = 46.5 \text{ cm}$$

$$t = 1.34 \text{ cm}$$

$$\left. \begin{array}{l} h \\ t \end{array} \right\} \frac{h}{t} = 34.7 < 260 ; \frac{Q}{h} = 12.9 > 3$$

$$\frac{Q}{h} > 1 \Rightarrow k = 5.34$$

$$C_r = \frac{2530}{2.89} (2.11) = 1847 \text{ Kg/cm}^2 > 1012 \text{ Kg/cm}^2.$$

∴ No se requieren atiesadores de rigidez

Revisión de esfuerzos de tensión y corte combinados.

$$f_{bx} = \left(0.525 + 0.375 \cdot \frac{693}{1012}\right) 2530 = 1433 \text{ Kg/cm}^2$$

$$M_{bx} = 1433 \times 2558 \times 1.33 = 48.75 \text{ ton-m} > 42.25 \text{ ton-m}$$

$$\eta = 37\%$$

∴ Usar I.P.R 18 × 8<sup>3/4</sup> × 126.7

## DISEÑO DEFINITIVO DE COLUMNAS.

Se revisa la sección propuesta. I.P.R 18 × 8<sup>3/4</sup> × 126.7

Propiedades:

$$A = 161.09 \text{ cm}^2$$

$$S_x = 2558 \text{ cm}^3$$

$$d = 46.5 \text{ cm}$$

$$\frac{d}{A_f} = 0.9 \text{ cm}^{-1}$$

$$r_x = 19.23 \text{ cm}$$

$$b_f = 22.4 \text{ cm}$$

$$I_x = 59517 \text{ cm}^4$$

$$r_y = 5.01 \text{ cm}$$

$$t_f = 2.31 \text{ cm}$$

$$t_w = 1.34 \text{ cm}$$

Elementos mecánicos máximos.

C.M + E.O

$$P_{\max} = 72.99 \text{ ton}$$

$$M_{xs} = 8.62 \text{ ton-m}$$

$$M_{xi} = 4.87 \text{ ton-m}$$

Sismo:

$$P = 25.63 \text{ ton}$$

$$M_{xs} = 8.55 \text{ ton-m}$$

$$M_{xi} = 20.85 \text{ ton-m}$$

C.M + E.O + S

$$P = 98.62 \text{ ton}$$

$$M_{xs} = 17.17 \text{ ton-m}$$

$$M_{xi} = 25.72 \text{ ton-m}$$

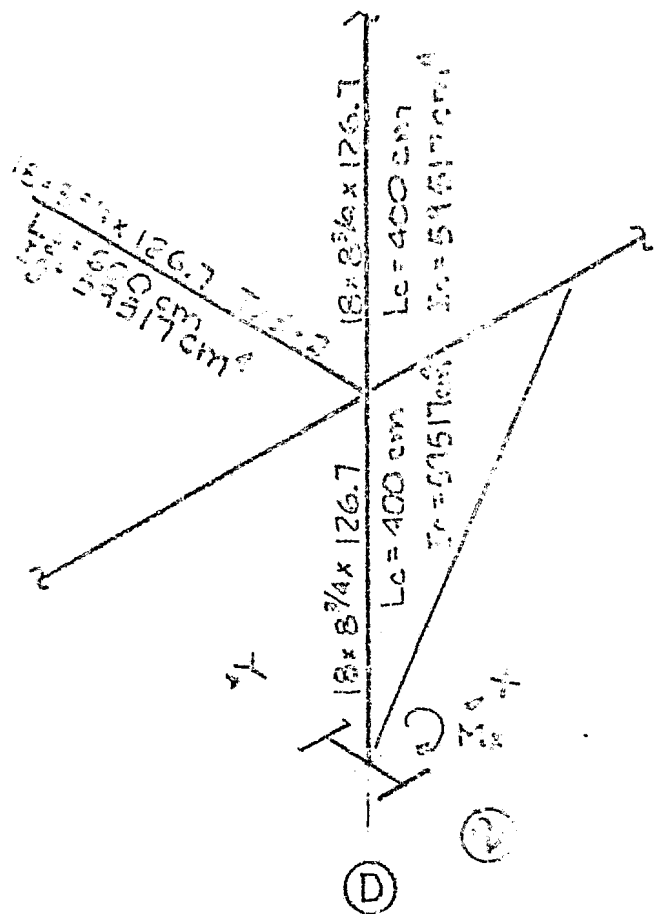
$$P_{\max} = 60.4 + 12.59 = 72.99$$

(C.M + E.O + S) 0.75

$$P = 73.97 \text{ ton}$$

$$M_{xs} = 12.88 \text{ ton-m}$$

$$M_{xi} = 19.29 \text{ ton-m}$$



$$l_x = l_y = 400 \text{ cm}$$

$$K_y = 1.0$$

Calculo de "Kx"

$$G_A = 1.0$$

$$G_0 = \frac{\sum \frac{I_c}{L_c}}{\sum \frac{I_g}{L_g}} = \frac{\frac{59517}{400} + \frac{59517}{400}}{\frac{59517}{600}} = 3 \quad \left. \vphantom{\frac{\sum \frac{I_c}{L_c}}{\sum \frac{I_g}{L_g}}} \right\} K_x = 1.57$$

$$f_a = \frac{73.97 \times 10^3}{161.09} = 459 \text{ Kg/cm}^2$$

$$\frac{K_y l_y}{r_x} = \frac{1.57 \times 400}{19.25} = 33 \Rightarrow F_{ex} = 9628 \text{ Kg/cm}^2$$

$$\frac{K_x l_x}{r_y} = \frac{1.0 \times 400}{5.01} = 80 \Rightarrow F_a = 1080 \text{ Kg/cm}^2$$

$$\frac{f_a}{F_a} = 0.43 > 0.15$$

∴ Se deben aplicar las siguientes expresiones.

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F_{ex}}\right) F_{bx}} \leq 1.0$$

$$\frac{f_a}{0.67 F_y} + \frac{f_{bx}}{F_{bx}} \leq 1.0$$

$$C_{mx} = 0.85$$

$$f_{bx} = \frac{19.20 \cdot 10^5}{2558} = 754 \text{ Kg/cm}^2$$

Calculo de  $F_{bx}$

Revisemos si la sección es compacta

$$L_b = 400 \text{ cm}$$

$$\frac{20.1416 \cdot 23.4}{\sqrt{2.53}} = 23.4 < L_b \quad \text{No es compacta}$$

$$\frac{22.4}{2 \times 2.31} = 4.85 = \frac{25.15 \bar{z}}{\sqrt{2.53}} = 15.33 \quad \text{Si cumple.}$$

$$\frac{I_b}{b^3} = \frac{400}{2.31^3} = 17.05$$

$$\frac{20.1476}{\sqrt{2.53}} = 12.67 \quad \left. \begin{array}{l} \frac{I_b}{b^3} > \frac{20.1476}{\sqrt{F_y}} \end{array} \right\}$$

$$h = 46.5 - 2 \times 2.31 = 41.88 \text{ cm}$$

$$I_y = 4137 \text{ cm}^4$$

$$r_t = \sqrt{\frac{\frac{4137}{2}}{\frac{161.99}{2} - \frac{41.88}{3} \times 1.34}} = 5.78 \text{ cm.}$$

$$\left. \begin{array}{l} \frac{I_b}{r_t^4} = \frac{400}{5.78^4} = 69 \\ \sqrt{\frac{7168.33 \times C_b}{F_y}} = 53 \end{array} \right\} \frac{I_b}{r_t^4} > \sqrt{\frac{7168.33 C_b}{F_y}}$$

$$C_b = 1$$

$$F_{bx} = \left[ \frac{2}{3} - \frac{2.53 (69)^2}{107525 \times 1} \right] 2.53 = 1402 \text{ Kg/cm}^2$$

$$\frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F_{ex}}\right) F_{bx}} = \frac{0.85 \times 754}{\left(1 - \frac{459}{9628}\right) 1402} = 0.48$$

$$0.43 + 0.48 = 0.91 < 1.0 \quad \text{Es correcto} \quad \eta = 91\%$$

Adicionalmente

$$\frac{459}{1520} + \frac{754}{1432} = 0.30 + 0.54 = 0.84 < 1.0 \quad \text{Es correcto}$$

$$\therefore \text{Usar I.P.R } 18 \times 8^{3/4} = 126.7$$

## CAPITULO 6

### DISEÑO DE CONEXIONES

En este capítulo se muestran los procedimientos a seguir para el diseño de conexiones. Se estudian solamente las siguientes conexiones:

- a). Diseño de conexión de contraventeo vertical
- b). Diseño de conexión a cortante
- c). Diseño de conexión a momento

## CONEXIONES

Diseño de conexión de contravantes vertical.

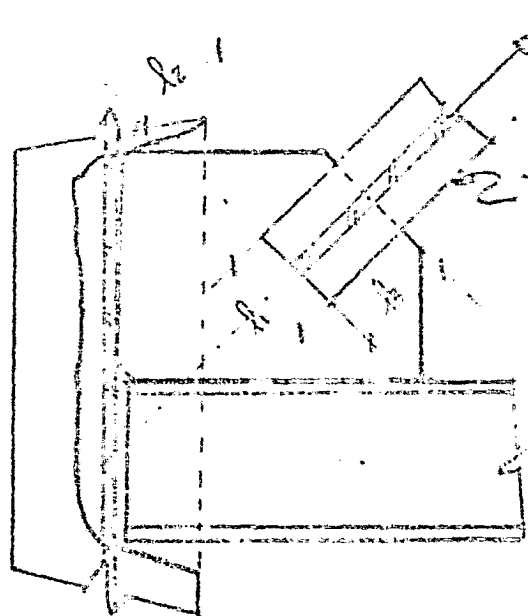
Elementos mecánicos.

$$C_{max} = 13.22 \text{ ton} \quad (C.M + E.O + S)$$

$$C_{diseño} = 13.22 \times 0.75 = 9.92 \text{ ton.}$$

Soldadura Electrodo E70XX.

$$F_s = 0.3 \times 70000 \times 0.07031 = 1476 \text{ Kg/cm}^2$$



2 T.P.R.  $5 \times 5^{3/4} \times 18.65$

$$b = 14.6 \text{ cm}$$

$$A = 23.71 \text{ cm}^2$$

$$t_f = 1.09 \text{ cm}$$

Se diseña la conexión para la capacidad máxima del contravanteo a tensión.

$$P_{max} = 0.6 F_y A$$

$$P_{max} = 0.6 \times 2530 \times 23.71 = 36,000 \text{ Kg.}$$

$$P_{max} > C_{diseño}.$$

Tamaño mínimo de soldadura =  $10.9 - 1.6 = 9.3 \text{ mm}$  ( $\frac{3}{8} = 9.5 \text{ mm}$ )

$$q_{soldadura} = 0.7071 \times 0.95 \times 1476 = 990 \text{ Kg/cm}$$

Longitud requerida de soldadura.

$$L = \frac{36000}{990} = 36 \text{ cm.}$$

$$l_1 + l_2 + l_3 = 36$$

$l_2 + l_3 = 36 - 14.6 = 21.4 \text{ cm.}$   $l_2$  y  $l_3$  deben ser de la misma longitud para evitar excentricidad.

$$l_2 = l_3 = \frac{21.4}{2} = 10.7 \approx 11 \text{ cm.}$$

## DISEÑO DE CONEXIÓN A CORTANTE (Soldada)

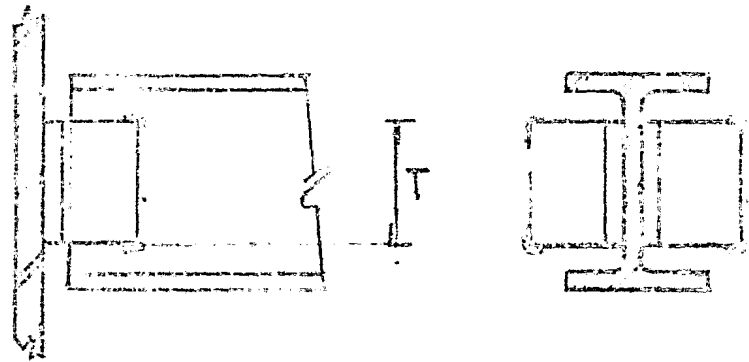
Se diseña la conexión de una I.P.R.  $18 \times 8^{3/4} \times 126.7$

$$a = 46.5 \text{ cm}$$

$$t_w = 1.34 \text{ cm}$$

$$k = 3.8 \text{ cm}$$

$$T = 39.1 \text{ cm}$$



Sección 1.5.1.2 del A.I.S.C.

Cortante:  $F_v = 0.4 F_y$

Sección 1.10.5.2

$$F_v = \frac{F_y}{2.89} C_v \leq 0.4 F_y$$

Donde:

$$C_v = \frac{45000}{F_y \left(\frac{h}{t}\right)^2} k \quad \text{cuando } C_v < 0.8$$

$$C_v = \frac{190}{\frac{h}{t}} \sqrt{\frac{k}{F_y}} \quad \text{cuando } C_v > 0.8$$

$$k = 4 + \frac{5.34}{\left(\frac{a}{h}\right)^2} \quad \text{cuando } \frac{a}{h} < 1.0$$

$$k = 5.34 + \frac{4}{\left(\frac{a}{h}\right)^2} \quad \text{cuando } \frac{a}{h} > 1.0$$

$t$  = Espesor del alma

$a$  = Distancia libre entre atiesadores transversales

$h$  = Distancia libre entre patines

Si no se ponen atiesadores.

$$a \rightarrow \infty$$

$$\frac{a}{h} \rightarrow \infty \quad \text{y} \quad \frac{4}{\frac{a}{h}} \rightarrow 0 \quad \therefore k = 5.34$$



Para:  $\frac{h}{t} = \frac{46.5}{1.34} = 35$

Y acero A-36  $F_y = 36 \text{ KFSI}$

$$C_v = \frac{190}{35} \sqrt{\frac{534}{36}} = 2.1 > 0.8$$

$$F_u = \frac{36}{2.57} \cdot 2.1 = 26.2 > 0.4 \times 36 = 14.4$$

$$\therefore F_u = 14.4 \times 70.31 = 1012 \text{ Kg/cm}^2$$

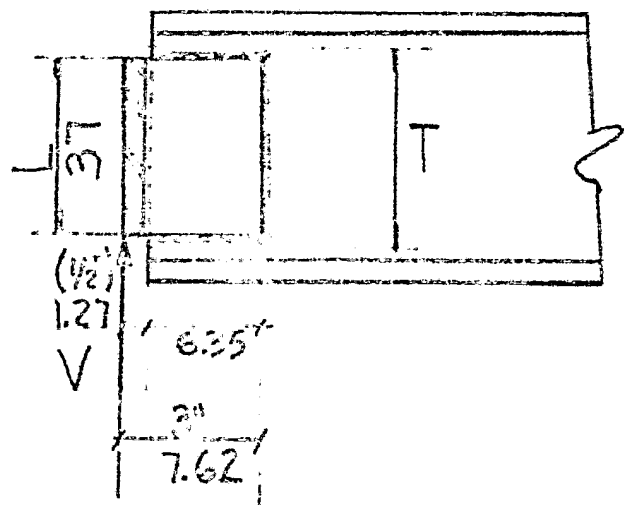
$$V = A_v F_u$$

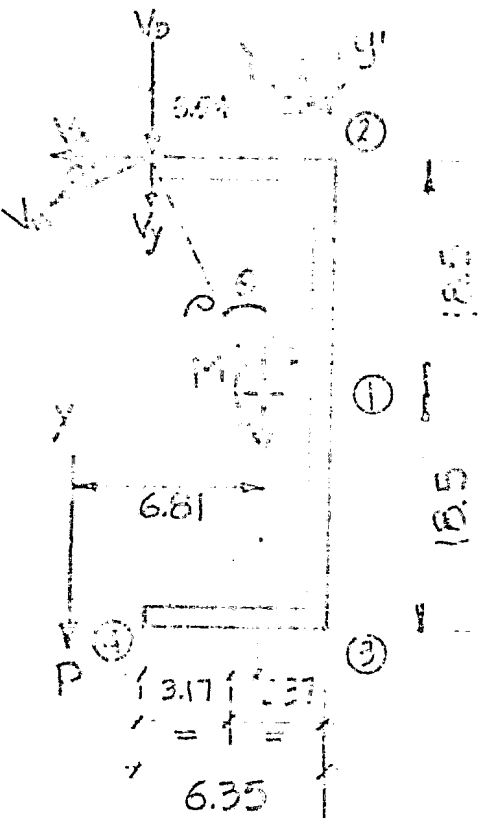
Donde  $A_v$  se puede tomar como el espesor del alma multiplicada por el peralte total de la traba (sección 1.5.1.2)

La soldadura de la placa al alma estara sujeta a cortante directo y a torsión.

$$V_D = \frac{V}{L}$$

$$V_M = \frac{V \cdot e \cdot \rho}{J} \begin{cases} V_{Mx} = \frac{V \cdot e}{J} y \\ V_{My} = \frac{V \cdot e}{J} X \end{cases}$$





$$A\bar{X} = \text{Mestatico}$$

$$\bar{X} = \frac{\text{Mestatico}}{A}$$

$$\text{Mestatico} = \left( \frac{6.35 \cdot 1}{2} \right)^2 = 6.35^2 \cdot \frac{1}{2} = 40.32 \cdot \frac{1}{2}$$

$$A = (2 \times 6.35 + 37) \cdot \frac{1}{2} = 49.7 \cdot \frac{1}{2}$$

$$\bar{X} = 0.81 \text{ cm.}$$

$$M = 6.81 P$$

$$P = (18.5^2 + 5.54^2)^{1/2} = 19.31 \text{ cm}$$

$$I_x = \frac{37^3}{12} + 2(6.35 \times 18.5^2) = 8568 \text{ cm}^4$$

$$I_y = \frac{2 \times 6.35^3}{12} + 2 \times 6.35 \times 2.37^2 + 37 \times 0.81^2 = 138 \text{ cm}^4$$

$$J = 8568 + 138 = 8706 \text{ cm}^4$$

$$V_D = \frac{P}{L} = \frac{P}{49.7} = 0.0201 P$$

$$V_M = \frac{6.81 P}{8706} \cdot 19.31 = 0.0151 P \quad \left\{ \begin{array}{l} V_{Mx} = \frac{6.81 P}{8706} \cdot 18.5 = 0.0145 P \\ V_{My} = \frac{6.81 P}{8706} \cdot 5.54 = 0.0043 P \end{array} \right.$$

$$V_v = V_D + V_{My} = (0.0201 + 0.0043) P = 0.0245 P$$

$$V_h = 0.0145 P$$

$$V_{\max} = \sqrt{0.0245^2 + 0.0145^2} \times P = 0.0284 P$$

Electrodos E70XX (21 KIPS)

$$0.0284 P = 0.707 h \times 1476$$

$$P = \frac{1066}{0.0284} h = 36741 h$$

$$\text{Si } h = \frac{5}{16} = 0.7938 \text{ cm}$$

$$P = 0.7938 \times 36741 = 29163 \text{ Kg.}$$

$$V_{\text{max}} = 0.0284 \times 29163 = 829 \text{ Kg}$$

$$P_{\text{total}} = 2 \times 29163 = 58326 \text{ Kg.}$$

Revisión del espesor del alma.

Tensión:

$$\frac{1}{2} t = \frac{829}{1520} = 0.545$$

$$t = 2 \times 0.545 = 1.09 \text{ cm} < 1.34 \text{ cm.}$$

Cortante:

$$V_v = 0.0201 P = 0.0201 \times 29163 = 586 \text{ Kg.}$$

$$V_x = 0.0145 P = 423 \text{ Kg.}$$

$$V_{y1} = V_{y2} = V_{y3} = \frac{6.81 P}{8706} \cdot 0.81 = 12 \text{ Kg}$$

En el punto ②

$$\left. \begin{array}{l} V_v = V_v - V_{y2} = 586 - 12 = 574 \text{ Kg.} \\ V_{vt} = 423 \text{ Kg.} \end{array} \right\} V = \sqrt{574^2 + 423^2} = 713 \text{ Kg.}$$

$$g = \frac{2 \times 713}{1012} = 1.4 \text{ cm} > 1.34 \text{ cm.}$$

En el punto ①

$$V = \frac{2 \times 713 + 574}{3} = 667 \text{ Kg}$$

$$g = \frac{2 \times 667}{1012} = 1.32$$

∴ Rige cortante en el punto 2

# DISEÑO DE CONEXION A MOMENTO

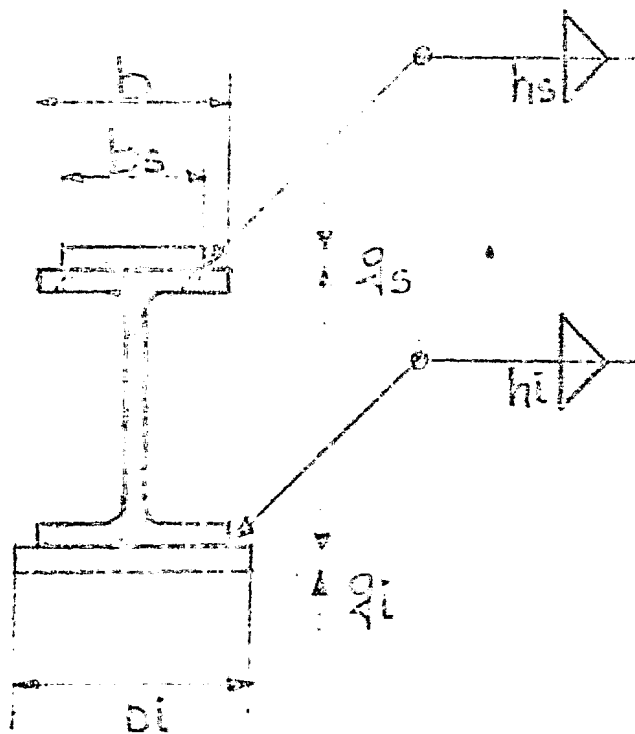
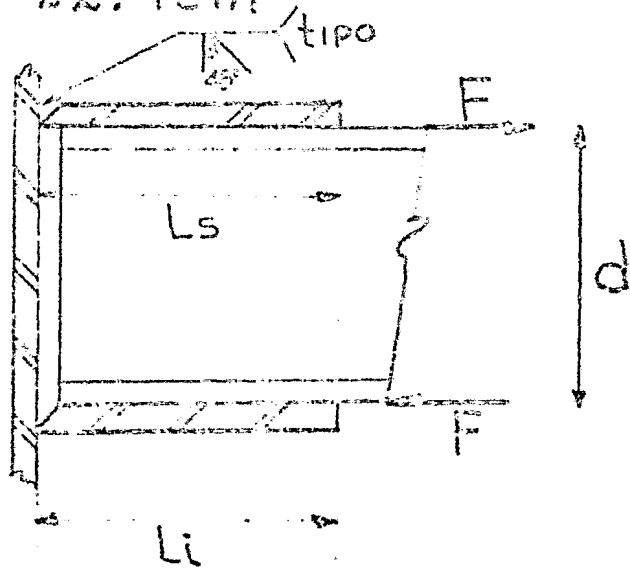
I.P.R 18 x 8<sup>3/4</sup> x 176.7

Propiedades:

$$c = 46.5 \text{ cm}$$

$$S_x = 2558 \text{ cm}^3$$

$$b = 22.4 \text{ cm}$$



$$M = Fd \Rightarrow F = \frac{M}{d}$$

Pero:

$$M = S F b_x \Rightarrow F = \frac{M}{S} F b_x$$

Se tomara  $F b_x = 0.66 F_y = 1670 \text{ Kg/cm}^2$

$$b_s = b - 2c$$

Soldadura E70XX

$$b_i = b + 2c$$

$$f_{\text{soldadura } 5/8} = 1476 \times 1.59 = 2347 \text{ Kg/cm}$$

$$q_s = \frac{F}{1520 b_s}$$

$$L_s = \frac{F}{f_{\text{res}}}$$

$$q_i = \frac{F}{1520 b_i}$$

$$L_i = \frac{F}{f_{\text{res}}}$$

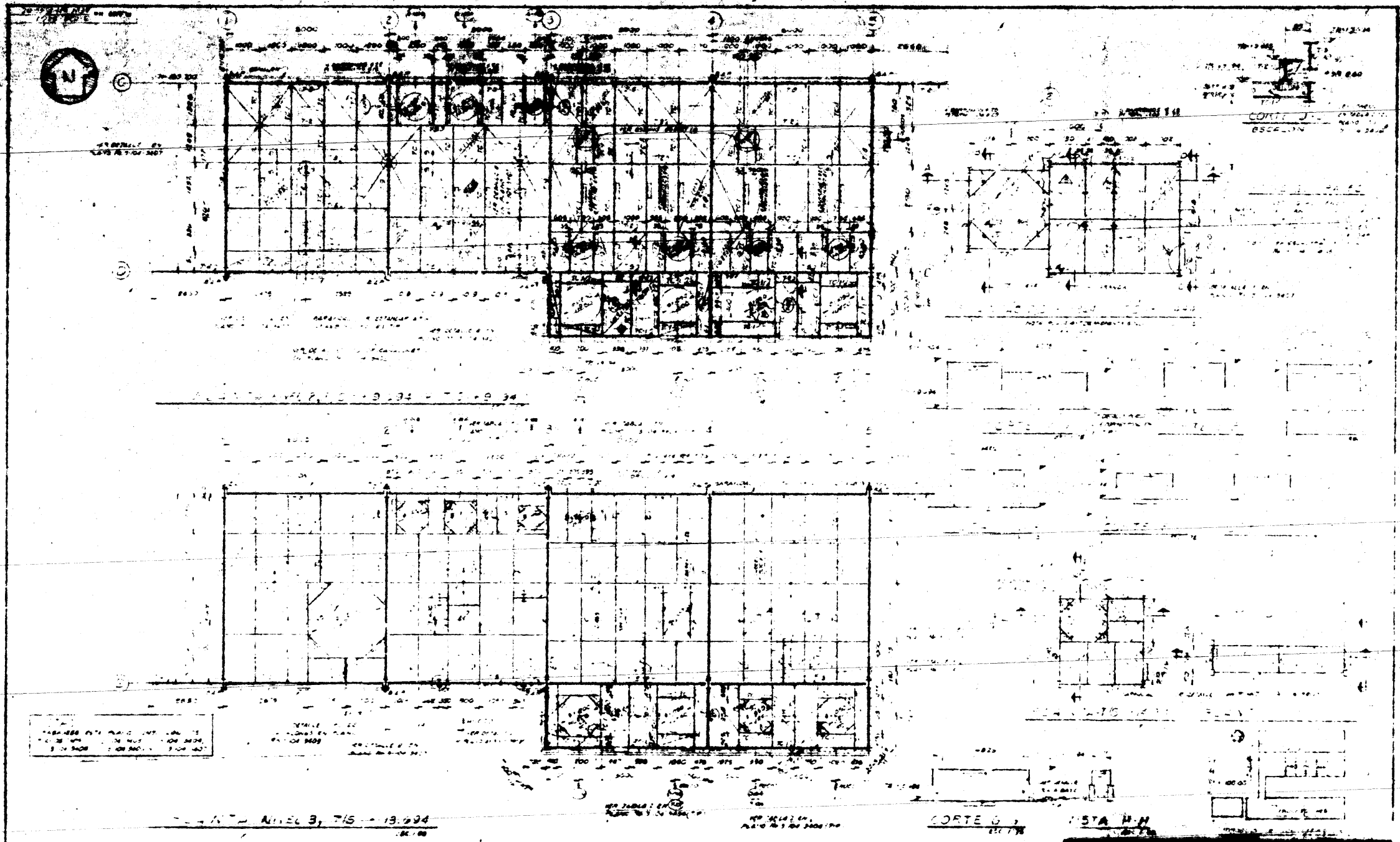
PERFIL	d	b	S	$\frac{M}{F}$	$F$	$b_s$	$b_i$	$q_s$	$q_i$	$L_s$	$L_i$	$h_s$	$h_i$	$c$
18 x 8 <sup>3/4</sup> x 176.7	36.2	22.4	2558	53	41833	27	37.6	3.61	2.33	2.54	1.59	30	1.59	20

## CAPITULO 7

### ELABORACION DE PLANOS

En este capítulo se muestra los planos estructurales en los cuales se indican plantas, elevaciones de marcos, desarrollos de escaleras, tamaños de perfiles, conexiones y detalles diversos. Se deben indicar igualmente las notas generales, las cuales deben tener los siguientes datos: acotaciones, especificaciones de acero, soldadura y tornillos, aclaraciones necesarias para fabricación y montaje, nomenclaturas, simbología, - etc.

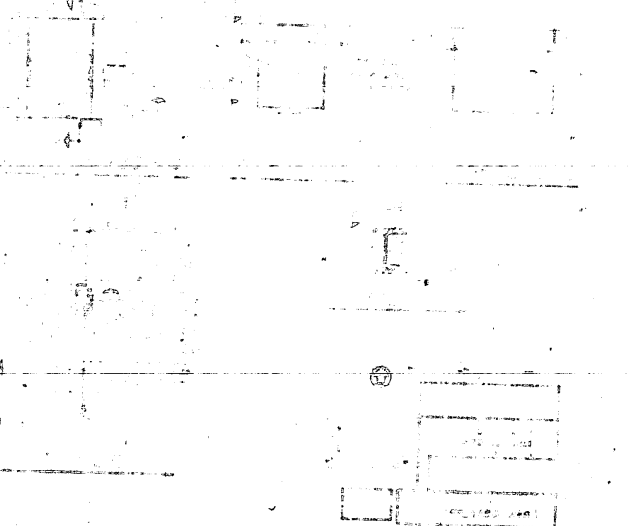
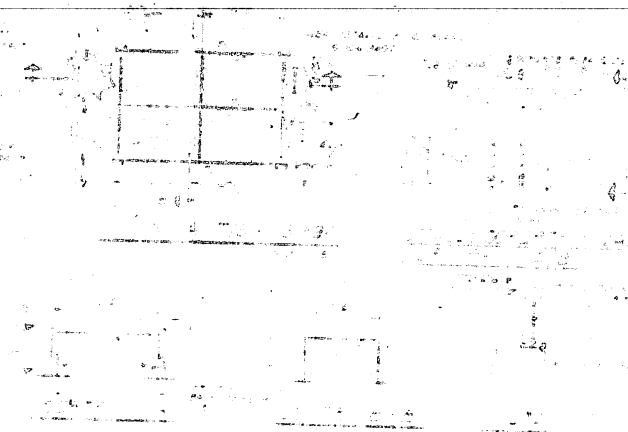
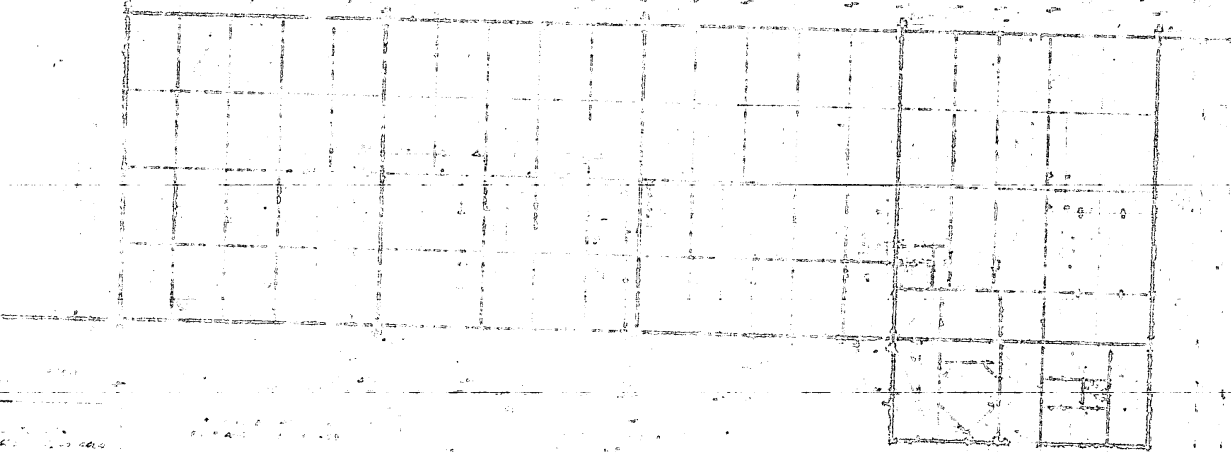
Se hace la aclaración de que estos planos fueron elaborados oficialmente por la Empresa Atlas Foster Wheeler, S. A.



Plano Nivel B, 7/5 - 19.994

CORTE G, VISTA N-N

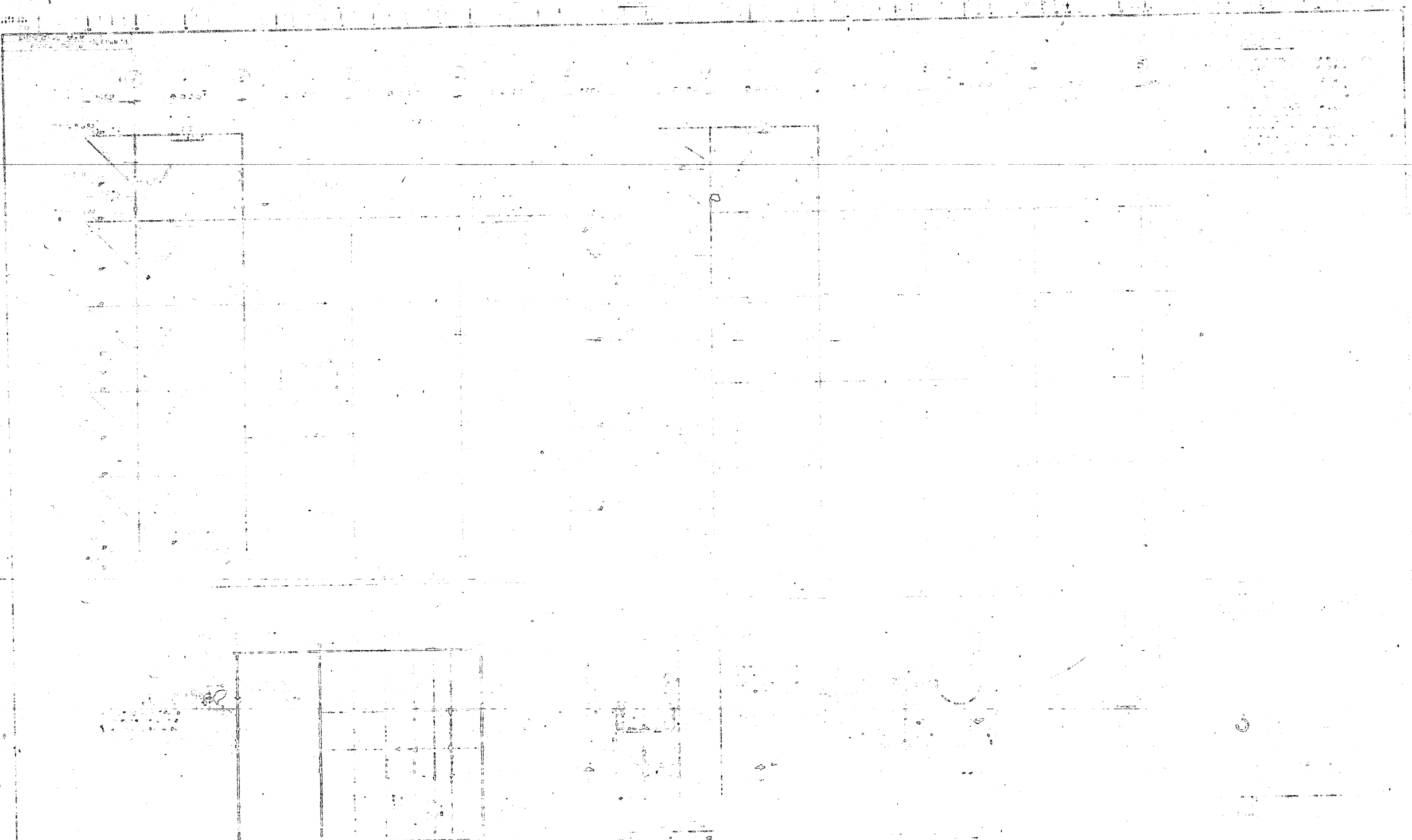
<p>1. SE DEBE ENTENDER QUE EL DISEÑO DE ESTE PROYECTO ES UN DISEÑO PRELIMINAR Y QUE SE DEBE REALIZAR UN ESTUDIO DE VIABILIDAD ANTES DE EMPEZAR LA CONSTRUCCIÓN.</p>	<p>2. EL DISEÑO DE ESTE PROYECTO SE REALIZÓ CON BASE EN LAS CONDICIONES DE LOS DATOS PROPORCIONADOS POR EL CLIENTE.</p>	<p>3. EL DISEÑO DE ESTE PROYECTO SE REALIZÓ CON BASE EN LAS CONDICIONES DE LOS DATOS PROPORCIONADOS POR EL CLIENTE.</p>	<p>4. EL DISEÑO DE ESTE PROYECTO SE REALIZÓ CON BASE EN LAS CONDICIONES DE LOS DATOS PROPORCIONADOS POR EL CLIENTE.</p>	<p>5. EL DISEÑO DE ESTE PROYECTO SE REALIZÓ CON BASE EN LAS CONDICIONES DE LOS DATOS PROPORCIONADOS POR EL CLIENTE.</p>	<p>6. EL DISEÑO DE ESTE PROYECTO SE REALIZÓ CON BASE EN LAS CONDICIONES DE LOS DATOS PROPORCIONADOS POR EL CLIENTE.</p>	<p><b>APROBADO</b> Firma del Arquitecto</p>	<p>APROBADO POR EL INGENIERO EN CARRETERAS Y OBRAS DE ACERVO</p>	<p><b>3-104-3401</b></p>
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UNAM  
Escuela de Arquitectura  
Carr. México-Toluca s/n. Ciudad de México, D.F.  
Tel. 5623 4000

SECRETARÍA DE EDUCACIÓN PÚBLICA SECRETARÍA DE ARQUITECTURA Y URBANISMO	UNAM Escuela de Arquitectura	CARRERAS DE ARQUITECTURA Y URBANISMO	CARRERA DE ARQUITECTURA	CARRERA DE URBANISMO	APROBADO PARA CONSTRUCCIÓN	UNAM	Escuela de Arquitectura Carr. México-Toluca s/n. Ciudad de México, D.F. Tel. 5623 4000
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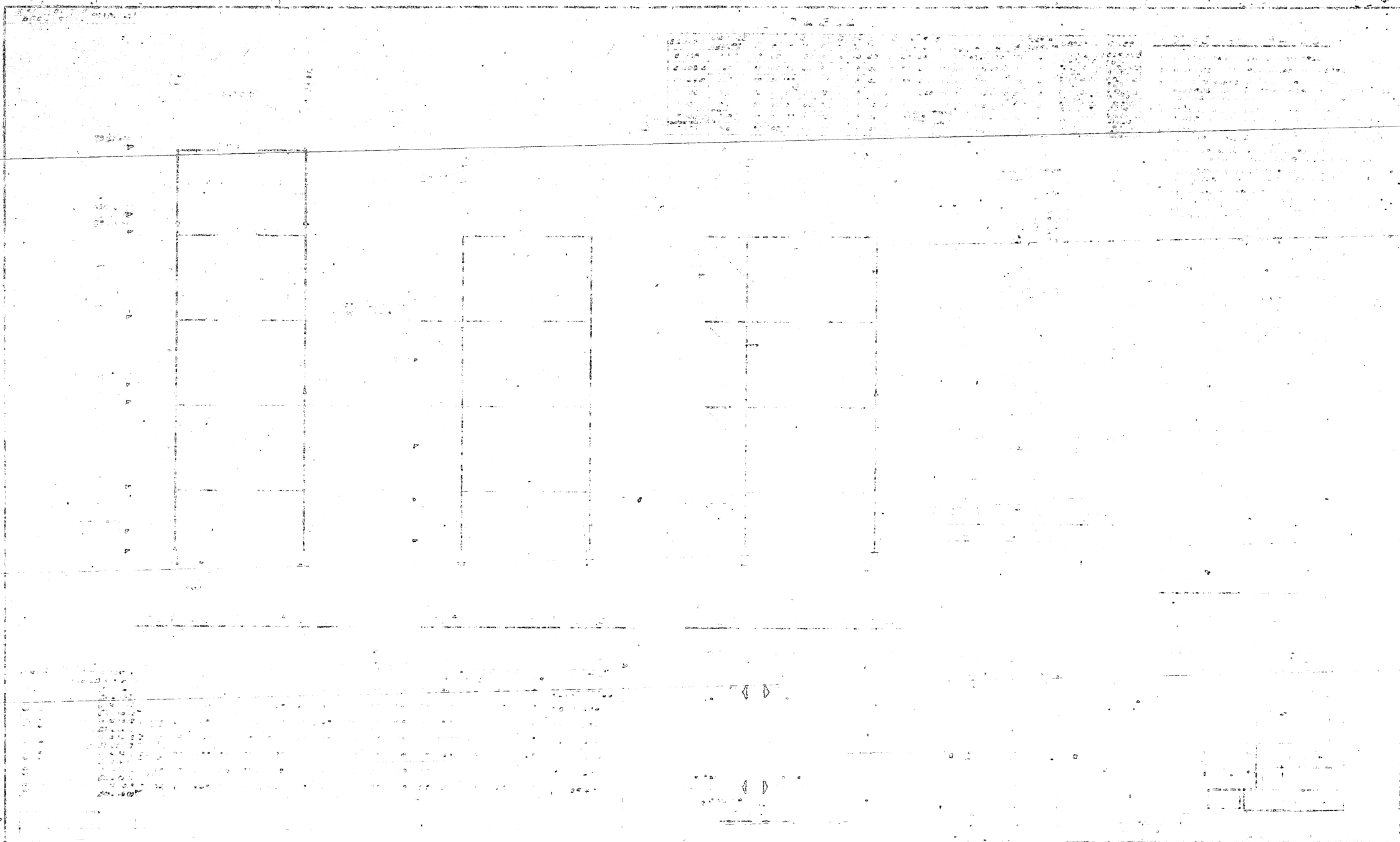
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	JESUS PEREZ SANDOZA

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APROBADO

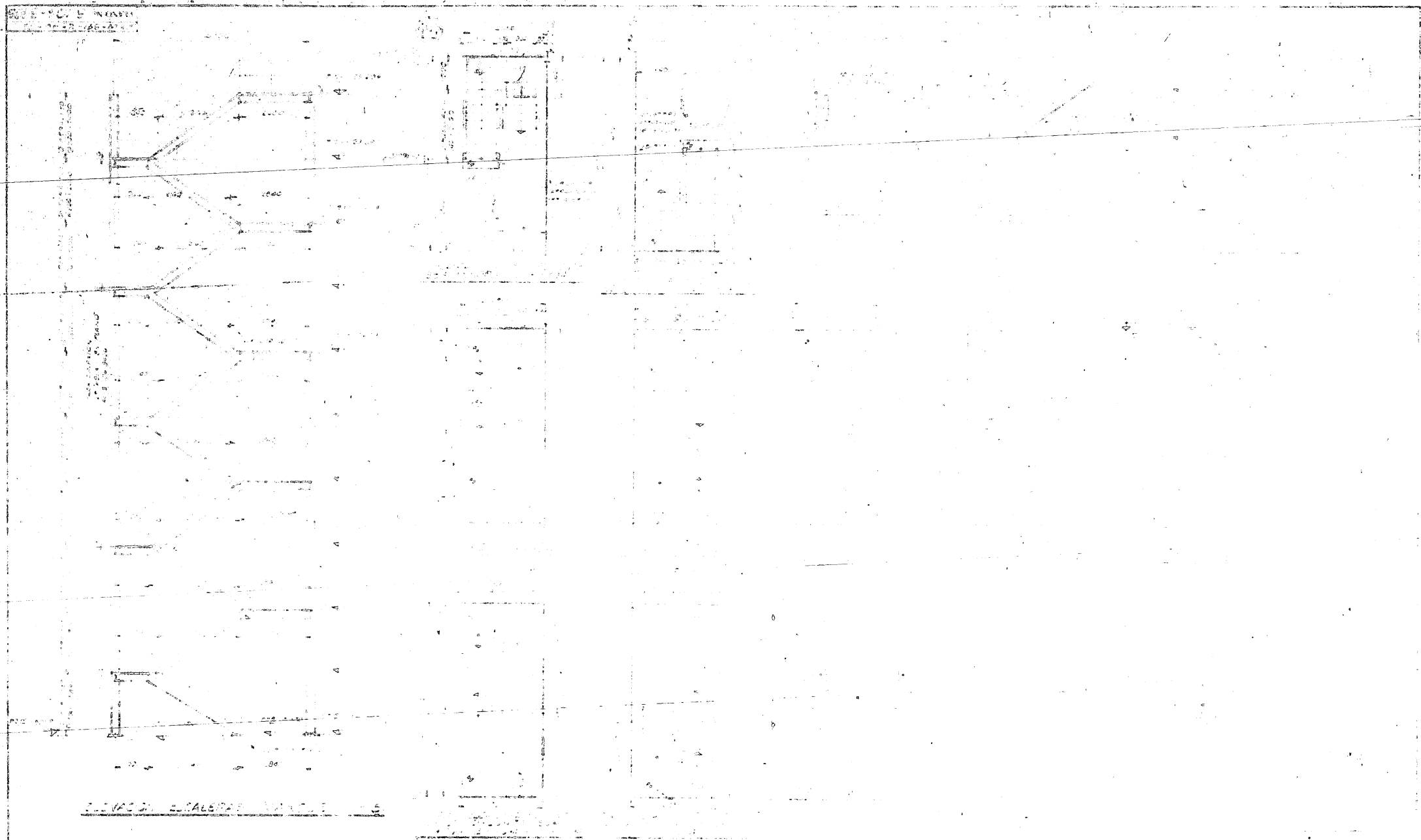
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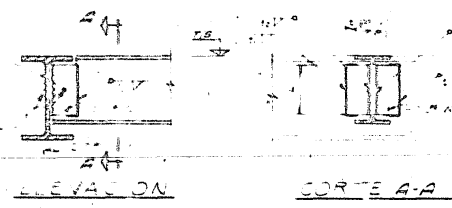
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 CATEDRA DE INGENIERIA  
 TRABAJO ESCRITO  
 -FECHA: 20/02/2014-

3-101-3405



TRABE SECUNDARIA		COLUMNA	
ALCANTARILLA	ALCANTARILLA	ALCANTARILLA	ALCANTARILLA
1.00	1.00	1.00	1.00
2.00	2.00	2.00	2.00
3.00	3.00	3.00	3.00
4.00	4.00	4.00	4.00
5.00	5.00	5.00	5.00

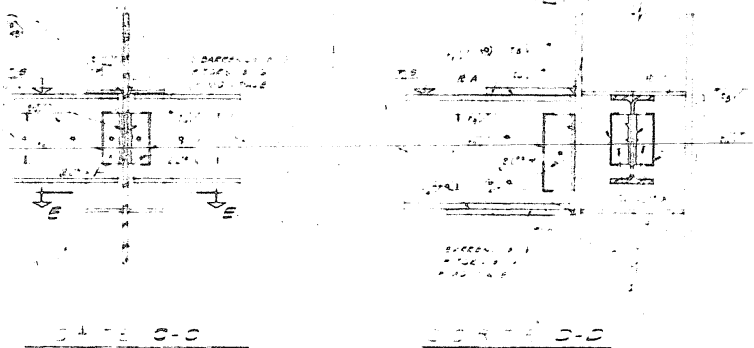
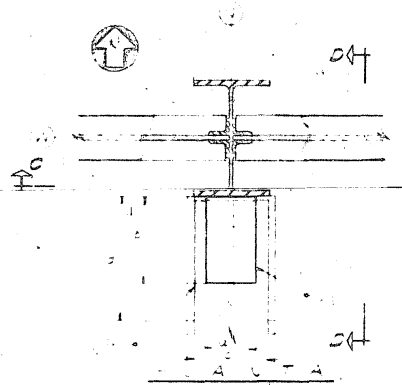


TABLA I		TABLA II	
ALCANTARILLA	ALCANTARILLA	ALCANTARILLA	ALCANTARILLA
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2.00	2.00	2.00	2.00
3.00	3.00	3.00	3.00
4.00	4.00	4.00	4.00
5.00	5.00	5.00	5.00

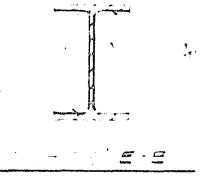
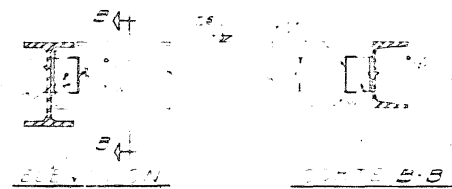
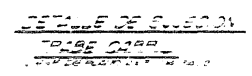
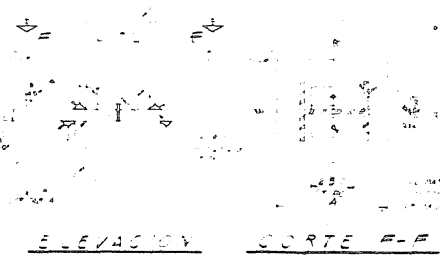


TABLA I - COEFICIENTES DE TRABES SECUNDARIAS

TABLA II - COEFICIENTES DE COLUMNAS

TABLA III		TABLA IV	
ALCANTARILLA	ALCANTARILLA	ALCANTARILLA	ALCANTARILLA
1.00	1.00	1.00	1.00
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3.00	3.00	3.00	3.00
4.00	4.00	4.00	4.00
5.00	5.00	5.00	5.00



CORTE H-H

NOTAS GENERALES

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 2. PARA NOTAS GENERALES EN PLANO A18 DE 3406  
 3. PARA NOTAS GENERALES EN PLANO A17 DE 3406  
 4. PARA NOTAS GENERALES EN PLANO A16 DE 3406  
 5. PARA NOTAS GENERALES EN PLANO A15 DE 3406

TABLA VII	
ALCANTARILLA	ALCANTARILLA
1.00	1.00
2.00	2.00
3.00	3.00
4.00	4.00
5.00	5.00

TABLA VII - EMPATES DE COLUMNAS

TABLA VIII		TABLA IX	
ALCANTARILLA	ALCANTARILLA	ALCANTARILLA	ALCANTARILLA
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4.00	4.00	4.00	4.00
5.00	5.00	5.00	5.00

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MERCANTANO          ES D.C.O. MET. MERCANTANO          ES TRUC. DE ACERO          ES MET. DE ALICATADO</p>	<p>3-104-3406A</p>
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