DISEÑO SISMICO DE ESTRUCTURAS ESPECIALES
1983

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DIVISION DE EDUCACION CONTINUA
FACULTAD DE INGENIERIA U.N.A.M.

IX CURSO INTERNACIONAL DE INGENIERIA SISMICA

DISEÑO SISMICO DE ESTRUCTURAS ESPECIALES

DISEÑO SISMICO DE EDIFICIOS DE CONCRETO PRESFORZADO

JOSE LUIS CAMBA CASTAÑEDA
Julio, 1983

DISEÑO SISMICO DE EDIFICIOS DE CONCRETO PRESFORZADO.

1.- Comportamiento de trabes en flexión.

2.- Ductilidad de miembros de concreto presforzado.

3.- Conexiones tipo en concreto presforzado.

4.- Reglamentos.

5.- Ejemplos.

José Luis Camba Castañeda.

INTRODUCCION.

La aplicación del presfuerzo en estructuras de concreto - ha tenido un incremento importante en los últimos años, debido a las ventajas que presenta sobre el concreto reforzado - principalmente en lo referente a escuadrías, a un mejor control de las deformaciones y el agrietamiento en el estado límite de servicio, bajo el efecto de cargas gravitacionales.

Sin embargo, la utilización del concreto presforzado para resistir efectos sísmicos es menos aceptada. Esto se debe principalmente a que se tiene poca información al respecto y a que comparativamente, con estructuras de concreto reforzado se observa cierto temor debido a que el primero tiene menor - capacidad para disipar energía y por tratarse de un material menos dúctil que el concreto reforzado. En las presentes notas se comentan algunos detalles del comportamiento de miembros - presforzados bajo cargas monotónicas y dinámicas, así como el detalle de conexiones y algunos lineamientos de reglamentos - de construcción referentes al concreto presforzado.

1.- COMPORTAMIENTO DE TRABES PRESFORZADAS EN FLEXION

1.1 Concepto acción respuesta.

1.2 Diagramas carga-deflexión.

1.3 Variables que intervienen en el comportamiento de trabes presforzadas.

1.4 Estado límite de Falla.

COMPORTAMIENTO DE TRABES DE CONCRETO PRESFORZADO EN FLEXION

1.- Introducción

En los ensayes de flexión de trabes presforzadas, se han hecho numerosos estudios con diferentes tipos de cargas, acciones, para conocer el comportamiento de las mismas, respuestas.

La característica acción - respuesta en trabes presforzadas se presenta, como en la mayor parte de los ensayos en flexión, mediante la gráfica carga - deflexión, de trabes libremente apoyadas con dos cargas concentradas iguales y colocadas simétricamente, esto último con objeto de que en la zona central sea nula la fuerza cortante (fig. 1).

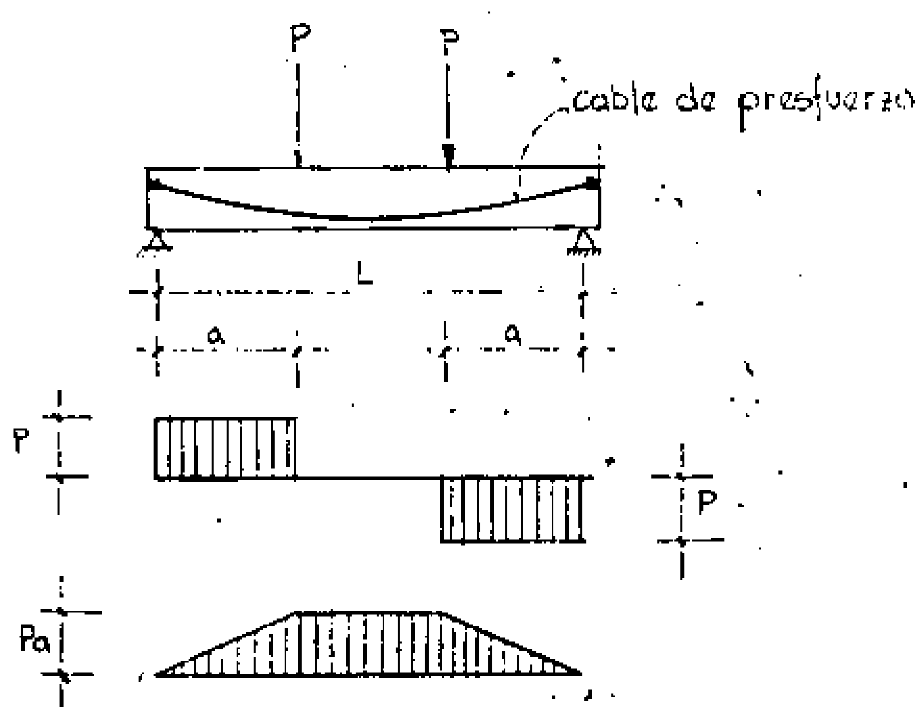


FIG. 1

En las presentes notas se estudiará la gráfica típica carga-deflexión para trabes presforzadas y posteriormente la influencia de ciertas variables en el comportamiento de las mismas.

1.2.- Diagrama carga-deflexión

Una trabe presforzada con presfuerzo excéntrico y con un porcentaje de acero de presfuerzo usual en la práctica, tiene una curva carga deflexión como lo muestra la fig. 2

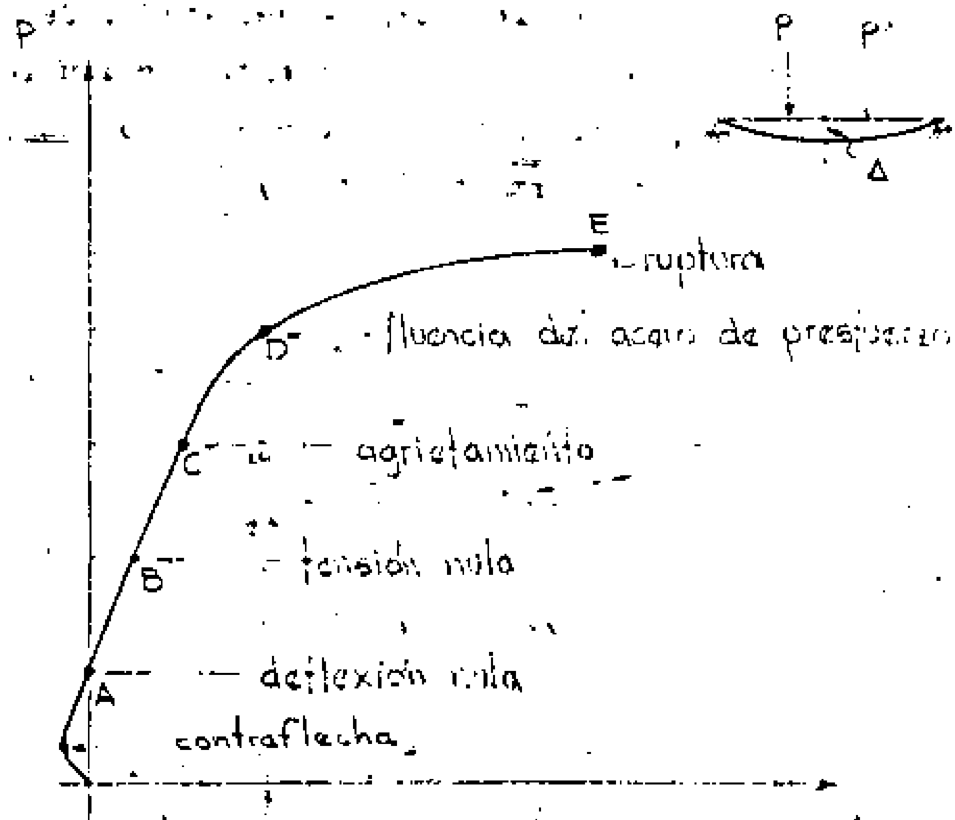


FIG. 2

Al empezar a cargar la pieza, su comportamiento es básicamente lineal, habiendo proporcionalidad entre cargas y deflexiones. La gráfica indica un valor negativo para las flechas debido a que bajo el efecto del presfuerzo, el peso propio no es suficiente para contrarestar el valor de la flecha de presfuerzo, resultando una contraflecha en la misma.

El punto A de la gráfica representa el punto de deflexión nula, que indica una distribución uniforme de esfuerzos en la sección.

Al seguir incrementando la carga, se llega al punto B, que significa el punto de tensión nula en la parte inferior.

El punto C, representa la aparición de la primera grieta, lo cual indica que el concreto alcanzó el valor de su resistencia al agrietamiento.

Cuando empiezan a aparecer las grietas, las deflexiones aumentarán mas rapidamente que antes del agrietamiento y por consiguiente ya no habrá una proporcionalidad entre cargas y deformaciones, al seguir aumentando la carga mas allá del punto C.

El punto D, representa el valor de la carga que provoca la fluencia del acero de presfuerzo.

Finalmente, el punto E, representa la carga de ruptura que provoca la falla la trabe al alcanzar su resistencia.

Conociendo la curva carga-deflexión, se puede dimensionar una trabe de concreto presforzado.

Es importante señalar que la aplicación de cargas en trabes de concreto presforzado se hace generalmente en dos o tres etapas de carga. Para una estructura colada in situ habrá la primera etapa al aplicarse el presfuerzo interviniendo también la carga permanente y una segunda etapa con las cargas de servicio. En el caso de elementos prefabricados, habrá una etapa adicional, anterior a las dos mencionadas que será solamente su peso propio y el presfuerzo, durante el transporte.

En general, la etapa crítica de carga en elementos presforzados es la que ocurre al tensar, ya que se tiene el valor de la fuerza máxima de presfuerzo por un lado y el concreto es relativamente joven, lo cual significa un "test" para el elemento en cuestión.

I.3. Reglamentación

Los nuevos reglamentos europeos para estructuras de concreto presforzado tienen un enfoque probabilístico empleando la noción de "estados límites", que permiten definir con un alto porcentaje de probabilidad, el punto que correspondería en la gráfica carga-deflexión.

1.4.- Variables que intervienen en el comportamiento de trabes presforzadas.

a) Si se incrementa el acero de presfuerzo en una trabe, aumentará también el valor del momento resistente, pero se perderá ductilidad.

b) Las trabes con presfuerzo adherido, caso del pretensado y también del postensado cuando se inyectan los cables, de acuerdo con los ensayes del laboratorio y la experiencia en la práctica, son mas dúctiles que sus equivalentes no adheridas.

c) El refuerzo no presforzado en tensión incrementa la capacidad resistente de momento, pero la trabe se hace menos dúctil. La presencia de dicho refuerzo la hace mas estable en la ruptura.

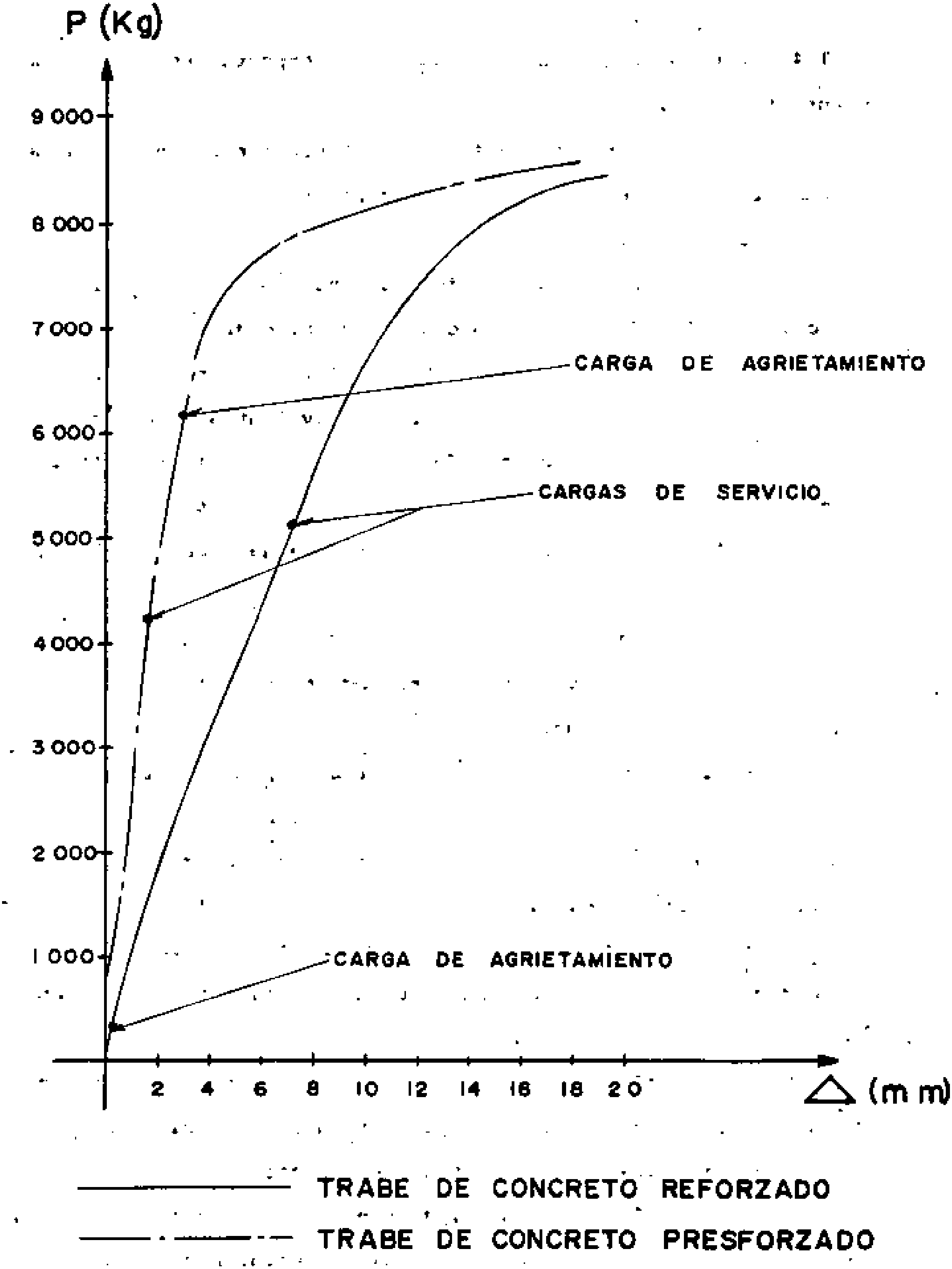
d) El refuerzo no presforzado en compresión no incrementa la capacidad de momento de una sección subreforzada pero la trabe se hace mas dúctil.

e) El comportamiento de una trabe depende de los diagramas esfuerzos-deformación de los materiales.

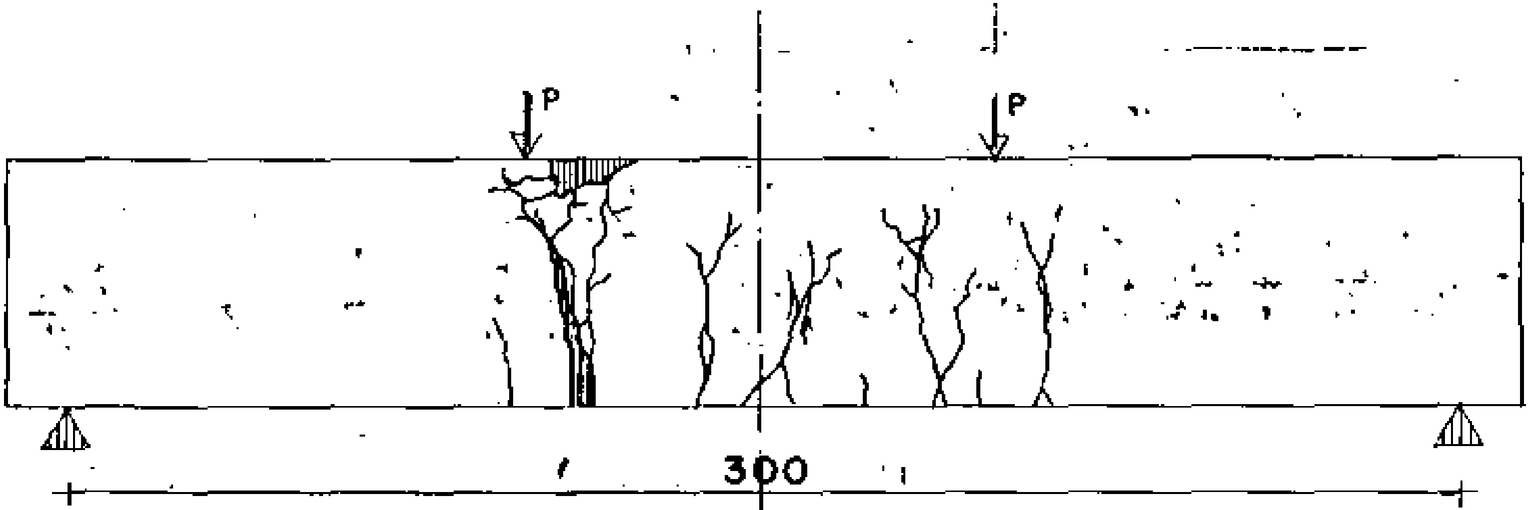
La idealización del diagrama esfuerzo-deformación del concreto en compresión, tiene poca influencia en el comportamiento de la trabe; en cambio el diagrama $f-\varepsilon$ para el acero de presfuerzo influye en el valor del momento resistente de la trabe y en la ductilidad de la misma.

En las páginas 8 a 10, se muestran la gráfica acción-respuesta de una ensaye típico por flexión de una trabe postensada y otra reforzada así como sus agrietamientos.

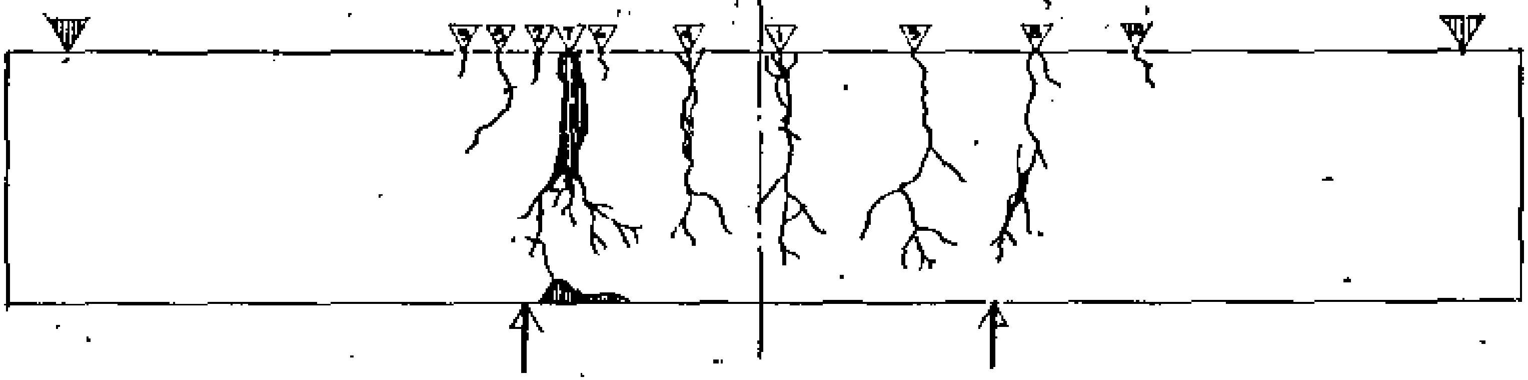
En las páginas 11 a 14, se presentan las características de elementos presforzados en la falla, incluyendo el caso de columnas.


P (Kg)
9 000
8 000
7 000
6 000
5 000
4 000
3 000
2 000
1 000
CARGA DE AGRIETAMIENTO
CARGAS DE SERVICIO
CARGA DE AGRIETAMIENTO
2
4
6
8
10
12
14
16
18
20
Δ (mm)
TRABE DE CONCRETO REFORZADO
TRABE DE CONCRETO PRESFORZADO

AGRIETAMIENTO TRABE DE CONCRETO PRESFORZADO

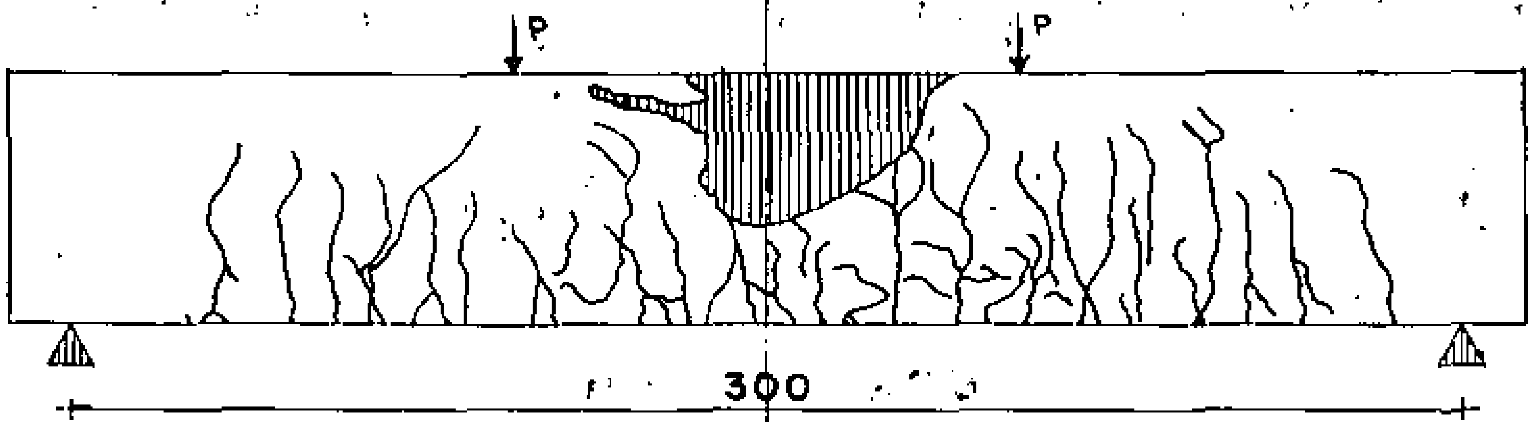


CARA SUR

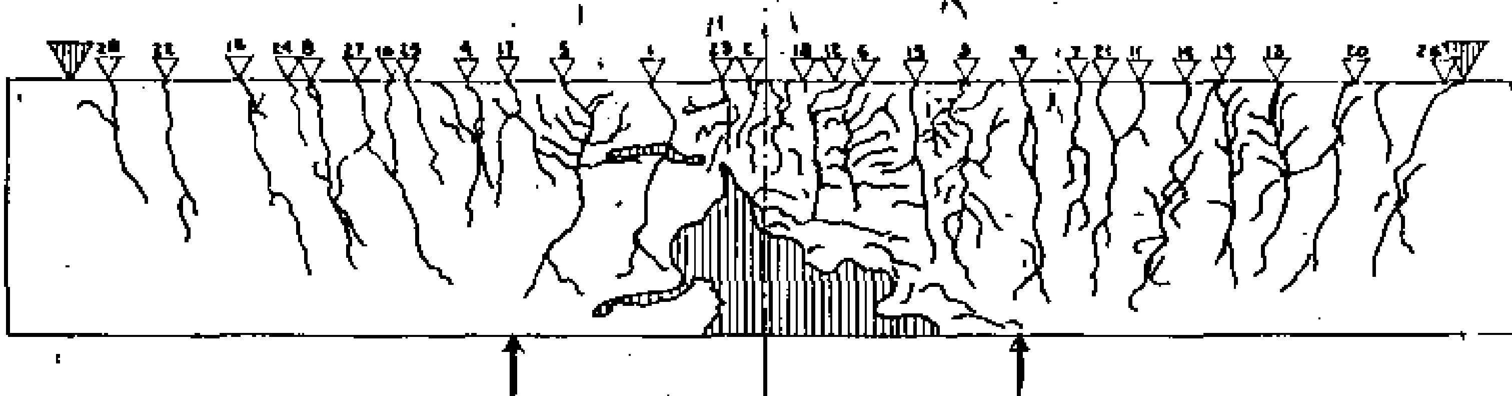


CARA NORTE

AGRIETAMIENTO TRABE DE CONCRETO REFORZADO



CARA SUR

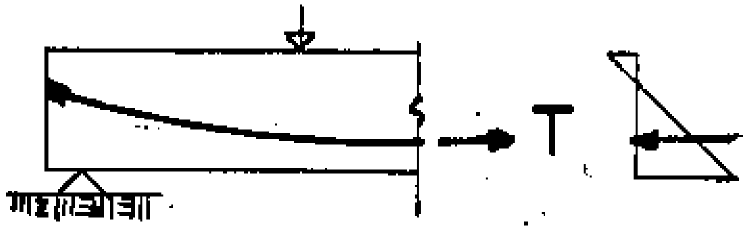


CARA NORTE

ESTADO LIMITE DE FALLA

a) Presfuerzo solamente

$P = 0$

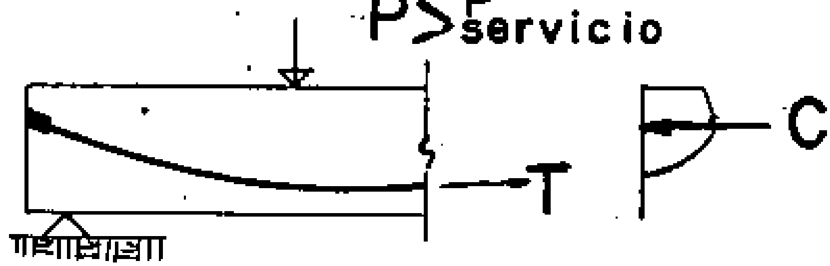


b).- En servicio

$P = P_{servicio}$

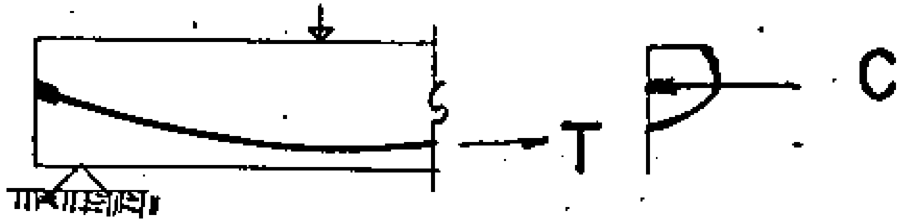
c).- Al presentarse el agrietamiento

$P > P_{servicio}$

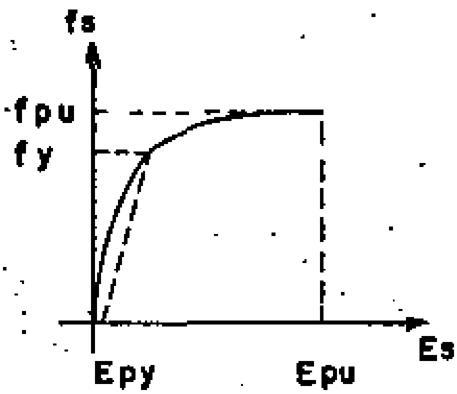


d).- En la falla

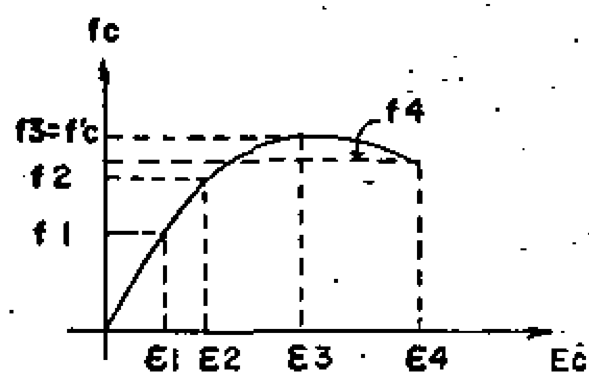
$P = P_u$



ESTADO LIMITE DE FALLA

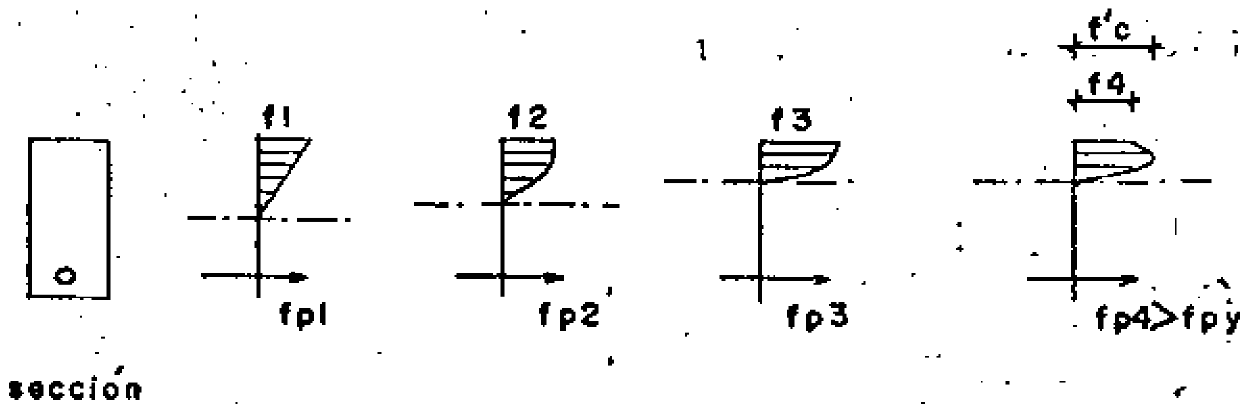


Acero de presfuerzo

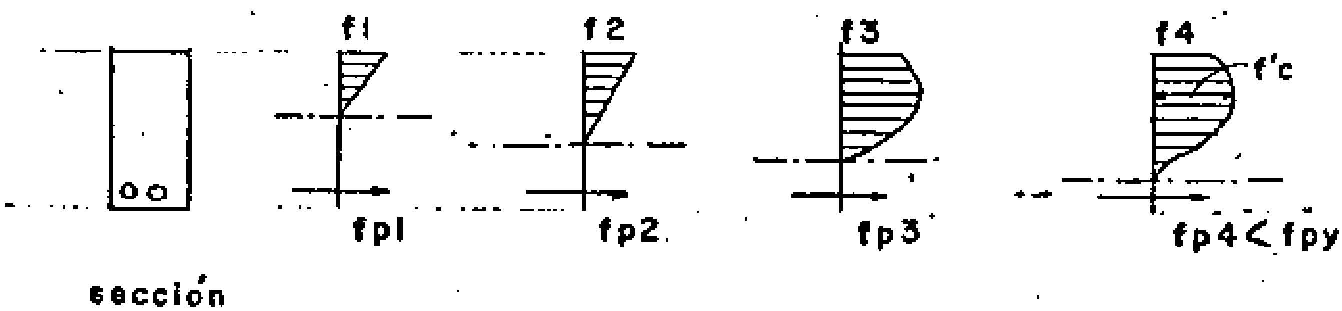


Concreto

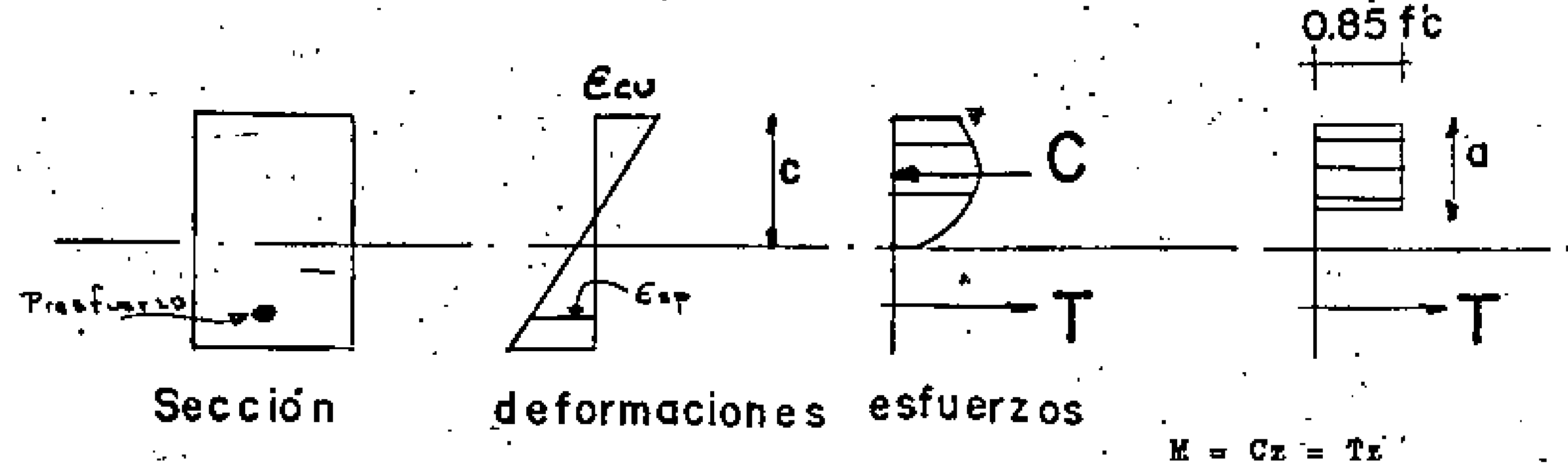
a) Trabe subreforzada



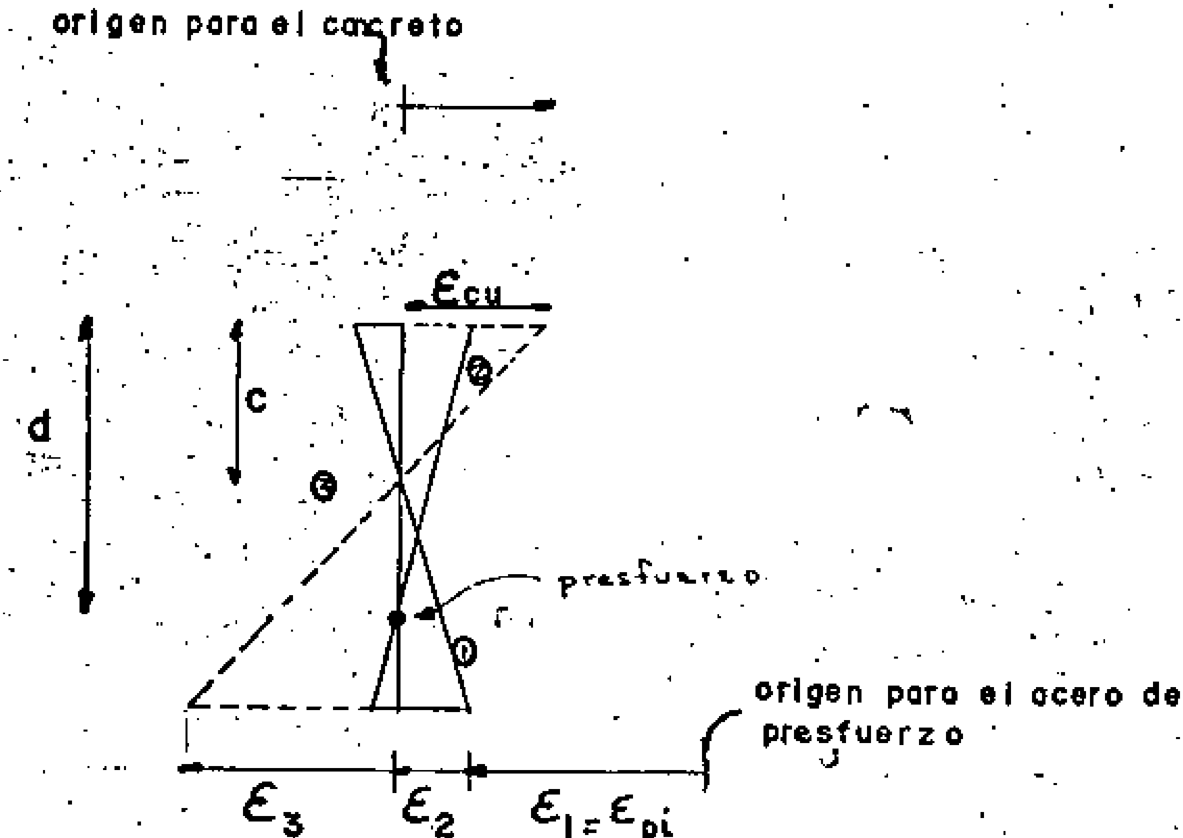
b) Trabe sobrerreforzada



ESTADO LIMITE DE FALLA



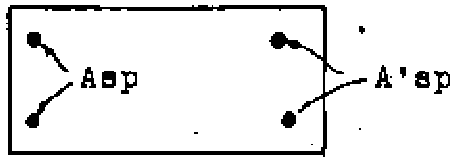
a).-Hipótesis



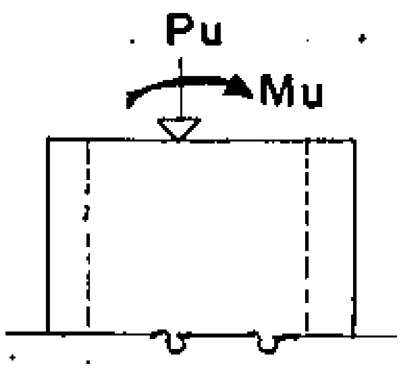
b).- Diagrama de deformaciones

COLUMNAS PRESFORZADAS

a).- Sección



b).- Elevación



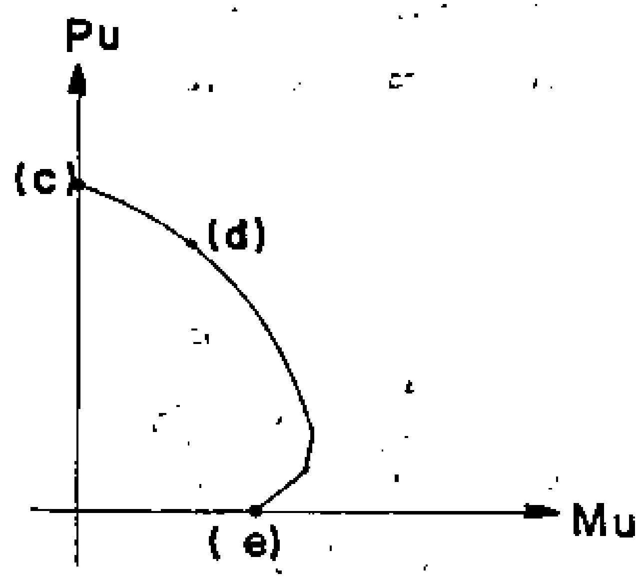
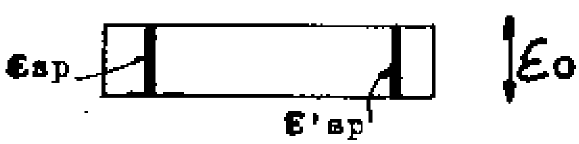
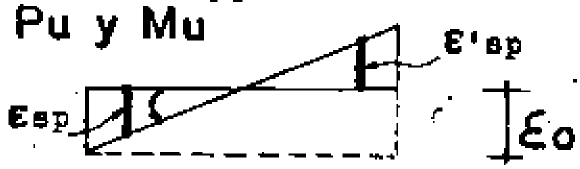


Diagrama típico de interacción

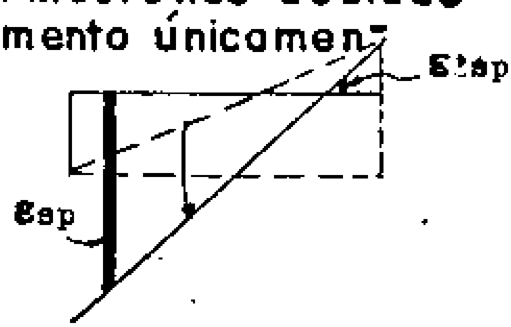
c).- Deformaciones bajo carga axial uniforme

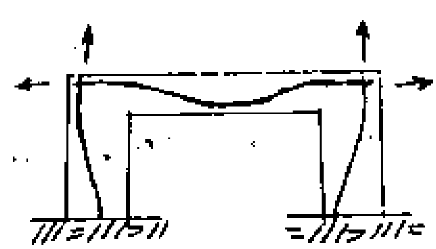


d).- Deformaciones bajo Pu y Mu



e).- Deformaciones debidas a momento únicamente





2.- DUCTILIDAD DE MIEMBROS DE CONCRETO PRESFORZADO.

2.1 Resumen histórico.

2.2 Análisis de miembros presforzados en flexión.

2.3 Amortiguamiento.

DUCTILIDAD DE MIEMBROS DE CONCRETO PRESFORZADO.

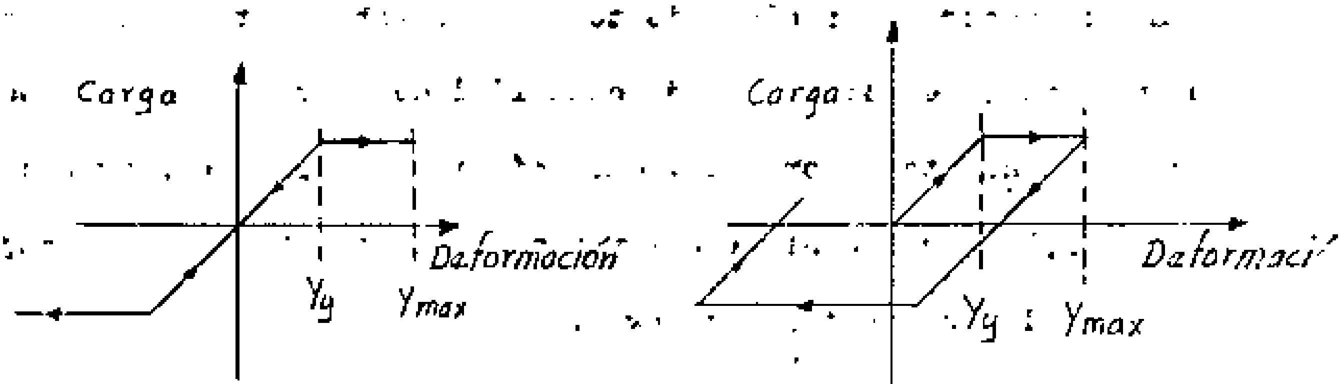
Resumen histórico de estudios realizados. Es conocido que un análisis dinámico de la respuesta elástica de estructuras usando aceleraciones sísmicas, ponen de manifiesto que una estructura puede estar sujeta a cargas mayores que las especificadas por reglamentos, lo cual implica que una estructura debe ser capaz de desarrollar grandes deformaciones antes de llegar a la falla en caso de sismos severos. Por tanto, es importante conocer la ductilidad que puede obtenerse en miembros de concreto presforzado.

La relación momento-curvatura para concreto presforzado bajo cargas monotónicas y cíclicas, permite comprender la ductilidad y la energía de disipación.

T.Y. Lin (1) presentó algunos aspectos importantes para el diseño sísmico de estructuras presforzadas, referentes a los factores de carga y esfuerzos permisibles así como algunos ensayes estudiando la capacidad de absorber energía, concluyendo Lin en su artículo en que los diagramas Momento-Curvatura en vigas de concreto presforzado en flexión se presentaban áreas importantes que mostraban alta capacidad para absorber energía.

Rosenblueth (2) comentando el artículo de Lin, enfatizaba el inconveniente de establecer conclusiones basadas en la curva de primera carga, indicando la importancia de las curvas idealizadas carga-deformación en la descarga Fig.1, para miembros de concreto presforzado y miembros de concreto reforzado presentaban que para masas y rigideces comparables una estructura de concreto presforzado tendría probablemente mayores deformaciones debido a su baja capacidad de amortiguamiento que una de concreto reforzado y que así mismo sería más flexible la primera, lo cual contrarrestaría en parte el efecto de su baja capacidad para absorber energía.

Despeyroux (3) concluye que las áreas bajo el diagrama Momento-Curvatura en concreto presforzado y reforzado son comparables y no necesariamente menores las de concreto presforzado, pero que un factor importante que afecta la respuesta sísmica de estructura es su capacidad para disipar energía, En su artículo, de acuerdo con la Fig.2 concluye que la energía absorbida es efectivamente comparable en miembros de concreto presforzado y reforzado pero que la energía disipada es bastante menor en los miembros de concreto presforzado, lo cual representará que la respuesta en estos últimos bajo el sismo será mayor.



$$\mu = \frac{Y_{max}}{Y_y}$$

(A) (B)

Fig. I Idealización de curvas típicas carga-deformación

A) concreto presforzado B) concreto reforzado

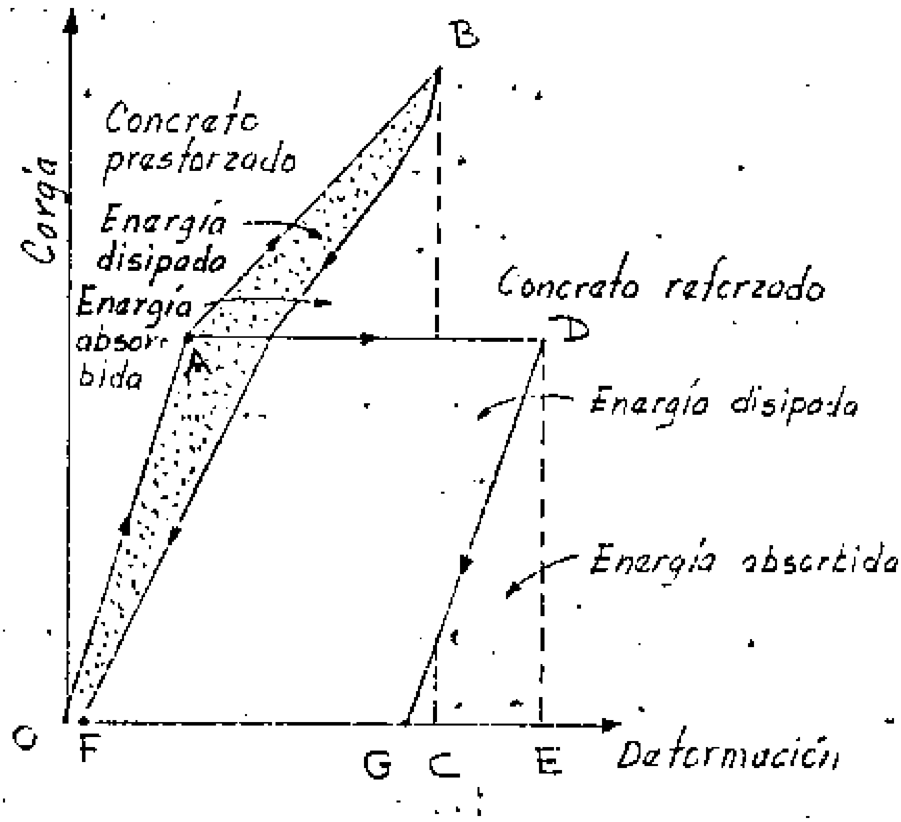


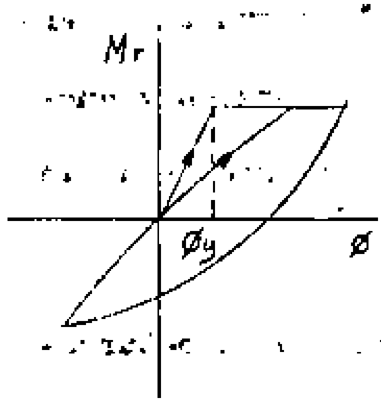
Fig. 2 Idealización de curvas típicas de disipación de energía.

Un estudio reciente realizado por Blakeley (4) sobre la respuesta dinámica no lineal de sistemas de concreto presforzado - concluyó que el desplazamiento máximo obtenido es del orden de - 40% mayor que el de un sistema de concreto reforzado con misma - resistencia, rigidez inicial y mismo porcentaje de amortiguamiento viscoso.

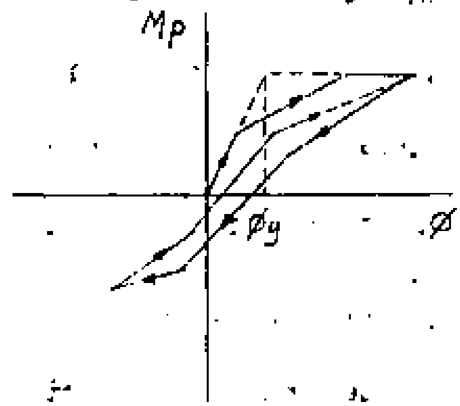
Thompson (5) hizo un estudio comparando las respuestas de - miembros presforzados, parcialmente presforzados y reforzados - bajo diversos movimientos sísmicos, idealizando los diagramas - Momento-Curvatura como lo indica la Fig. 3 y tomando los registros del sismo de El Centro 1940, N-S.

El factor de ductilidad se define como la relación que existe entre el desplazamiento en la falla y el desplazamiento correspondiente a la primera fluencia. Thompson encontró que para pequeños periódos el factor de ductilidad era mayor y que la ten - dencia a disminuir el desplazamiento se debía a un incremento en el acero de presfuerzo.

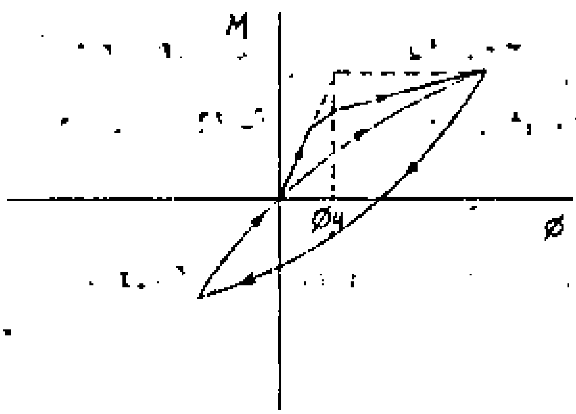
<u>Análisis de miembros presforzados en flexión.</u> Los estudios realizados por Blakeley (4) para determinar las relaciones Momento-Curvatura bajo carga monotónica, demostraron que la curva ob -



a) Concreto reforzado



b) Concreto presforzado



c) Concreto parcialmente presforzado

Fig. 3 Diagramas idealizados de Momento Curvatura tenida para este tipo de carga es colineal con la curva envolvente de cargas cíclicas en miembros de concreto presforzado y que por lo tanto este análisis puede efectuarse para el estudio de la ductilidad bajo cargas sísmicas.

Se realizaron ensayes para obtener diagramas Momento-Curvatura en trabes haciendo variar el valor de la fuerza de presfuerzo, las posiciones del mismo en la sección y la cantidad de refuerzo transversal.

Así mismo, Thompson (5) realizó ensayes en uniones presforzadas viga-columna, reforzando el núcleo de acuerdo con las especificaciones de cortante del ACI 318-71. Las columnas se diseñaron de tal forma que tuvieran una mayor resistencia que las vigas y los ensayes se hicieran con carga cíclica estática simulando la carga sísmica.

Los resultados obtenidos por los estudios mencionados (4) y (5) se resumen a continuación:

a) Porcentaje del acero de presfuerzo.

El efecto de la relación entre el área de acero de presfuerzo y la de concreto, p = As/bh, se muestra en la Fig. 4. La forma de las curvas indican claramente que a un incremento de capacidad de momento corresponde una disminución de ductilidad. El --

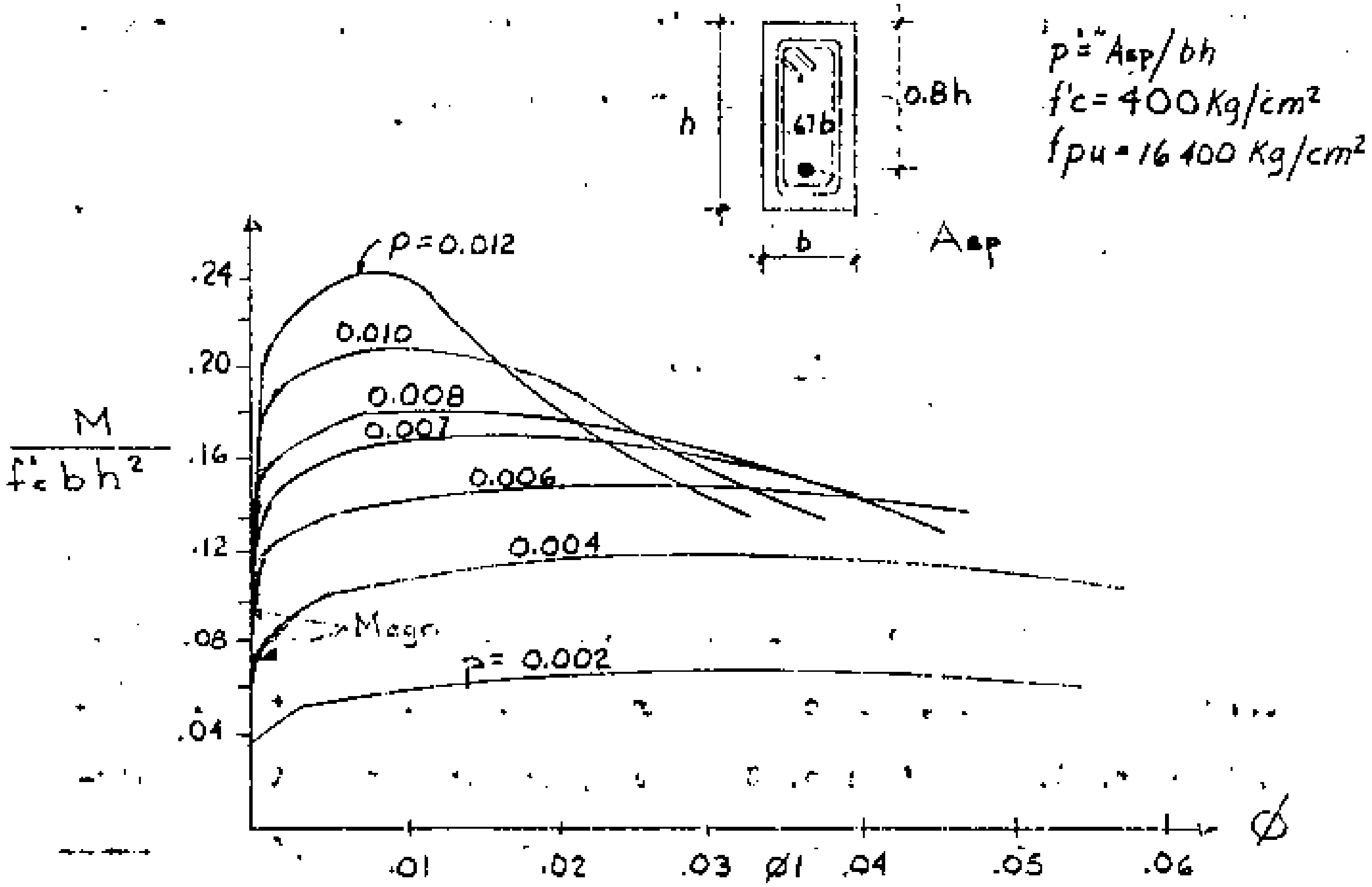


Fig. 4 Relaciones momento-curvatura para una sección con diferentes cantidades de presfuerzo excéntrico

ACI 318-71, especifica que la máxima cantidad de acero de presfuerzo que debe tener una trabe para prevenir una falla frágil es:

$$\frac{f_{sp} As}{bdf'_c} \leq 0.3 \qquad (1)$$

Este límite corresponde a p = 0.0069. El estudio de las curvas de la Fig. 4 indica que para asegurar una ductilidad razonable en diseño sísmico, Blakeley y Thompson recomiendan disminuir la expresión anterior a 0.2, lo cual conduciría a p = 0.0043. La ecuación (1) significa que la máxima fuerza de tensión es $0.2f'_c\,bd$, lo cual implica que en el bloque de esfuerzos en una sección rectangular se tendrá:

$$a = \frac{0{,}2f'c\ bd}{0.85f'_c b} = 0.235\ d$$

y si $d = 0.85h$, la condición queda como:

$$a \leq 0.20h$$

b) Distribución del acero de presfuerzo.

En una sección transversal de una trabe se hizo variar el número y la posición en los cables de presfuerzo, permaneciendo constante la fuerza total de presfuerzo, p = 0.0069 - Asímismo, se observó que si se aumenta el acero de presfuerzo en la zona de compresión, la curvatura no disminuye, debido a que el cable de presfuerzo actúa como acero de compresión en curvaturas grandes. Cuando el acero de presfuerzo se concentra en un solo cable centrado hay una pérdida considerable de capacidad de momento para grandes curvaturas. En cambio solo existe una pequeña diferencia entre dos o más cables. Por tanto, se recomienda que el acero de presfuerzo se distribuya en dos o más posiciones por efecto de ductilidad.

c) Efecto del refuerzo transversal.

En los ensayes realizados, la cantidad de refuerzo transversal tuvo poco efecto en la ductilidad de trabes, ya que triplicando el número de estribos normalmente especificado se logró un incremento relativamente pequeño en la capacidad de momento.

d) Ductilidad en columnas de concreto presforzado.

En los ensayes de columnas bajo cargas cíclicas, las curvas experimentales se trazaron para una articulación plástica directamente sobre la trabe, provocándose así el mecanismo en un marco de un nivel. El diagrama Momento-Curvatura en la columna de concreto presforzado se reduce con un nivel de carga axial y se requiere un refuerzo transversal especial cuando la carga alcanza valores de 0.1 Po, siedo Po la resistencia de la columna con carga axial concéntrica únicamente. Hay poco conocimiento del comportamiento de acero presforzado de miembros a compresión, sin embargo de los estudios realizados se pudo concluir que en las curvas de Momento-Curvaturas, la correspondiente a p/f´cbd = 0.12, corresponde a la máxima curvatura obtenida en los ensayes.

Amortiguamiento de miembros de concreto presforzado. En la referencia (2), se menciona la relativamente baja capacidad de amortiguamiento en estructuras presforzadas.

Depeyroux (3) hace notar que el amortiguamiento del concreto presforzado es comparable al de las estructuras metálicas, es decir del orden del 3% del crítico. En cambio en concreto reforzado es del orden 10% del crítico. Nakano (6) encontró valores mayores del 7% del crítico para estructuras presforzadas.

Esto significaría que deberán tomarse coeficientes sísmicos mayores para estructuras de concreto presforzado, por-ejemplo del orden de 20% mayores que los aplicados al concreto reforzado.

Una investigación reciente de Penzien, (7) sobre el amortiguamiento en trabes de concreto presforzado, mostraron que el presfuerzo y la resistencia del concreto tienen efecto sobre el amortiguamiento solo cuando se aproximaba al momento del colapso.

Sin embargo el efecto desfavorable del concreto presforzado referente a su baja capacidad de amortiguamiento que se traduce en desplazamientos mayores, se contrarrestra en parte por el hecho de que las estructuras de concreto presforzado debido a sus menores escuadrías que en el concreto reforzado, requieren una reducción en la demanda de ductilidad (8).

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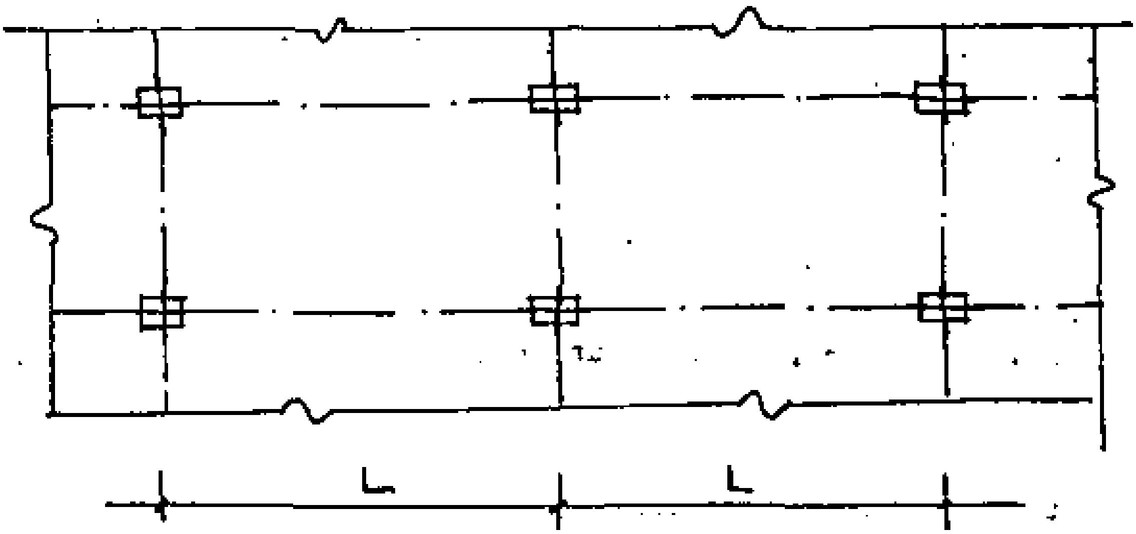
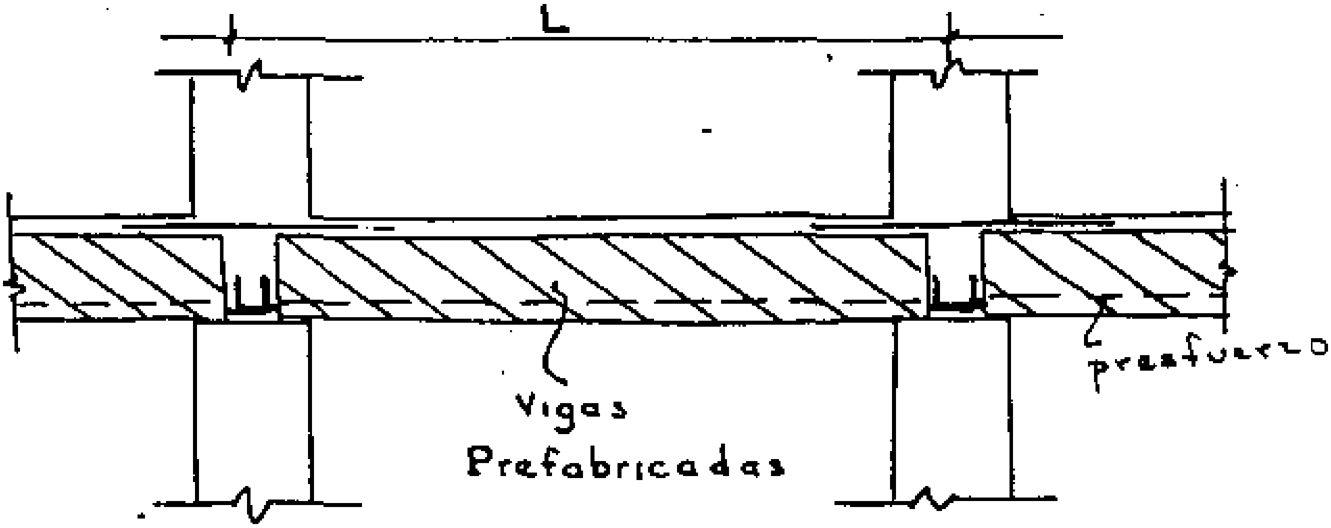
3.- CONEXIONES TIPO DE MIEMBROS PRESFORZADOS.

3-1.- Estructuraciones pretensadas.

3.2 Estructuraciones postensadas.

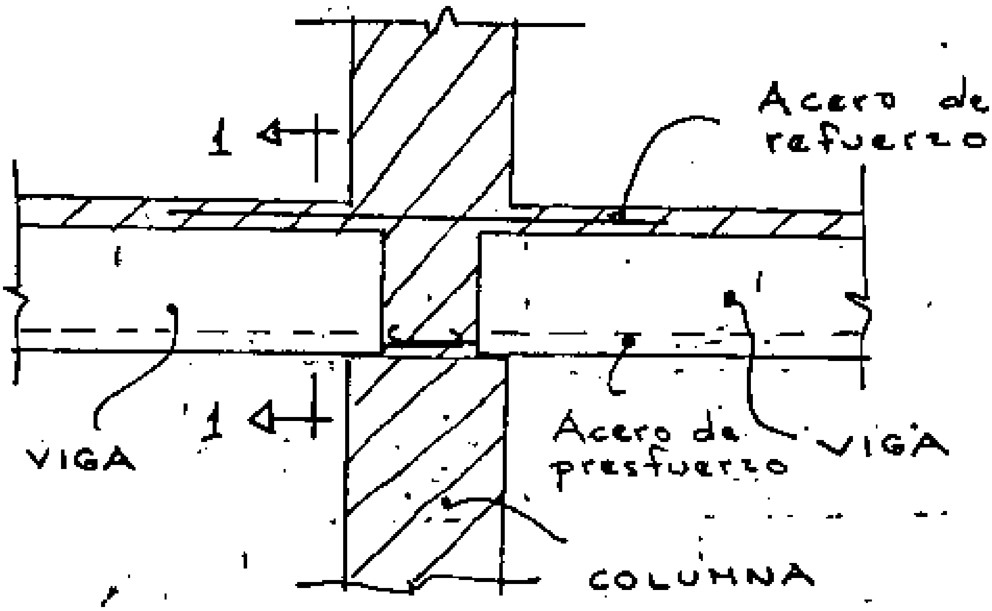


PLANTA

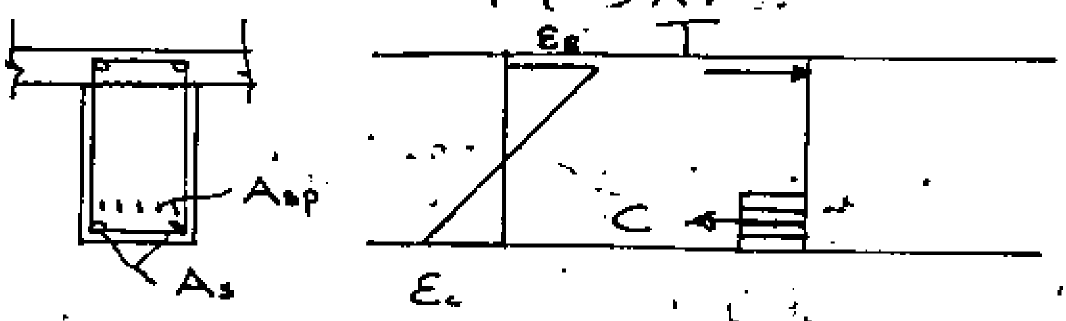


ELEVACION

ESTRUCTURACION PRETENSADA

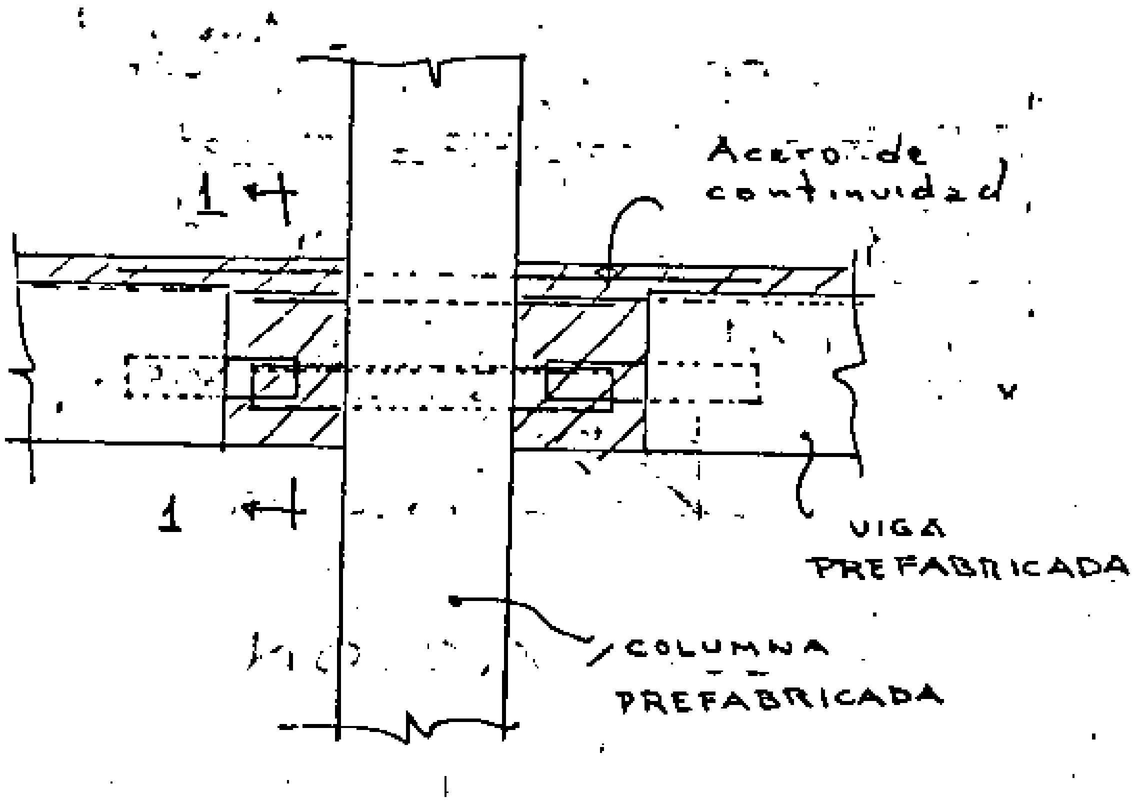


ELEVACION

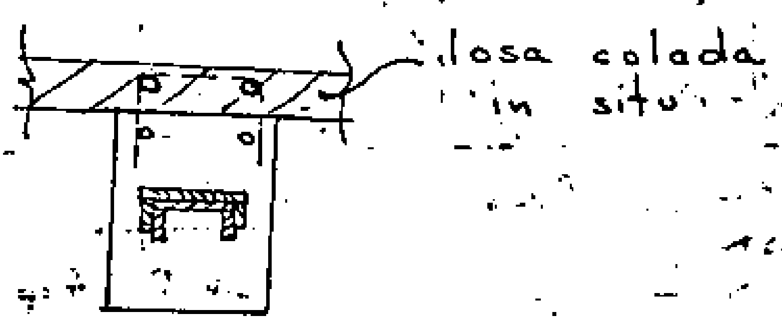


SECCION TRANSVERSAL

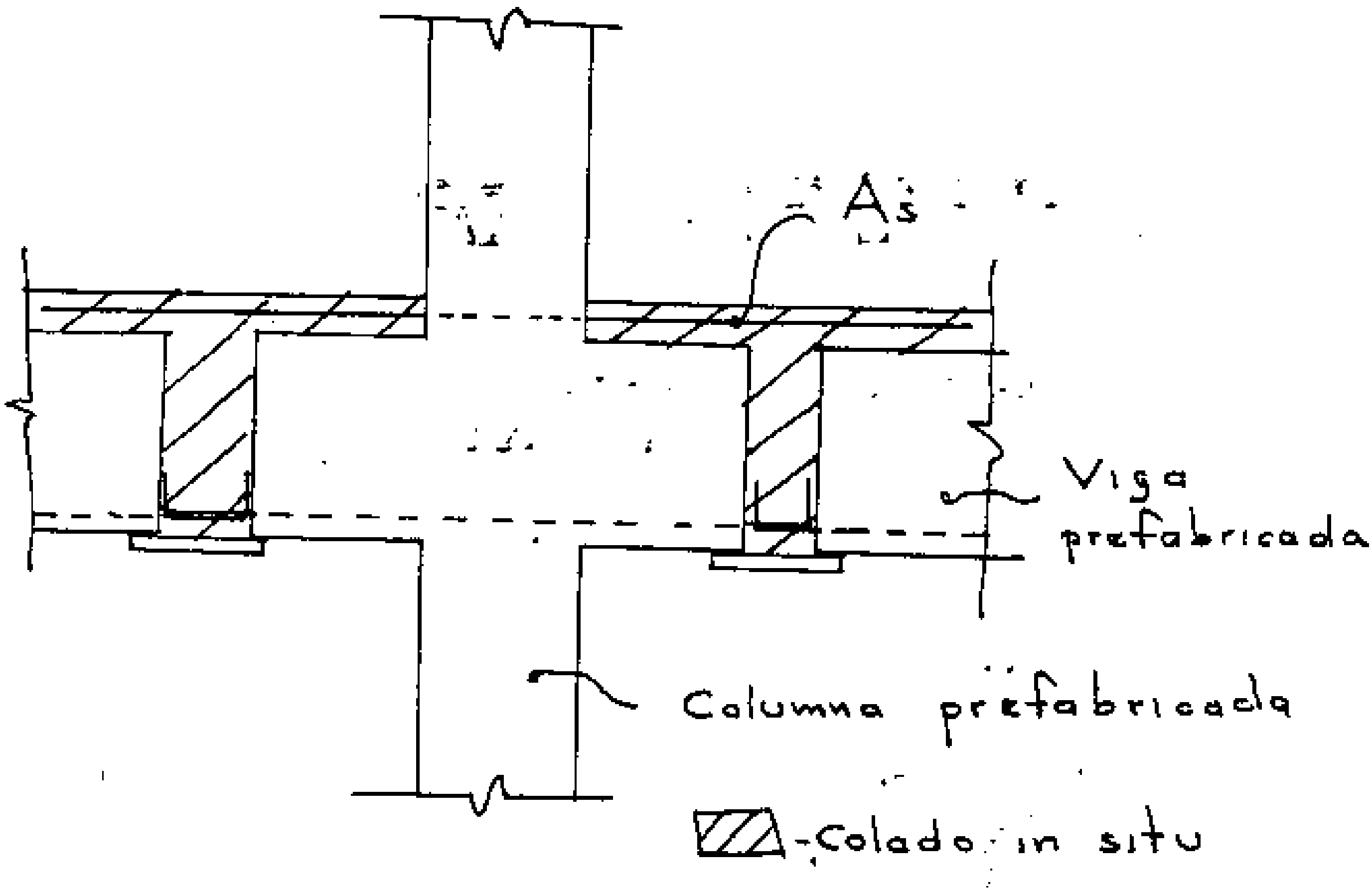
CORTE I-I



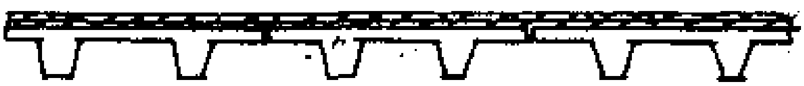
ELEVACION



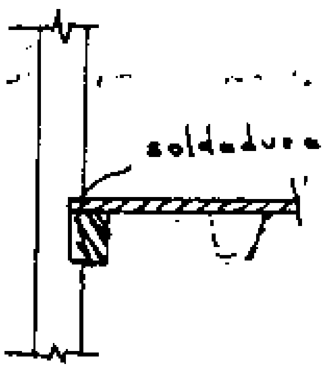
CORTE 1-1



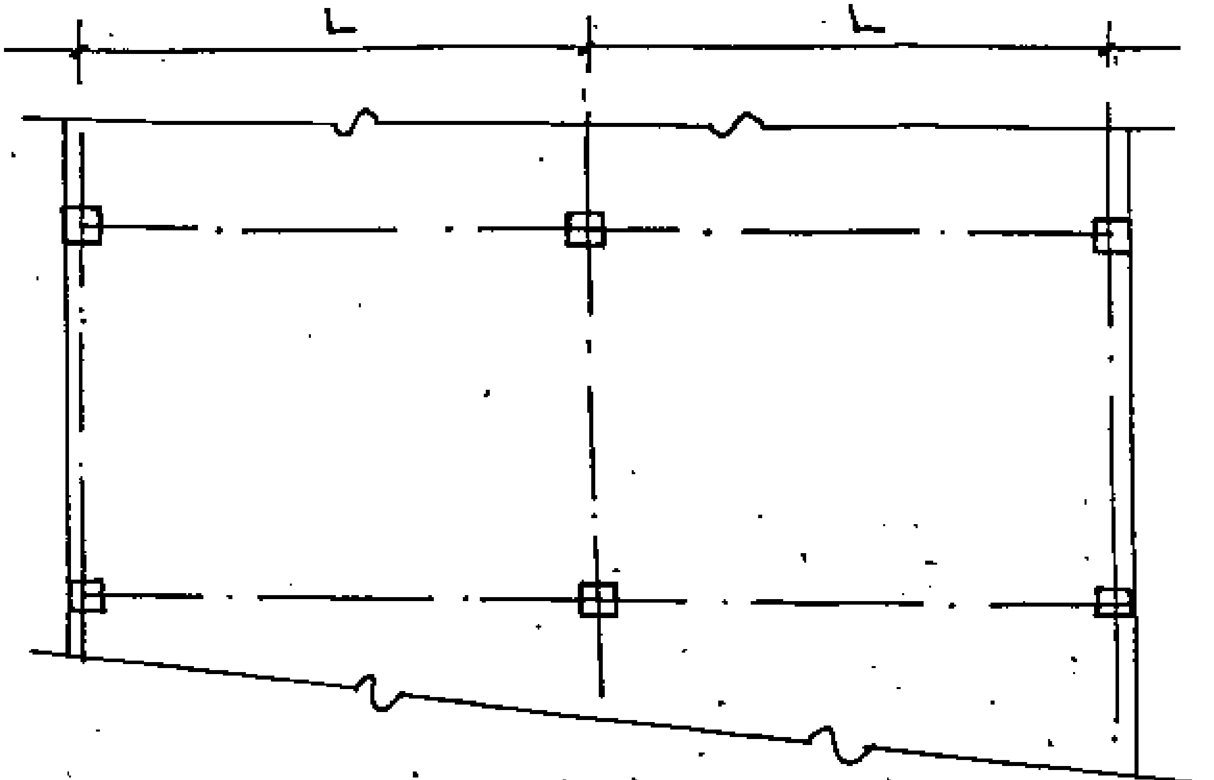
CONEXION PREFABRICADA FUERA DEL NUDO



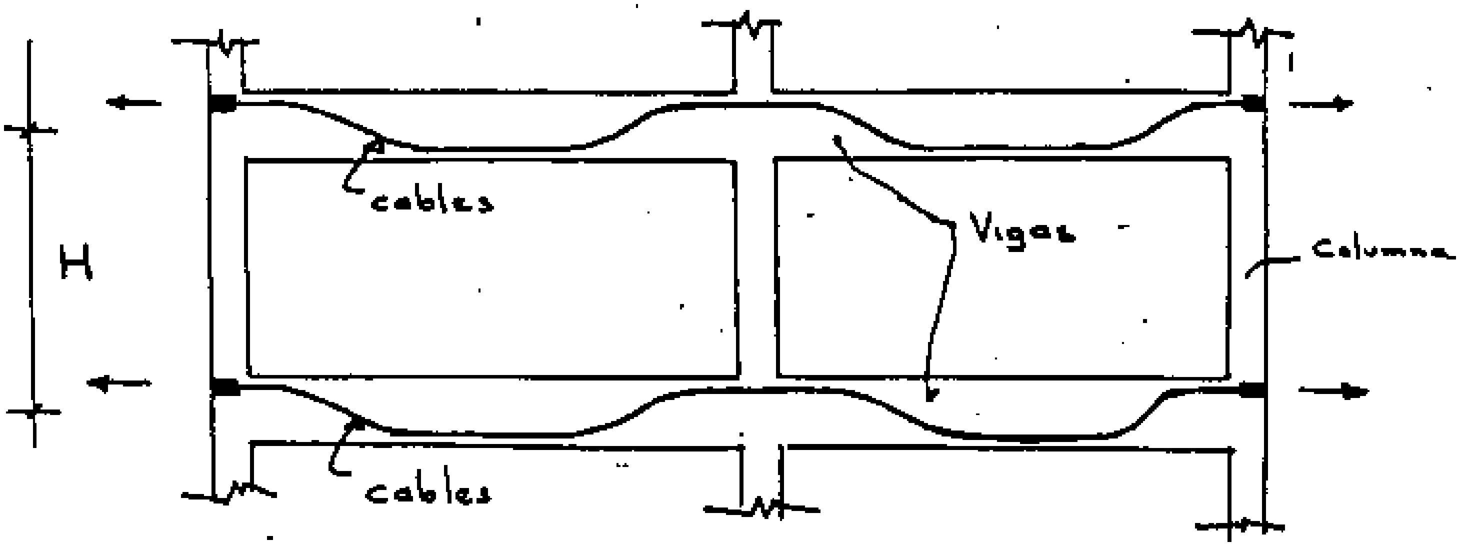
DIAFRAGMAS SOBRE ELEMENTOS PRETENSADOS



UNION DE DIAFRAGMA CON MUROS DE CORTANTE

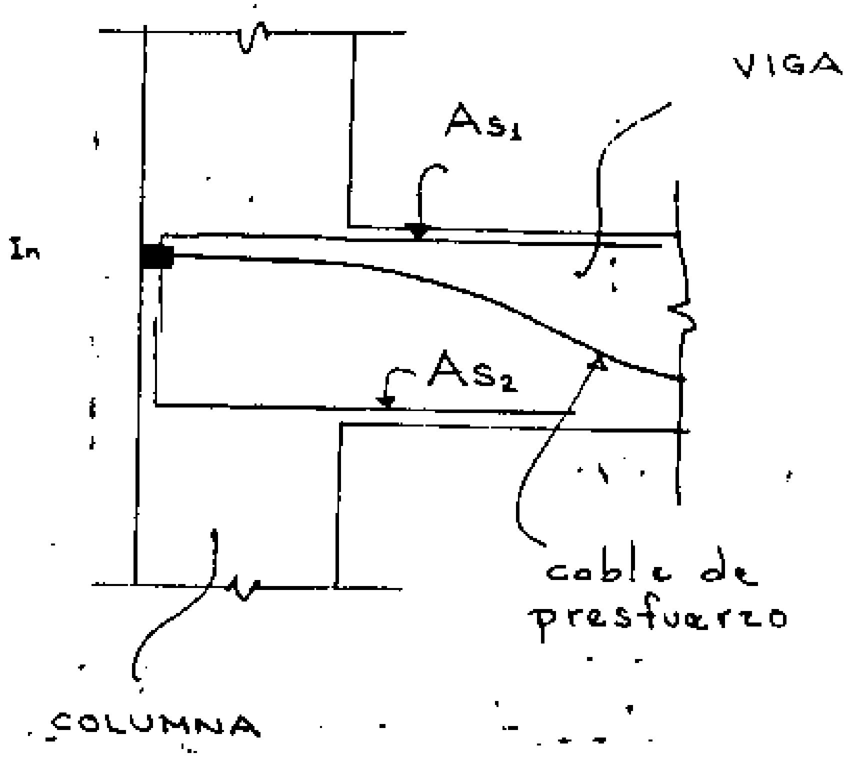


PLANTA

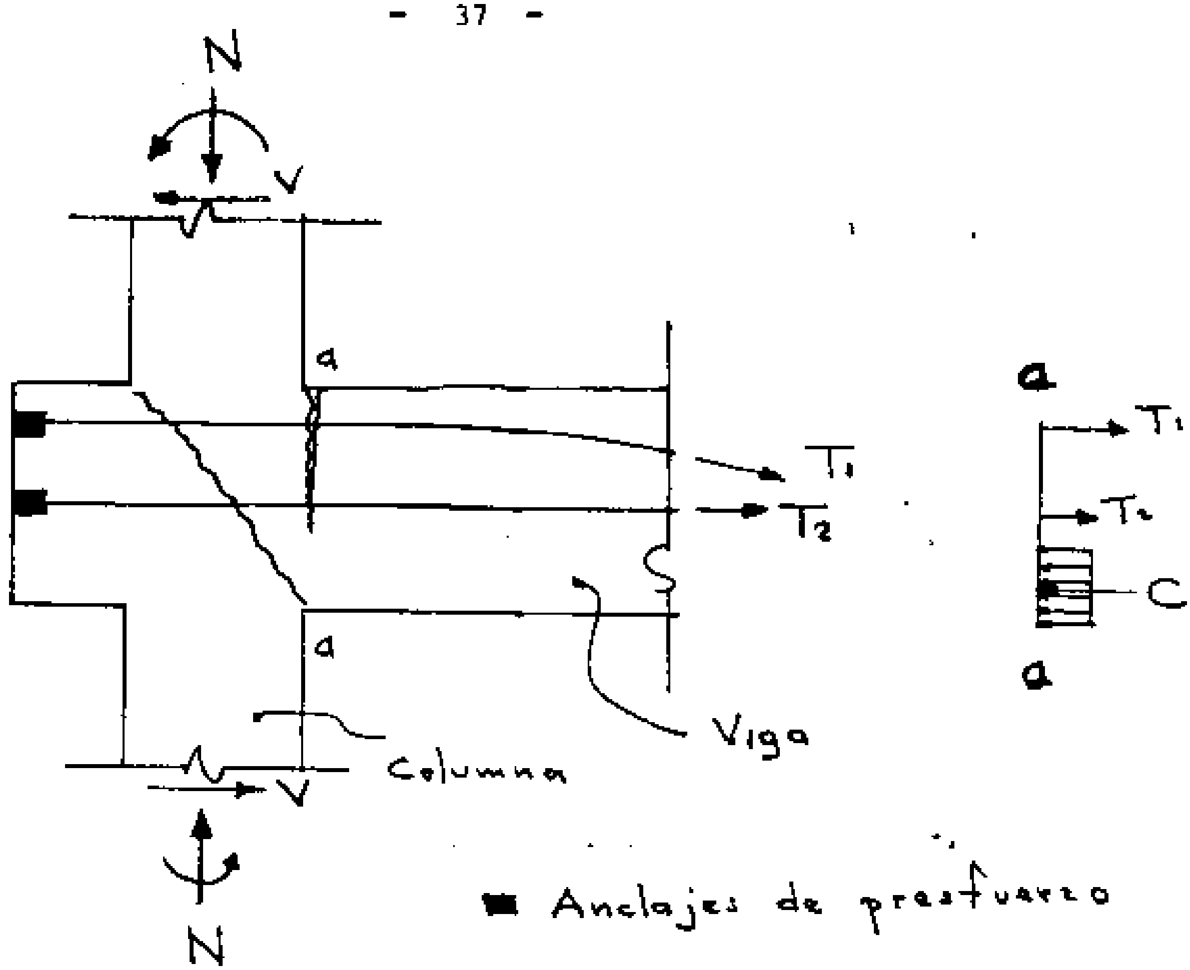


ELEVACION

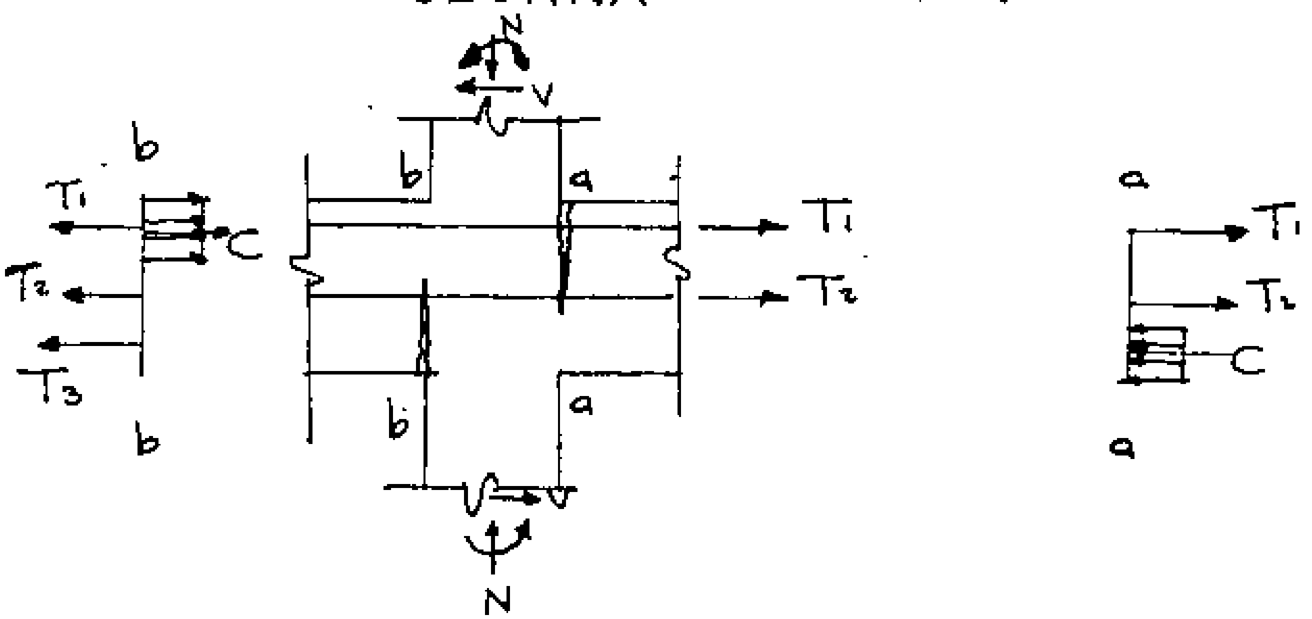
ESTRUCTURACION POSTENSADA



ELEVACION



COLUMNA EXTERIOR



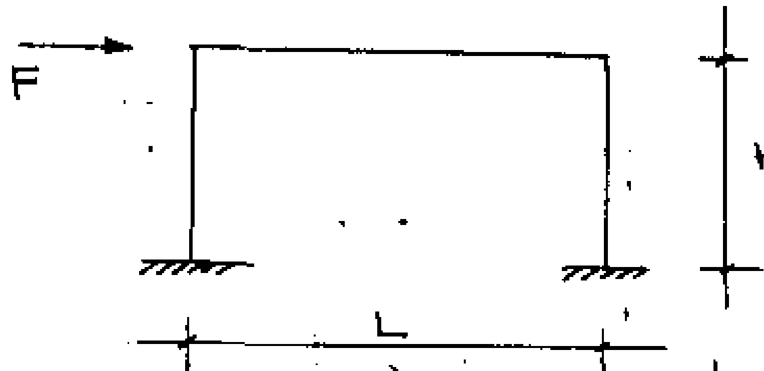
COLUMNA INTERIOR

4.- REGLAMENTOS.

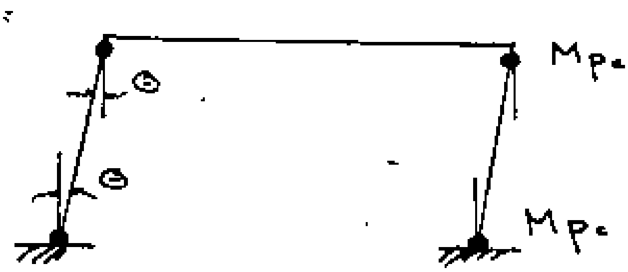
4.1 Reglamento del Distrito Federal.

4.2 Reglamento ACI 318-77.

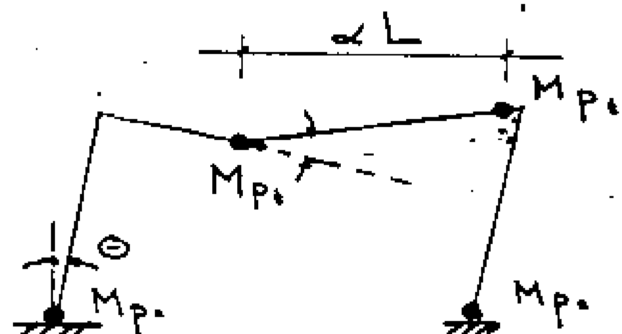
4.3 Recomendaciones C.E.B.- FIP.



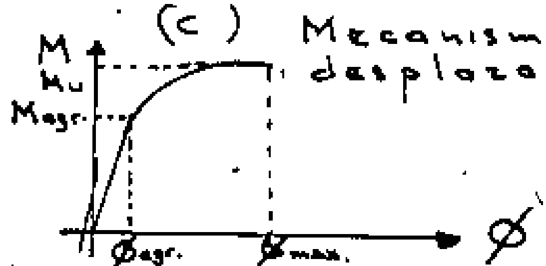
(a) Marco sujeto a fuerza sísmica



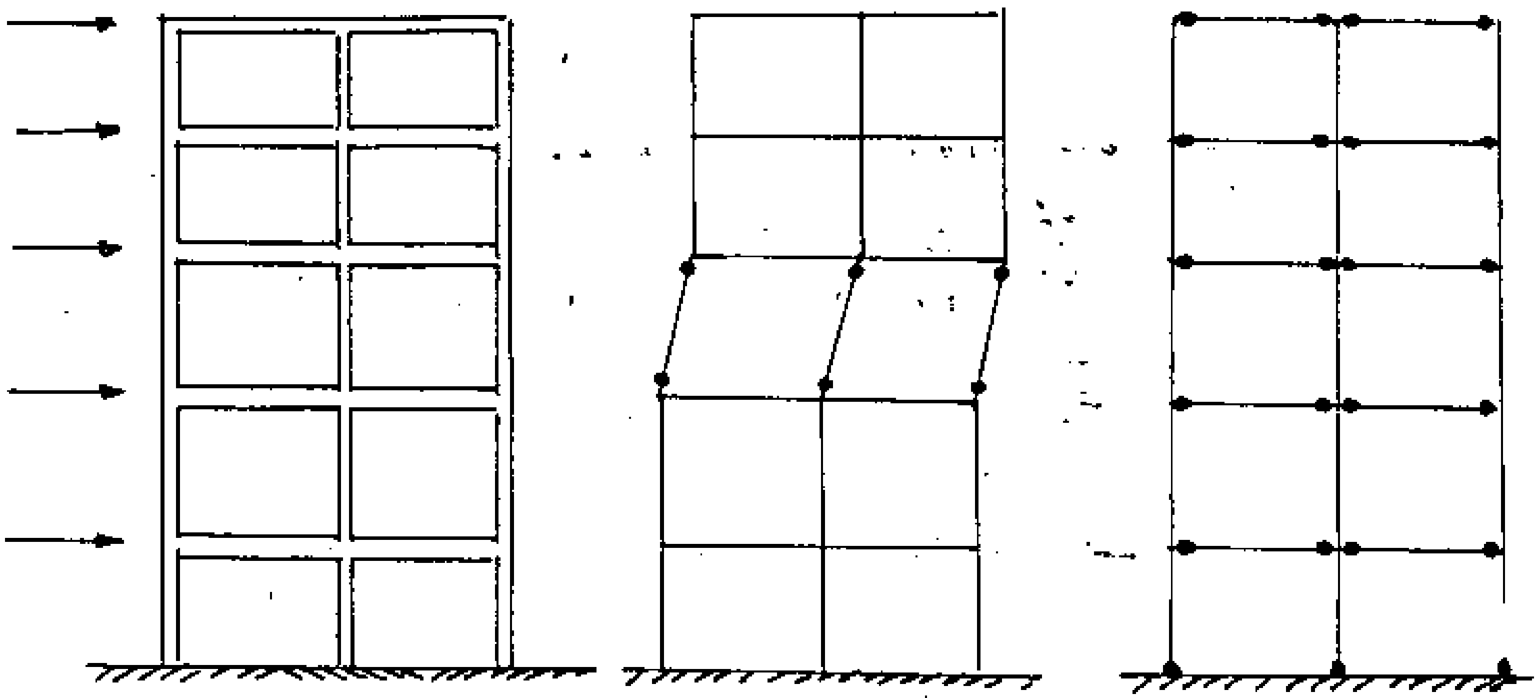
(b) Mecanismo de columnas



(c) Mecanismo en trabes con desplazamiento



(d) Relación M - Ø

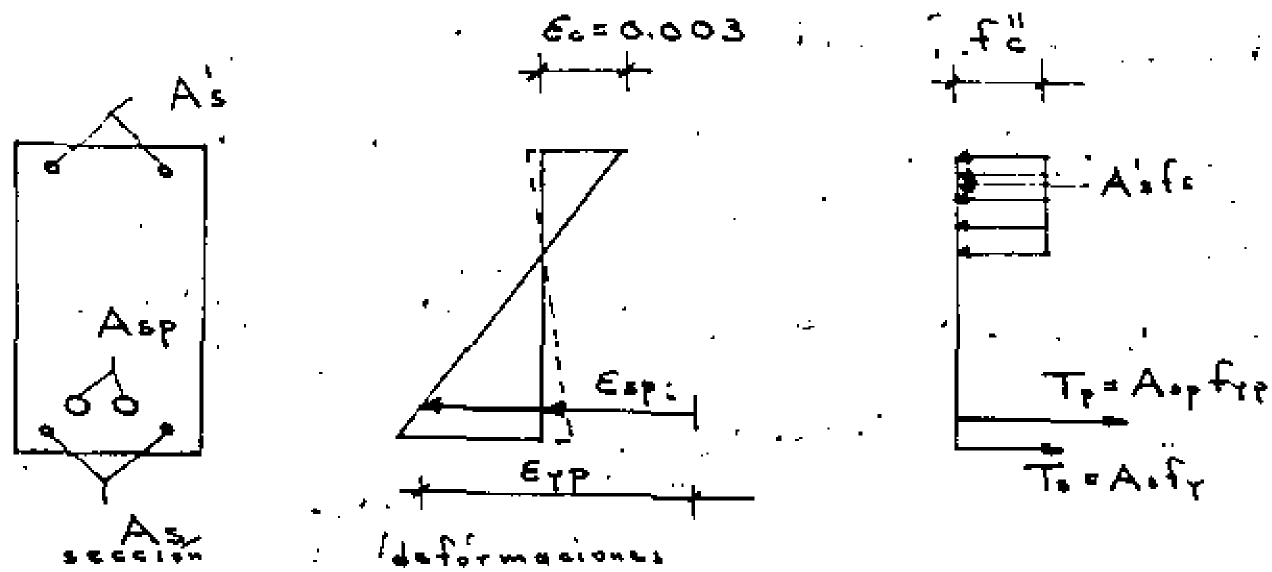


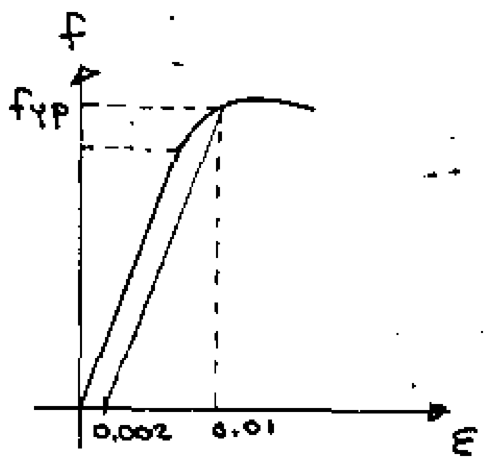
(a) Marco Rígidos

(b) Mecanismo de columnas

(c) Mecanismo de trabes

MECANISMOS DE COLAPSO BAJO FUERZAS SISMICAS





Gráfica f-ε del acero de presfuerzo

Refuerzo máximo en miembros a flexión:

$$T_{máx} = A_s f_s + A_{sp} f_{sp} \le 0.75\ T_{bal}.$$

en la cual:

A_s = área de acero, ordinario en tensión

A_{sp} = " " " de presfuerzo " "

f_s = esfuerzo en el acero ordinario de tensión al alcanzar la resistencia

f_{sp} = esfuerzo en el acero de presfuerzo cuando se alcanza la resistencia.

Falla balanceada en una sección presforzada

REGLAMENTO D.F.

Refuerzo presforzado unicamente:

(1) $q_p = p\,\dfrac{f_{sp}}{f'_c} \leq 0.3$; $p = \dfrac{A_{sp}}{bd}$

Refuerzo presforzado y refuerzo normal:

$q_p + q - q' \leq 0.3$; $q = \dfrac{A_s}{bd}$

$q' = \dfrac{A'_s}{bd}$

A) Porcentajes máximos de refuerzo

$$\%_{máx} = 20\left(1 - \frac{q_p + q - q'}{0.30}\right)$$

B) Redistribución de momentos negativos

REGLAMENTO ACI 318-71

RECOMENDACIONES DE LA FIP PARA EL DISEÑO SISMICO DE ESTRUCTURAS. (LONDRES 1978).

Se presentan a continuación un resumen de las principales recomendaciones.

1) Se considerarán dos estados límite de sismo: moderado y severo. En sismos severos la estructura no debe fallar, debiendo formarse un número significativo de articulaciones plásticas capaces de disipar energía.

2) Son válidos los ánalisis estático o dinámico para determinar las fuerzas sísmicas y las estructuras deberán analizarse en dos direcciones principales.

3) La ductilidad por flexión debe asegurarse mediante la posición de articulaciones plásticas bajo sismos severos. En esas articulaciones el eje neutro debe estar a 0.25h en puntos donde ocurran inversión de momentos y el momento último deberá ser como mínimo 1.3 el momento de ruptura.

4) En las articulaciones plásticas, todo el cortante deberá ser tomado con estribos.

5) De preferencia los cables deberán lechadearse.

6) Los anclajes de presfuerzo deberán colocarse en zonas alejadas a las de máximos esfuerzos, como lo son las articulaciones plásticas.

7) Las uniones trabe-columna deberán diseñarse en tal forma que aseguren que la falla por cortante no ocurre en el núcleo de la unión.

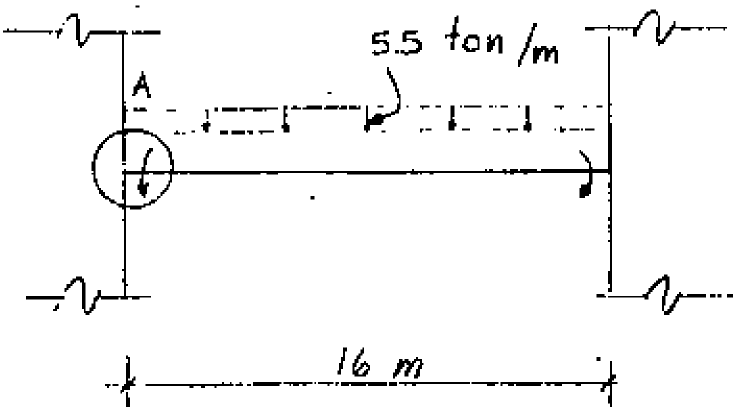
Una consideración importante en las Recomendaciones de Nueva Zelandia para estructuras presforzadas en zonas sísmicas es la de tomar un coeficiente de 20% mayor que las de concreto reforzado. Como un intento que permita incrementar la respuesta en estructuras presforzadas (5).

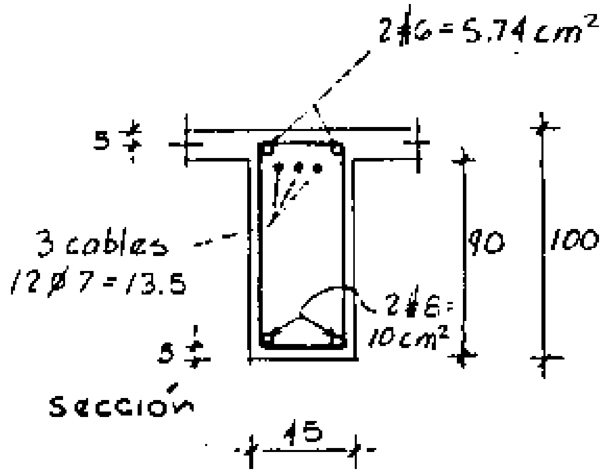
5.- EJEMPLOS.

5.1.- Trabe postensada.

5.2 Trabe pretensada.

EJEMPLO 1.- Verificar si la sección propuesta en concreto presforzado cumple los requisitos del reglamento del D.F. Los elementos mecánicos son los de servicio.





$M_{c.v.} = -75$ tm

$M_{sismo} = -50$ tm

Características de materiales:

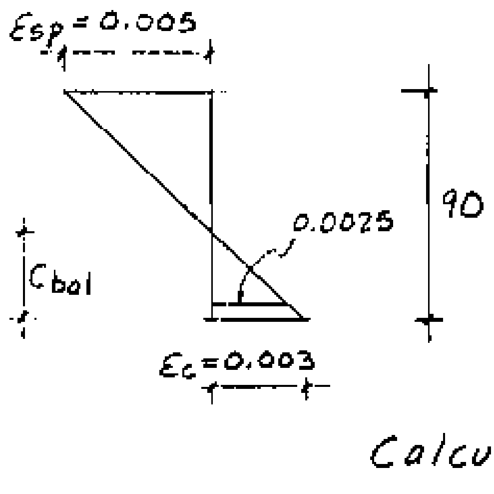
$f'c = 300$ Kg/cm²

$fsp = 13,000$ Kg/cm²

$fy = 4,000$ Kg/cm²

SOLUCION.-

a) Verificación de limitación de acero



$$C_{bal} = \frac{90 \times 0.003}{0.008} = 33.7 \text{ cm}$$

$$\therefore a_{bal} = 0.8 \times 33.7 = 27 \text{ cm}$$

Calculo de resistencias reducidas:

$$f^*_c = 0.8 \times 300 = 240 \text{ Kg/cm}^2$$

$$f''_c = 0.8 \; f^*_c = 192 \text{ Kg/cm}^2$$

La fuerzo de compresión valdrá:

$$C_{bal} = 45 \times 27 \times 192 + 10 \times 4000$$

$$= 233\,280 + 40\,000 = 273\,280 \text{ Kg}$$

$$\therefore T_{bal} = 273.3 \text{ ton}$$

De acuerdo con el reglamento del D.F.

$$T_{máx.} = 0.75\ T_{bal} = 0.75 \times 273.3 = 204 \text{ ton}$$

En la seccion propuesta, suponiendo la fluencia del acero de presfuerzo.

$$T = A_{sp}\ f_{yp} + A_s\ f_y$$

$$= 13.5 \times 13\,000 + 5.74 \times 4\,000$$

$$= 175\,500 + 22\,960 = 198.4 \text{ ton}$$

$$\therefore T \leq T_{máx.} \qquad \text{o.k.}$$

b) Cálculo del momento resistente

Suponiendo la fluencia del acero de presfuerzo:

Por equilibrio de fuerzas:

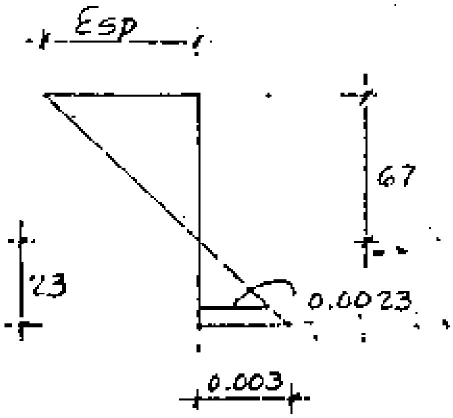
$C = 45 \times 192 \times a + 10 \times 4\,000$

$T = 175.5 + 22.9 = 198.4$

$$a = \frac{158,400}{8\,640} = 18.3 \text{ cm}$$

$$c = \frac{18.3}{0.8} = 23 \text{ cm}$$

Verificando el tipo de falla



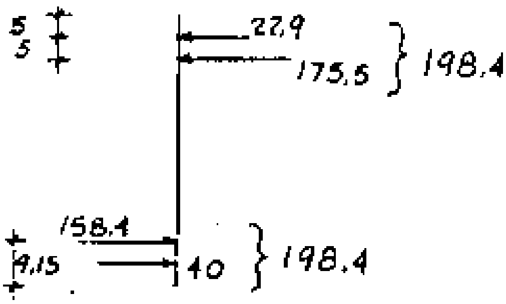
$$\varepsilon_{sp} = \frac{6.7}{23} \times 0.003 = 0.0087$$

$$\varepsilon_{sp}\,\text{total} = 0.005 + 0.0087 = 0.0137$$

$$\varepsilon_{sp} > \varepsilon_y$$

∴ El acero de presfuerzo fluye y la suposición es correcta.

El momento resistente valdrá:



$$z = 100 - 8.31 - 9.42 = 82.2 \text{ cm}$$

$$M\,resist. = \phi\, C z = \phi\, T z$$

$$M\,resist = 0.9 \times 198.4 \times 0.82$$

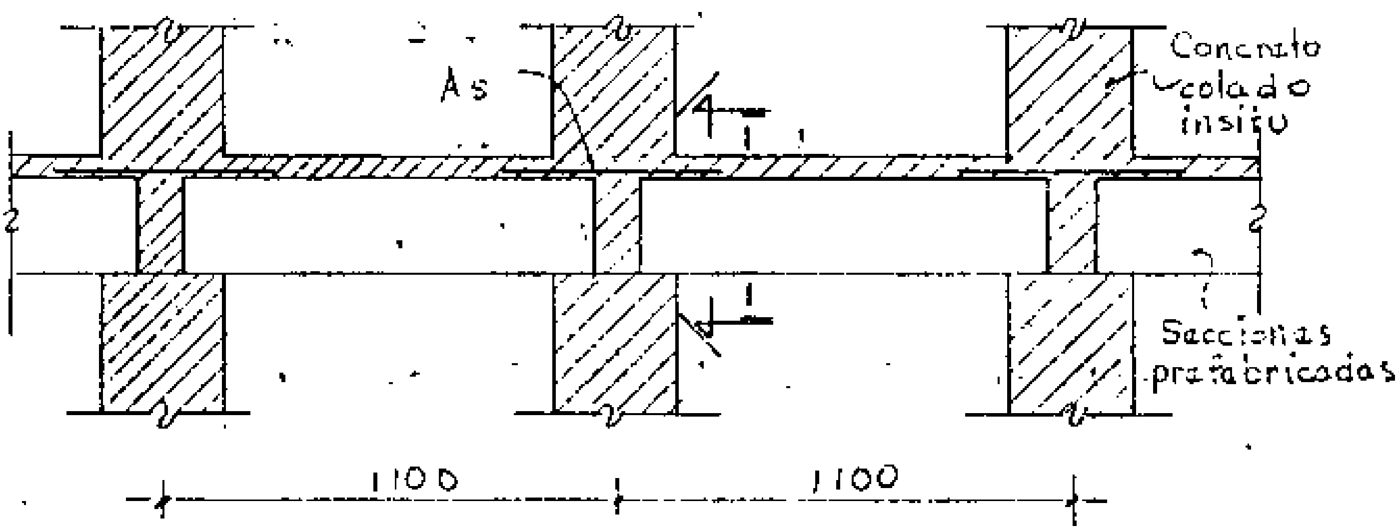
$$= 146.4 \text{ tm}$$

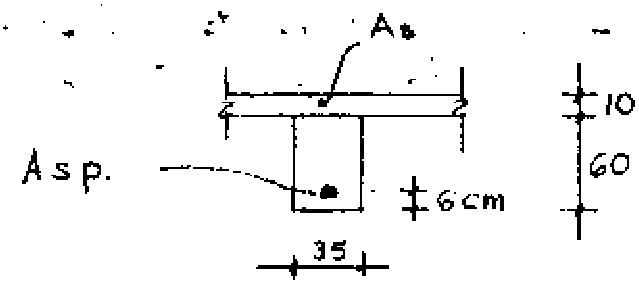
$$M\,actuante = (75 + 50)\, 1.1 = 137 \text{ tm}$$

$$M\,resist > M\,actuante$$

∴ La sección y armado propuestos sí cumplen los requisitos del reglamento del D. F. en flexión.

EJEMPLO 2.- Calcular el área de acero de refuerzo en la viga pretensada de la figura para momento negativo debido a carga viva y sismo.





CORTE 1-1

$M_{c.v.} = -10$ tm

M sismo $= -16$ tm

$f'_c = 350$ Kg/cm²

$f_y = 4000$ Kg/cm²

$f_{sp} = 15,000$ Kg/cm²

$A_{sp} = 6$ torones ½"

SOLUCION

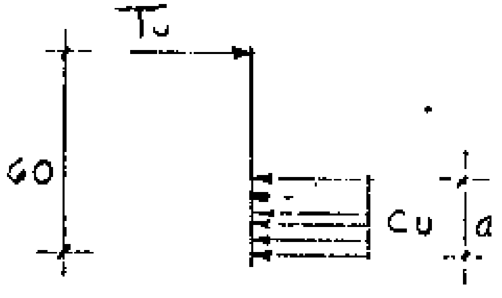
1) Cálculo del área acero para momento negativo.

$$M_u(-) = (10+16)\,1.1 = 28.6 \text{ tm}$$

$$A_s \approx \frac{M_u}{0.9 \times 0.85 d \times f_y} = \frac{28.6 \times 10^5}{0.9 \times 0.85 \times 65 \times 4000} = 14.5 \text{ cm}^2$$

Se pondrán 2#8 + 2#6 ⟶ As 15.7 cm².

Estableciendo el equilibrio en el apoyo:



$\underbrace{f''_c - f_{cp}}_{\text{compresión debida al presfuerzo}}$

$$C_u = T_u$$

$$C_u = (224 - f_{cp})\,35\,a$$

$$T_u = 15.7 \times 4000$$

$$T_u = 62{,}800 \text{ Kg.}$$

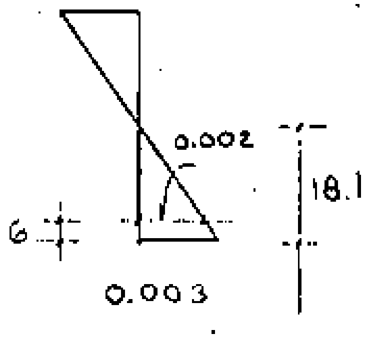
Suponiendo una compresión debida al prefuerzo de 100 Kg/cm².

$$a = \frac{62\,800}{124 \times 35} = 14.5 \text{ cm}$$

$\therefore$ $C = \dfrac{14.5}{0.8} = 18.1$ cm ; verificando la falla se confirma la fluencia del acero As

Si verificamos ahora la compresión supuesta en el concreto :

Del diagrama de deformaciones obtenido:



$\varepsilon_{sp} = 0.002$

La deformación del acero al tensarse se supuso de 0.005

$\therefore$ $\varepsilon_{sp} = 0.005 - 0.002 = 0.003$

$\therefore$ $f_{sp} = \varepsilon_{sp} \times E_s = 0.003 \times 2 \times 10^6$

$= 6{,}000$ Kg / cm²

La fuerza de presfuerzo valdrá:

$$F = 6\,000 \times 0.93 \times 6 = 33\,480 \text{ Kg}$$

y la compresión en el concreto debida al presfuerzo será:

$$f_{cp} = \frac{33\,480}{35 \times 14.1} = 67 \text{ Kg/cm}^2$$

$$67 \neq 100 \text{ Kg/cm}^2 \text{ supuestos}$$

Haciendo un segundo tanteo con el promedio de de los dos: 80 Kg/cm² y repitiendo el proceso anterior, se tendrá:

$$a = \frac{62\,800}{144 \times 35} = 12.5 \text{ cm}$$

$$c = 15.6 \text{ cm}$$

$$\varepsilon_{sp} = 0.0018$$

$$\therefore \varepsilon_{sp} \text{ final} = \underbrace{0.005}_{\text{al tensar}} - 0.0018 = 0.0032$$

$$\varepsilon_{sp} = 0.0032 \times 2 \times 10^6 = 6\,400 \text{ Kg/cm}^2$$

$$F = 6\,400 \times 0.93 \times 6 = 35\,712 \text{ Kg}$$

$$f_{cp} = \frac{35\,712}{35 \times 12.5} = 81 \text{ Kg/cm}^2$$

$$80 \approx 81 \text{ Kg/cm}^2 \qquad \text{O.K.}$$

El momento resistente valdrá:

$$M_{resist} = \phi \, T_u \, z$$

$$= 0.9 \times 62\,800 \left(60 - \frac{12.5}{2} \right)$$

$$= 30.4 \text{ tm} > 28.6 \text{ tm}$$

Nota.- La condición de $T < T_{bal}$ se cumple con amplio margen, ya que el valor de T_{bal} es de 108.86 ton.

DIVISION DE EDUCACION CONTINUA
FACULTAD DE INGENIERIA U.N.A.M.

IX CURSO INTERNACIONAL DE INGENIERIA SISMICA

DISEÑO SISMICO DE ESTRUCTURAS ESPECIALES

SEISMIC RESISTANT DESIGN OF CYLINDRICAL LIQUID STORAGE TANKS

VITELMO VICTORIO BERTERO
Julio, 1983

S.M.I.S. Ingeniería Sísmica, dic. 1981

1

SEISMIC RESISTANT DESIGN OF CYLINDRICAL LIQUID STORAGE TANKS

by Vitelmo Victorio Bertero*

ABSTRACT

This is a paper that summarizes the state of the practice and state of the art in the prediction of seismic behavior of cylindrical liquid storage tanks. It can be divided into five parts. In the first part the seismic performance of these types of tanks during recent earthquakes is briefly reviewed. From this review it becomes evident that a large percentage of these tanks have failed or suffered severe damages. The different types of failure are classified into several categories. The second part of the paper discusses the design of some of the tanks that suffered damages and the state of the practice is summarized by reviewing present seismic code design provisions. Thirdly, the soundness of these code provisions is analyzed in view of the state of the art in the prediction of the seismic behavior of this type of tank. Results obtained in recent theoretical and experimental investigations of such behavior are summarized and implications regarding needed improvement in seismic design are assessed. Results from analyses of an existing tank using different methods are presented and compared. An improved procedure for the practical seismic resistant design of these tanks is outlined in the fourth part of the paper. A series of practical design rules which provide extra margins of safety are offered and the extra cost required is discussed. Finally, recommendations for future research to improve the design and construction of this type of liquid storage tanks are formulated.

INTRODUCTION

GENERAL REMARKS

Surveys of the seismic performance of industrial facilities during the past two decades show that many are potential hazards when subjected to the effects of moderate or severe earthquake ground shakings. Analyses of the performance of liquid storage tanks show that this type of facility is very vulnerable to such effects. A large percentage of these tanks suffered serious damage during recent earthquakes. For example, during the 1964 Alaskan Earthquake in Anchorage, of a total of 21 tanks only 4 suffered no damages, giving a damage ratio of nearly 80 percent. In the 1964 Nigata Earthquake, failure of tanks led to fire in three of the four yards of oil storage tanks and some of these yards burned for several days.

The observed seismic performance of this type of industrial facility demands that more attention be given to its design when located in seismic areas. Tank builders, owners, and regulatory agencies have recognized the need to improve the seismic design, and researchers, particularly in the U.S. and Japan, have been devoting considerable research efforts to

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investigating the theoretical aspects of the problems that have arisen with the observed performance. Significant analytical and experimental investigations have already been conducted and results published. Therefore it has been considered desirable to summarize in one paper the present state of the practice and state of the art in the design of cylindrical liquid storage tanks with the following main objectives.

OBJECTIVES

This paper has the following main objectives:

(1) To summarize observed performance of cylindrical liquid storage tanks during recent moderate and severe earthquake ground motions, pointing out the problems that have arisen from such performance.

(2) To present the state of knowledge regarding the dynamic behavior of such types of structures.

(3) To analyze the reasons for the observed performance in reference to the state of knowledge.

(4) To assess the implications of the observed performance and of the state of knowledge regarding the practical seismic resistant design of such tanks.

(5) To suggest some guidelines to improve present seismic resistant design practice and to formulate recommendations for needed research.

SCOPE

To achieve the above objectives the paper is divided into five main parts. First, the performance of such types of industrial facilities, particularly cylindrical steel tanks, during recent moderate and severe earthquake ground motions is reviewed and illustrated and the different types of failures and/or damages observed are classified into several categories.

The second part discusses the design of existing steel tanks and summarizes the present state of the practice by reviewing present seismic code specifications and the latest proposed seismic design provisions.

The state of knowledge regarding the dynamic behavior of such tanks is reviewed in the third portion of the paper by summarizing recent analytical and experimental investigations on this subject. In light of this knowledge, the reasons for the observed performance are sought.

The fourth part of the paper discusses the need for improving practical seismic resistant design methods for such an industrial facility. This is done by reviewing all the different aspects that should be considered in such a design. After pointing out the uncertainties involved in these different aspects, a comprehensive design procedure is outlined and a series of guidelines and practical rules are suggested to provide greater margins of safety without greatly increasing costs.

Finally, recommendations for research needed to improve the present state of the practice in seismic resistant design and construction of liquid storage tanks are formulated.

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SEISMIC PERFORMANCE OF CYLINDRICAL LIQUID STORAGE TANKS

STATEMENT OF PROBLEM

The type of tank considered in this paper has a flat bottom with either an open top or roof; a cylindrical shell of total height H_s, is filled with liquid to some arbitrary depth, H, as shown in Fig. 1(a).

When the tank built on the ground is subjected to earthquake ground motions, its liquid content will be excited into a sloshing motion and the amplitude of this motion will depend on the interaction between the dynamic characteristics of the ground motion and those of the tank and the liquid. In a very simplistic way, fluid pressures originated by the horizontal acceleration of the ground can be conveniently divided into those associated with the inertia of the liquid accelerating with the ground [impulsive pressures Fig. 1(b)] and those associated with the liquid accelerating with the sloshing motion [convective pressures Fig. 1(c)]. These dynamic pressures act both on the wall and the bottom of the tank. In addition to causing forces and moments on the shell wall of the tank if the tank is unanchored, the wall pressure combines with the bottom pressures to cause a net overturning moment on the tank as a whole.

PERFORMANCE OF TANKS DURING RECENT EARTHQUAKES

There are many recorded failures of cylindrical tanks during earthquakes. During the 1933 Long Beach earthquake a water tank collapsed and this aroused interest in studying the dynamic behavior of tanks. Considerable damage to liquid storage tanks occured during the following recent earthquakes.

March 27, 1964 Alaska Earthquake. Hanson [1] has detailed the performence of liquid storage tanks during this earthquake. In the community of Whittier (Alaska) the release of combustible liquids started a fire which burned for three days. In Anchorage, of 21 tanks, only 4 suffered no damage. Seven tanks containing combustible fluids collapsed, releasing their contents; three tanks released 750,000 gal of aviation fuel; fortunately these did not cause a fire. Although much of the widespread damage to storage tanks was caused by tsunamis, earth settlement and liquefaction of the subsoil resulting from the earthquake, a significant portion of the observed damage resulted from direct structural action of the tank and its content. An important conclusion drawn by Hanson is that while those cylindrical, flat-bottomed tanks that were nearly full sustained severe damage, similar tanks that were partly filled were not damaged. There were many types of damage: 1) total collapse -- the roof of one tank was propelled 75 yards in the direction of the collapse; 2) roof buckling or roof collapse, due to failure of the interior column supporting the roof; 3) failure of the roof-to-shell connection, or 4) failure precipitated by shell buckling occuring around the bottom.

Analysis conducted by Hanson indicated that the tanks were designed without accounting for the actual forces generated by earthquake ground shaking. An assumed ground motion that has 20 percent g maximum acceleration and lightly damped spectral velocity S_v = 2.0 fps was found to be sufficiently intense to cause a typical tank to uplift, accounting for the observed damage. Therefore it was recommended that these tanks be designed and constructed to resist realistic earthquake forces without significant uplift or that provisions be made to contain the content of the tank after its collapse.

1964 Nigata, Japan Earthquake. Several tanks were damaged during this earthquake and their failure released combustible materials that caused extensive uncontrolled fires, illustrated in Fig. 2(a). This fire caused considerable damage to tanks, particularly inducing buckling, Fig. 2(b). Medium or small-scale tanks which were directly set on sandy soil were mainly inclined because of the drop in the bearing capacity of the soil. There was some damage due to buckling of the shell because of vibration. There were also a few tanks which were damaged at the welded joint of the shell wall and roof; some of these failures are illustrated in Ref. 2.

February 9, 1971 San Fernando Earthquake. Notable structural failures occurred to liquid storage facilities during this earthquake [3]. A steel welded tank of 92 ft in diameter, 42 ft high with a capacity of 2 million gal, which contained approximately 1.9 million gal the morning of the earthquake, suffered significant damage. The tank was constructed on a fill which varies from 40 to 53 ft deep under the tank. It was designed in accordance with Los Angeles Building Code using a 20 percent g horizontal earthquake loading. Construction was finished in 1965.

Just prior to the February 9, 1971 earthquake, total settlement since 1965 varied from 3-1/2 to 4-1/2 in. After the earthquake, a settlement check indicated a total settlement varying from 10-1/4 to 13-1/4 in. Thus approximately 7 to 9 in. (2/3 of the total settlement) occurred during the earthquake. Damage to the tank consisted of a horizontal buckling in the shell plate (elephant's foot) 24 ft above the bottom and along approximately a 150° circumference. Also some damage was observed in the redwood roof. The steel sketch plate suffered a permanent deformation leaving a separation of about 2 in. above concrete footing. From analysis of behavior of this tank and two similar ones, located in an area of substantial ground movement and severe structural damage, which did not suffer any serious damage, it was concluded that the buckling of the tank shell was due to overload stresses caused by foundation failure and differential settlement. The two undamaged similar tanks are founded on a non-yielding bedrock formation.

A steel washwater tank located at the Joseph Jensen Filtration plant in the Sylmar area, having a diameter of 100 ft and a height of 36-1/2 ft, suffered some damage in the tank skin and the foundation. The tank is supported on a reinforced concrete ring-wall foundation 14 in. thick by 3 ft deep and is filled inside with sand. There are twelve 1 in. diameter anchor bolts which connect the tank to the ring-wall footing through 12 saddles welded to the outside tank wall [see Fig. 3(a)]. The anchor bolts were designed as tie-down points during construction, not to hold down the tank after completion. The tank was about half full of water and there was evidence of considerable movement of the tank, causing the water to slosh and additional rocking of the tank. This was followed either by pull out or by tension failure of the anchor bolts; one bolt pulled out about 13 in. [Fig. 3(a)]. The upper part of the shell buckled slightly, as can be seen in Fig. 3(b). The buckled part has a thickness of 1/4 in. (The steel tank walls have four vertical courses, maximum course height was 8 ft and the maximum wall thickness 11/16 at the bottom and the minimum, 1/4 in. at the top.) The momentum of the tank roof and support system following the impact of the tank base upon the ring-wall was the probable cause of buckling. This buckling was most noticeable on the southwest side of the tank where vertical movement was greatest.

Field observations made after the earthquake [3] indicated that there had been considerable vertical tank movement, but negligible horizontal movement.

In the Los Angeles county district No. 21 (Kagel Canyon) there are five reservoirs ranging in size from 15 ft diameter by 18 ft in height, to 27-1/2 ft in diameter by 24 ft in height. These reservoirs are all of welded steel construction with slightly conical-shaped roofs and were built from 1937 to 1958. Earthquake damage to all five tanks consisted of slight displacement near the base, and separation from connecting valves and fittings. Compressive stresses in the tank walls from motion acceleration caused buckling in tank shells near the base. From the field survey it was concluded that of the tanks affected by the earthquake, those where the foundation was damaged had less buckling or bulging of the shell than when the foundation remained rigid.

In the Olive View Hospital there were three reservoirs: one 700,000-gal steel tank, and two of concrete. The steel tank, built in 1968, was 55 ft diameter and 40 ft in height. It had a standard foundation consisting of a 1/4 by 8 in. steel retaining ring 57 ft in diameter and filled with three parts crushed rock and one part sand. Damage to the 700,000-gal steel reservoir consisted of a torn tie rod used for bracing the roof rafters. Most of the rafters buckled sideways, all in the same direction, as if the tank shell rotated counter-clockwise and the center column support in the roof rotated clockwise. Some of the rafters south of the overflow were also bent vertically to such a degree that water puddles were formed on the roof in an area 3 to 6 ft from the roof edge. The shell buckled within the lower 2 ft all around the tank except at the drain outlet connection and between the break in the tank floor and the inlet-outlet connections. The buckling caused a bulge of up to 6 in. outwardly. Near the top of the shell above the middle of the staircase, the shell also showed deformation patterns over an area within 10 ft from the roof edge and about 15 ft wide.

The tank floor showed deformation of three different types. There was another side effect of the shell and floor plate deformation. On the south side, between the stilling well and drain outlet, the bottom edge of the tank was not resting on the foundation, but was raised about 1 to 2 in. from the foundation surface. On the north side, the edge of the tank appears to have been pushed into the foundation to a depth of more than 3 in.

Besides the above-mentioned deformations, the weld between shell and floor plate was torn open for a distance of 10 ft north of the stilling well.

Stresses beyond those for which the reservoir was designed caused buckling of the shell and tearing or pulling apart of the bottom plates in the 700,000-gal reservoir. Draining of water from the reservoir caused breaking of the foundation and erosion of the earth pad. There were many other small steel tanks which suffered similar damage.

Inside the tank, the foundation under an area of roughly 100 sq ft between the drain outlet, interior ladder, and center column seemed to have collapsed. The floor plates in that area did not show any support when walked or jumped on. Of course, this "hole" under the floor plates could have existed before the earthquake.

In addition to steel reservoirs, there were many reinforced concrete tanks, most of which were partially buried. The concrete tanks suffered considerable damages and had to be removed or abandoned.

From the field survey and analysis of performance of the reservoirs conducted [3] after the San Fernando Earthquake, the following main observations and recommendations were made.

To minimize tank failures, the first requirement is an adequate geologic investigation. Second, consideration should be given to constructing tanks, where possible, so that the height of the tank is not in excess of 0.7 nor less than 0.4 of the diameter. If these ratios are exceeded, then extra heavy sketch plates may have to be added to the outer radius portion of the floor.

The behavior of the steel tanks does seem to indicate that the design criteria for these tanks should be reviewed because of wall buckling in almost all steel tanks that were surveyed. The solution to this problem might be more involved than simply limiting height-to-diameter ratio, since this does not directly take into account any vertical accelerations or potential buckling up in the tank shell. Another difficult problem is to prevent the breaking or pulling apart of external tank connections.

<u>November 23, 1977 San Juan, Argentina Earthquake</u>. This earthquake of Richter magnitude 7.4 devastated the town of Caucete near the city of San Juan, and produced severe damage to many civil engineering structures. One of the main, and perhaps most important features, was the extensive damage to wine reservoirs. Although no official figures of the wine lost because of reservoir failure was released, some newspaper evaluations ranged from 10 to 20 million liters (10,567,000 to 21,134,000 quarts). Personnel from the "Peñaflor" wineries claimed their losses at San Martin and San Isidro establishments to be 2 million liters. This loss was not only a disaster to the economy of San Juan, but raised serious questions regarding the design and construction of these reservoirs.

There is a wide variety of reservoirs particularly in the larger wineries. Besides the common outdoor cylindrical tanks, there are rectangular, multicell indoor tanks of one or two stories built of reinforced concrete. No serious damage was observed in this last type of reservoir, except for some cracked roofs.

Most of the tanks were made of steel plate or reinforced concrete, although some were made of wood. These were small and did not suffer damage except minor support problems. Although all the steel reservoirs

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were cylindrical, their capacity, slenderness ratio, and roofs varied considerably -- from 50,000 to 1,500,000 liters. In general the reservoirs were short, i.e., of a slenderness ratio (diameter/height) close to 1. Cooling tank reservoirs were tall with a slenderness ratio higher than 2, some with a diameter of 6 meters and a total height of 15 meters (Fig. 4).

No detailed design procedure of these reservoirs was available, but it appeared that the seismic design was based on a static analysis of the filled tank, using seismic coefficient C = 0.2, and very low allowable stresses. According to this method, the tanks were actually overdesigned in spite of the very thin steel plates used in their construction. Details for tanks of 1 and 1.1 million liters are shown in Figs. 5 and 6.

As shown in Fig. 6(a), anchorage consists of rods 14 mm in diameter, each spaced at 80 cm. These rods were welded to the plate of the tank as specified in this figure. In some cases these rods were made of deformed twisted reinforcing bar which during construction were bent flat against the reinforced concrete base to facilitate the placement (erection) of the tank, as illustrated in Fig. 6(b). This is a poor technique because the twisted, cold elongated steel used as reinforcing bars often broke during the rebent operation, and those that did not break were so weakened that they broke during the earthquake [Fig. 6(c)]. Another serious difficulty with the design and use of anchor rods, even when mild (ductile) rods were used, was that in order to weld these rods to the tank steel plate they needed to be rebent with a kink, because of the way the bottom plate (base) of the tank was welded to the tank wall plate [Fig. 6(b)]. In several cases rods failed at these kinks because of shear off or tensile forces that developed (Fig. 7).

The following different modes of failure and/or damage of tanks were observed during this earthquake:

(1) Damage to roofs due to sloshing and suction. Fig. 8.
(2) Torsional buckling of wall. Fig. 9
(3) Buckling of the lower part of side wall. Fig. 10
(4) Breaking of the welded joint of side wall and bottom plate. Fig. 11
(5) Horizontal sliding of tanks. Fig. 12
(6) Inclination of tank (Peñaflor). Fig. 13
(7) Subsiding of tank.

<u>June 12, 1978 Miyagi Oki Earthquake</u>. During this earthquake of magnitude 7.4, several buildings located on soft ground collapsed and some significant damages were sustained by various lifeline systems. Many tanks had significant damages. For example, the Sendai Refinery of the Tokoku Oil Company had a total of 87 tanks -- many of these tanks subsided, and three of the larger tanks containing topped refined fuel failed, spilling, or flushing out through cracks of the annular plates, approximately 68,100 kl of oil [4,5]. The surrounding dike could accomodate only 35,000 kl. The oil overtopped the dike, innundated much of the refinery area, and spilled over into the port. The facility was not in operation when the earthquake struck, having been shut down for annual maintenance and check up. Thus a serious hazard from the spilled oil was fortuitously avoided.

A large water storage tank near the main refinery complex experienced significant rocking. The anchor bolts stretched, or pulled out, from 1 to 6 in.

8

CONCLUDING REMARKS

From the above summary of performance of cylindrical liquid storage tanks during earthquakes, it can be concluded that these facilities can undergo several types of damages. In the case of steel tanks, Sogabe et al [6] have classified damages into the following modes.

(1) Swinging and rotation of floating roof
(2) Failure of top angle
(3) Failure of cone roof
(4) Torsional buckling of side wall
(5) Oval deformation of side wall
(6) Buckling of the lower part of side wall
(7) Breaking of the welded joint of side wall and bottom plate
(8) Failure of inner column supporting roof
(9) Horizontal sliding of tank
(10) Inclination of tank
(11) Subsiding of tank

While the modes 1 - 9 are thought to be the damages caused directly or indirectly by the sloshing of liquid, 10 and 11 are caused by the deformation of the foundation of the tank. The causes of these different damages need to be investigated and suggestions made for the design and construction of new tanks and retrofitting of existing ones. A discussion of these issues follows.

SEISMIC RESISTANT DESIGN OF CYLINDRICAL LIQUID STORAGE TANKS: STATE OF THE PRACTICE

INTRODUCTORY REMARKS

Although most existing tanks have been designed according to codes and standards, these codes fail to define properly seismic excitations or to provide guidance on how to estimate the effects of these excitations on liquid storage tanks [7]. The effects of earthquake ground motions on any structure, usually referred to as seismic loadings, are due to the interaction between the dynamic characteristics of the ground motions and those of the structure itself. Ground motions originated by an earthquake have six components, three translational and three rotational. As shown below, code procedure generally considers only the effects of one horizontal component of the actual ground motion.

CODE DESIGN PROCEDURE

The Uniform Building Code, UBC, [8] is most often referenced when seismic design is a consideration. This code assumes that the only significant effects of ground shaking on structures are produced by the horizontal components. This not only neglects the effects of the three rotational components of the earthquake ground motion, but also those of the vertical translational component. Furthermore, the effects of the two horizontal translational components are assumed to act nonconcurrently in the direction of each of the main axes of the structure. According to UBC, every structure (including tanks) should be designed and constructed to resist minimum total lateral seismic forces assumed to act nonconcurrently in the direction of each of the main axes of the structure in accordance with the following formula

$$V = Z\,I\,K\,C\,S\,W \tag{1}$$

where

- V = The total lateral force or shear at the base.
- Z = Numerical coefficient dependent upon the seismic zone.
- I = Occupancy Importance Factor.
- K = Numerical coefficient dependent on type and arrangement of resisting elements.
- C = Numerical coefficient dependent on the period, T, of the structure.
- S = Numerical coefficient for site-structure resonance.
- W = The total dead load (Weight of reactive mass).

Considering the case of Z = 1, i.e., a tank located in the regions of highest seismic risk in the United States and a value of I = 1 as it was considered until 1976, the UBC formula reduces to

$$V = KCSW \leq 0.14KW \tag{2}$$

Until 1976 the value of S did not appear and $V = KCW \leq 0.12$ KW.

While the present value of $C = 1/15\sqrt{T} \leq 0.12$, until 1976 it was $C = 0.05/\sqrt[3]{T} \leq 0.12$. In other words, before 1976 most of the tanks should have been designed for a total base shear of

$$V = \frac{0.05}{\sqrt[3]{T}}\ KW \leq 0.12\ KW \tag{3}$$

In applying this equation to tanks, the designer faced three problems: (1) The estimation of T; (2) The selection of K; and (3) The estimation of W.

Estimation of T. Until 1976 no guideline was given regarding the estimation of T for tanks. In 1976 the following formula

$$T = 2\pi\sqrt{\frac{\sum_{i=1}^{n}(w_i\delta_i^{\,2})}{(g\sum_{i=1}^{n} f_i\delta_i)}} \tag{4}$$

was suggested as appropriate to estimate T. This formula is based on the Rayleigh method of analysis. The period T should be computed using the structural properties and deformational characterization of the tank and considering the mass of the liquid. To avoid its computation a value of C = 0.12 could be selected.

Value of K. Although no specific value of K for steel cylindrical tanks is given in UBC, a value of K = 2.00 can be adopted which will be in accordance with the K recommended by UBC for a structure other than buildings and other than those set forth in Table No. 23-J of 1979 UBC.

Value of W. This should be the weight of the tank itself plus the weight of the liquid stored and, in some cases, the weight of snow that accumulated on the roof.

Based on the above values, the tanks, according to present (1979) seismic code requirements, should be designed for a

$$V \leq 0.12 \times 2 \times W = 0.24\ W \tag{5}$$

For example, most of the tanks that suffered damages during the San Juan Earthquake were designed for V = 0.20 W.

When this total base shear force is compared with those used for buildings, it is three or more times the base shear required for buildings based on ductile moment resistant frame. Thus two questions arise: First, why should tanks be designed for seismic forces considerably higher than considered for buildings; Secondly, does the use of seismic design coefficient 0.24 provide sufficient safety guarantee against tank failure?

In answering the first question, it is necessary to recognize that the equivalent static lateral seismic forces established by the UBC for buildings are forces based on the assumption that the structure is capable of undergoing plastic deformations which will dissipate part of the input energy to the structure from the ground motion. In discussing these code requirements in cases of ductile resistant frame building structures, it is usually pointed out that the assumed seismic force implicitly involved a displacement ductility of four to six. This assumed ductility (energy dissipation capacity) is reflected in the value of K = 0.67 recommended for ductile moment resisting frame.

In cases of steel cylindrical tanks, discussed and illustrated previously, a main reason for tank failure is buckling of the thin shell. As will be shown later, this buckling occurs in the elastic range of the steel; thus no ductile action (dissipation of energy through plastic deformations) can occur in the tank shell. So the value of K has to be larger than for buildings and should reflect the elastic intertia forces that would be developed due to the elastic response of the tank to critical earthquake ground motion.

Regarding the second question, if the suggested minimum seismic design coefficient of 0.24 provides sufficient safety for tanks located in sites of high seismic risk, the observed damages indicate otherwise. What are the reasons for this discrepancy? In judging the soundness of any seismic code, it is not possible to separately consider each of the different provisions. It is necessary to look at the whole design criteria or process, so that it is not possible to judge the soundness of the code specified seismic forces by analyzing them alone. A seismic resistant design is not specified until at least the following factors are defined: the design earthquake (seismic forces), the form of estimating the period, the damping, and the method of design (allowable stress, strength or limit design).

The UBC specifies the seismic forces at a service level, i.e., these specified forces should be used in conjunction with the allowable stress design method. In cases of steel, the allowable stress against buckling under normal service load conditions (gravity loading) usually assumes a safety factor of two. In cases of combined normal service loads and UBC seismic forces, the safety factor is reduced to about 1.5. This coefficient is associated with the computations of stresses based on the assumed linear elastic superposition of the stresses due to axial forces imposed by gravity, W_T, and those due to the bending moment, BM, produced by seismic lateral forces, i.e.,

$$\sigma = \frac{W_T}{2\pi R t} \pm \frac{BM \times R}{I} \leq \sigma_{all} \tag{6}$$

Where W_T is the tank weight above its base, R is the radius of the tank, t is the thickness of the shell, and I is the moment of inertia. Therefore, in analyzing how sound the code seismic design forces are, it is necessary to see how they are used in the whole design procedure, i.e., it is necessary to see the different factors involved in the two sides of the basic design equation

[DEMANDS]	$\leq$	[SUPPLIES]	
IN		IN	
STRENGTH		STRENGTH	(7)
STIFFNESS		STIFFNESS	
DUCTILITY		DUCTILITY	

are estimated and used.

The first question to ask is, are the demands in strength estimated correctly -- this means, are the forces and stresses correctly determined? It will be shown that the elastic seismic forces for severe earthquakes in the zones of highest seismic risk in the United States can be considerably higher than the one specified by UBC. Secondly, the effects of the actual seismic hydrodynamic forces developed are considerably more severe than those obtained following UBC procedure. Thirdly, the use of just Eq. (6) to estimate the stress demands is questionable because of out-of-roundness, construction errors, and effect of hydrodynamic forces. Thus many uncertainties exist in the estimation of the demands and strength demands may be highly underestimated when UBC design procedure is followed.

<u>Strength Supply</u>. Following UBC design procedure, the next question becomes, what is the available or supplied strength? The allowable stresses specified in the code need to be known for the different load combinations to be considered.

Effects of Gravity Forces and Hydrostatic Pressure of Liquid. In this case the design is controlled by hydrostatic pressure thickness requirements for the vessel. The allowable stress, $\sigma_{all} = F_t$ is usually selected on the basis of using a certain safety factor with respect to the specified minimum yield stress for the steel, F_y, that is used. At present the $F_t = 0.6\ F_y$.

Combined, Gravity, and Seismic Forces. When the effect of seismic lateral forces is considered acting simultaneously with the effect of gravity forces, the buckling of thin shells usually controls. In the case of combined loading conditions, the code allows an increase in the allowable stresses of 33-1/3 percent, which reduces the safety factor to about 1.5. The UBC does not provide any specification and/or guidelines on how to estimate the supplied strength against buckling of tank shell. Following code procedure and specifications, the hydrostatic pressure thickness requirements for the vessel usually far overshadow the critical buckling earthquake stress in the vessel because of the increase in allowable stress that is tolerated. However most of tank damages have been due to shell buckling. Thus this factor needs further discussion.

BUCKLING OF CYLINDRICAL TANKS UNDER EARTHQUAKE EXCITATION

One of the most common failure modes of a cylindrical liquid storage tank under earthquake excitation is the buckling of the tank wall. The process of predicting the level of excitation necessary to cause this type of failure is as follows [9]:

(1) Determine the fluid pressure acting on the tank wall due to ground motion and find the critical stresses in the tank due to this fluid pressure.

(2) Establish the tank wall stresses which will cause buckling.

The problem of predicting the critical buckling stresses of a cylindrical tank is complicated by two factors. First, the stress distribution in the tank prior to buckling is fairly complicated and closed form buckling solutions are not available. The second difficulty is that the buckling analysis will predict the buckling condition of the perfect tank structure. The actual buckling stress will be less than this, the difference depending upon the magnitude and shape of the initial geometric imperfections of the tank wall. Thus it is necessary to determine the following ratio.

$$\frac{\sigma \text{ critical actual}}{\sigma \text{ critical computed}} = \text{Knockdown Factor} \qquad (8)$$

This knockdown factor is found from buckling tests for the shell/load combination of interest. However, reliable data for the seismic critical loading condition is very scarce. Then most of the equations (formula) in use or suggested in the pertinent literature are based on results for simple conditions that somehow represent the more complex actual condition. Miller [10], after reviewing all experimental data available from over 700 tests conducted by different investigators prior to 1976, developed equations for critical axial compressive stress for both stiffened, and ring stiffened, fabricated cylinders of finite length. The equation derived for the buckling stress σ_{cr} for elastic failure is of the type

$$\sigma_{cr} = \frac{C\,E\,t}{R} \tag{9}$$

where C = elastic buckling coefficient

E = Young's modulus

t = thickness of shell

R = radius of cylinder

This equation is of similar form to the theoretical formula

$$\sigma_{cr} = \frac{E}{\sqrt{3}\,\sqrt{1-\nu^2}}\,\frac{t}{R} \tag{10}$$

Because tests have indicated that the critical stress actually developed is usually only 50 to 60 percent of this theoretical value, it is better to use formula 9 where it can be shown that C is a coefficient that depends upon a geometry parameter $M = \frac{L}{\sqrt{Rt}}$ where L is the length between end restraints or between ring stiffness, and upon the imperfection of the cylinder. Because the data on fabricated cylinders are very limited, a constant value of C = 0.125 is recommended by Miller for unstiffened cylinders. This agreed with the value, $\sigma_{cr} = 0.12\,\frac{Et}{R}$, suggested by Younger in 1935 [11]. Therefore, a conservative value for estimating the critical stresses for very thin wall shells is

$$\sigma_{cr} = 0.125\,\frac{E\,t}{R} \tag{11}$$

As discussed by Wozniak and Miller [12], considering as E = 29,000 ksi and a safety factor of about 1.5, the following value for allowable stress, F_a, can be obtained

$$F_a = \frac{400{,}000\ t}{D} \tag{12}$$

where t is in inches and D in feet and F_a in psi.

Lo, Crate and Schwartz [13] determine that σ_{cr} increases with internal pressure. Theoretically, with sufficient internal pressure the critical buckling stress will reach the classical limit of $0.6\,\frac{Et}{R}$ which is the theoretical value given by Eq. (10). However, only limited tests have been made to date. The tests conducted by these investigators showed a doubling of the critical buckling stress as the nondimensional parameter $\frac{p}{E}\left(\frac{R}{t}\right)^2$ (where p is the internal pressure) increased from zero to a value of 0.1028. This value is reached when the value of GHD^2/t^2 (where G is the specific gravity of the liquid and H the maximum filling height of the tank) is about 200,000. Based on this it has been proposed [12] that the allowable longitudinal compressive stress for thin wall tanks for values of GHD^2/t^2 greater than 200,000 be

$$F_a = \frac{800,000\ t}{D} \tag{13}$$

and for values of $\frac{GHD^2}{t^2} < 200,000$

$$F_a = 400,000\,\frac{t}{D} + \frac{2\ GHD}{t} \tag{14}$$

placing a limit $F_a \leq 0.5\ F_y$ (where F_y is the minimum specified yield strength of tank shell) to maintain an adequate safety factor throughout the intermediate range of t to D.

The results obtained by Lo, Crate, and Schwartz have been confirmed by Shih and Babcock [9] by testing the same model tanks under true loading conditions -- uniform internal pressure and axial load and inclined tank loading. The uniform loading results illustrated in Fig. 14, show an increase in critical stress from 0.25 to about 0.5 with increasing internal pressure, while the analytical results for a perfect shell do not show this trend and are 0.6. When the values obtained from the inclined tank tests are compared with the predictions of the critical stress based on uniform loading (using the critical stress at the toe of the tank uniformly distributed around the tank and along its length) it was found that the prediction from uniform loading is much lower (20 to 50 percent) than the test results (Fig. 15). Therefore, the results show that the commonly used buckling criterion for cylindrical tanks is somewhat conservative. This no doubt results from the localized nature of buckling in the inclined tank as opposed to the global buckling of uniform loading. A similar situation exists when comparing pure bending and axial compression buckling. Shih and Babcock [9] thought that the differences in these cases were due to the possibility that the nonuniform loading case may not involve the most imperfect part of the tank shell.

Shih and Babcock [9] also noted that for a small angle of inclination the mode of buckling is similar to that of a cylinder under pure bending. For higher angles the buckling mode indicates that the shear stress may be important. This is not considered in present methods of predictions.

To summarize, it appears that the present methods of predicting the critical buckling stress, i.e., the buckling failure criterion used in estimating the strength supplied, is somewhat conservative, however, this does not mean that the present overall design method is conservative. Because of the uncertainties involved in estimating strength demands (computing the state of stress that is developed in the actual situation

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at the critical region of the tank), it may well be that the actual strength demand is so underestimated (because of all the uncertainties involved in such estimation) that it cannot be overcome by the conservatism involved in computing available strength.

STATE OF THE ART IN PREDICTING SEISMIC RESPONSE OF CYLINDRICAL LIQUID STORAGE TANKS

INTRODUCTORY REMARKS

When liquid storage tank foundations are subjected to earthquake ground motions, dynamic fluid pressures are developed which are of great importance in the response of tanks and, therefore, in their seismic resistant design. In 1933 Westergaard [14] offered a solution in the determination of fluid pressure on a vertical dam due to horizontal acceleration. In 1949 Jacobsen [15] solved the corresponding problem for a cylindrical tank. Graham and Rodriques [16] gave a very thorough analysis of the impulsive and convective pressure in a rectangular container. In 1957 Housner [17] studied the dynamic pressure developed on accelerated liquid containers and in 1960 Cooper [18] and in 1963 Abramson [19] presented review articles on the state of the art of liquids in moving containers. Since the 1957 Housner investigation and until 1969, most of the work involving liquids in tanks has had direct application to space vehicle technology. In 1969 Edwards [20] studied the validity of the rigid tank assumption made by Housner and formulated a procedure which incorporates the elastic properties of the cylindrical shell. Newmark and Rosenblueth [21] in a chapter devoted to hydrodynamics, include a discussion of the solution advanced by Housner, to which they introduce some corrections. In 1974 Veletsos [22] presented a simple procedure for evaluating the dynamic forces induced by the lateral component of an earthquake motion in liquid-filled cylindrical tanks of circular cross section, giving due consideration to the effects of tank flexibility. Since 1973 many studies have been conducted on this problem. Epstein in 1976 [23], after reviewing the state of the art and of the practice, suggested a design procedure incorporating the state of knowledge to that year. Some very important work has been conducted by the Japanese, for example, Takayama, 1976 [24], Sogabe, et al., 1976 [6], Fujita and Shiraki, 1977 [25], published results of their studies.

Research has been conducted since 1974 at the University of California [26,27], the University of Massachusetts [28], Rice University [29,30], The California Institute of Technology [31], Southwest Research Institute [32,33], and at institutions around the world, Hunt and Priestly [34], Fisher[35], Kobayashi [36], Rigaudeau [37], and Langer and Prager [38].

In spite of the above studies in current design practice, seismic analysis and seismic resistant design of liquid storage tanks are based on the method developed by Housner in 1957 [17] and recommended by the U. S. Atomic Energy Commission in August 1963 [39], and subsequent modifications introduced by the results of studies conducted by Veletsos [22] and others. In 1977 Miles [40], after summarizing the application of theoretical research to the design of steel reservoirs, made a series of recommendations to attain extra margin of safety in seismic resistant design of these reservoirs. The analysis of demands is based on the model suggested by Housner but with certain modifications to include the flexibility of the reservoir-liquid system; in 1978 Wozniak and Mitchell [12] describe the "basis of seismic design provisions for welded steel oil storage tanks" which are proposed to be included in API Standard 650. This estimation of the demands are also based on the simplified Housner procedure.

Before discussing Housner's method it is convenient to discuss qualitatively the seismic response of these tanks.

QUALITATIVE DISCUSSION OF ACTUAL SEISMIC RESPONSE OF CYLINDRICAL LIQUID STORAGE TANKS

When a tank that contains a liquid at rest is subjected to an earthquake, the subsequent ground motions cause the tank to accelerate in the horizontal plane (Fig. 1) as well as in the vertical direction. The problem is to caluculate the resulting fluid motion and the pressures on the wall of the tank. Before discussing different numerical solutions to this problem, a simple qualitative discussion shall first be presented of the overall response based on the assumption of a rigid tank, considering the effects of only one horizontal component of the ground motion.

Hanson [1] and Sogabe et al. [6] offer discussion of the seismic response of this type of tank. Considering that the effect of the impulsive and convective pressures on the tank are qualitatively similar, Hanson uses the convective pressure in discussing the effect of these pressures on tanks. The distribution of the pressure on the walls and bottom of the tank due to sloshing motion is indicated qualitatively in Fig. 16(a). This makes clear the origin of the bending moment acting on the base of the tank and of the overturning moment acting on the foundation. If the tank were rigid, the distribution of this new pressure distribution in the liquid would be to redistribute pressure in the supporting foundations to satisfy force and moment equilibrium. However in practice, steel tanks are built of thin flexible steel plates, thus the soil pressure cannot contribute resistance to the overturning effect of the fluid moments. For this type of tank, the overturning resistance must be supplied directly to the shell wall from the foundation. This resistance depends on whether the tanks are anchored. If the tank is not bolted to the foundation, the tension force cannot be transmitted to the foundation, therefore the tank wall must rise in this tension zone until a large enough portion of the bottom fluid pressure overcomes the tensile force. This action is illustrated in Fig. 16(b) which shows that the tank could be in equilibrium without uplift if both tension and compression forces could be developed at the bottom of the wall.

Fig. 16(c) shows that when the tension side of the tank rises, a large compressive load is applied to the shell at the opposite side of the tank. The portion of the shell between the compression zone and the tension zone carries the stress as the web of a beam. The large compressive force in the thin shell of the tank can cause the plate to buckle. Because the internal fluid pressure at this point is outward, the natural tendency is for the shell to buckle outward. Even without uplift, the shell may buckle, but it is less likely to do so. The maximum compressive stress is greatly increased when uplift occurs, and prevention of uplift will therefore reduce the possiblity of shell buckling. Probably one of

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the most acceptable methods of prevention is to stiffen the bottom plate around the edge so that the tank wall cannot rise a significant amount without also raising several feet of the bottom plate. This stiffening could be accomplished by increasing the thickness of the bottom plate around the edge or by using either structural shapes or gusset plates. In addition to buckling the wall, the uplift of a tank could also break pipe connections and release the contents, which might then be ignited - another important reason for preventing significant uplift.

So far, only reasons for possible damage to the tank bottom and lower shell have been considered. As previously mentioned, the roof and roof-shell connections are usually also damaged. This damage has two possible causes. If the amplitude of the sloshing motion of the liquid is greater than the freeboard provided, the direct contact of the liquid with the roof may damage the roof-shell connection or buckle the shell at the roof support member-shell connection. Such damage can best be prevented by providing freeboard equal to the maximum expected sloshing amplitude. The other possible source of roof damage is the same as that which causes buckling of the shell at the bottom; that is, the uplift of one side of the tank requires the roof to act as a structural diaphragm to hold the top of the shell circular. This diaphragm action tends to make the roof buckle unless it has been designed as a structural element.

Thus, according to the above seismic response discussion, observed tank damage can be attributed to the combined action of the liquid sloshing motion and the impulsive inertia pressures. The magnitudes of the overturning moment and of the horizontal shear force and the amplitude of the sloshing motion can be determined from the velocity response-spectrum values for the earthquake being considered. The overturning moment caused by pressures on the walls is the significant item which Housner's method of computation simplifies.

HOUSNER'S METHOD

Assuming that the tank contains a liquid that is incompressible, inviscid, and is at rest, and that the tank is rigid, and considering only the effects of one of the horizontal components of the ground motion, Housner [17] showed that the results obtained from a vigorous analysis based on a solution of La Place's equation, allows one to derive satisfactory solutions by an approximate method which avoids partial-differential equations and infinite series. According to Housner's approximate method, part of the contained liquid moves rigidly with the excited tank while another portion of the liquid is considered as mass attached with spring to the walls and is a participant in sloshing.

Dynamic effects due to the rigidly constrained liquid have been termed "impulsive" responses because pressures generated through this mechanism are directly proportional to impulsive accelerations of the rigid container. Effects due to the free sloshing of the liquid are termed "convective" because they are a consequence of liquid flow (Fig. 1). According to Housner, the forces due to lateral acceleration of the liquid in the tank can be found using the equivalent mechanical models of masses—springs--dash pots, shown in Fig. 17.

The impulsive pressures associated with the inertia forces are directly related in time to the ground accelerations and are primarily of high frequency (low period in the range of 0.1 to 0.5 seconds). By contrast, the convective pressures associated with the sloshing motion of the liquid are directly related in time to the oscillatory response of the liquid generated by the earthquake accelerations and are primarily of low-frequency close to natural frequency of the sloshing fluid (high periods). For example: for a water cylindrical tank of 1 ML (Diameter: 10.50 m and Height: 10.55 m), the fundamental period is 3.4 sec and the 2nd order is 2 sec: for a 100 ML cylindrical oil storage tank (Diameter: 80 m, Height: 20 m), the fundamental period is 11.0 sec and the 10th order is 2.3 sec.

The maximum ground acceleration and the maximum liquid sloshing motion probably will not occur at the same time, however, it is possible that for a combination of impulsive and convective pressures at a given time to exceed the maximum impulsive or convective pressures considered separately. As pointed out by Hanson [1], engineering judgment must be used in selecting the maximum dynamic-load condition for which the liquid storage tank is to be designed.

It has to be also noted that the hydrodynamic pressures that are computed are those in excess of the hydrostatic pressure. Formulas for the main parameters in the models of Fig. 17 were originally derived by Housner [17]. In 1976 Epstein [23], based on these formulas and subsequent modifications and corrections, published a comprehensive summary of formulas for cylindrical and rectangular tanks. Epstein presents curves that are useful in quickly estimating the bending and overturning moments and he gives a detailed step-by-step procedure for finding these moments.

As indicated in Fig. 17, for the computation of the bending and overturning moments at the base of the shell, it is necessary to distinguish between shallow and tall tanks. A shallow tank is defined as one in which $\alpha = \frac{h}{R} \leq 1.5$. Tall tanks are $\alpha > 1.5$. For these slender tanks the calculation of the convective forces should be based upon that portion of the liquid considered to be in motion, and not on the full liquid depth. As a consequence of this new treatment, Epstein developed simplified expressions for estimating the moments caused by the convective pressure, without sacrificing accuracy. Although all the equations and curves are developed considering the tank subjected to just one-dimensional horizontal earthquake excitation, the detailed procedure also includes consideration of the effects of the two horizontal components of the base excitation acting simultaneously.

Although Epstein mentions possible effects of the vertical component of ground acceleration, he considers that these do not affect the maximum moment acting on the shell. It has to be noted that the procedure presented by Epstein is just a procedure for estimating the strength demands on the bottom of the shell wall and on the foundation. The suggested procedure does not cover the problem of how to design against these demands, thus it is not by itself a complete design procedure. Epstein did, however, make an important observation. He points out that in most cases of large storage tanks, the full tank condition will give a maximum bending moment, however, if higher accelerations are encountered as the liquid level decreases, lower liquid levels should be investigated. In particular, that liquid height which gives the peak acceleration should be considered. Heights less than this need not be considered, but one or two heights between the maximum acceleration height and the full tank, might also be investigated. The critical liquid level depends upon the nature of the response spectrum curve and where the pseudo acceleration corresponding to the sloshing material period of vibration, T, for the full tank condition falls on the pseudo acceleration curve. As the liquid level lowers, the T increases.

Epstein also gives simplified expressions to estimate the necessary freeboard required to prevent seismic forces from causing buckling and tearing of the roof-to-shell joint, and the resulting damage to the shell itself. This is done by estimating the maximum free-surface displacement, d_{max}. For cylindrical tanks $d_{max} = 0.831 \left(\frac{A}{g}\right)R$ where A is the spectral acceleration and R the radius of the cylindrical tank.

Summarizing, the design procedures in use are either the UBC Seismic Code or the more refined ones based on Housner's analytical model as suggested by Epstein, which is perhaps the more detailed and straightforward one to follow. However, this is more an analysis procedure to estimate the demands on bending moment strength of the wall shell at the bottom than a complete design procedure. A criticism in the application of these methods is that designers may use the formulas and graphs without considering the actual response or the uncertainties involved in estimating demands.

As discussed previously, design requires simultaneous consideration of the two sides of the design equation, i.e., Demands vs. Supplies, and the recognition of the uncertainties involved in estimating these demands and supplies. First to deal with the uncertainties, it is necessary to have a clear understanding of the real response of the liquid storage tank to earthquake ground motions. Again because of these uncertainties, more importance should be given to conceptual design than to numerical design. Designers need to be aware of the real problems involved in the actual response of the whole soil-foundation structure (tank-fluid) system. Some of these problems are briefly discussed below.

EFFECTS OF TANK FLEXIBILITY

Tanks usually are not rigid; they typically have a natural period of vibration in the range of 0.10 to 0.25 sec. Their thin shells, with or without contained liquids, do not display rigid characteristics, but exhibit a multitude of vibration modes for which multiple flexural waves occur in patterns that depend on the basic geometry. For cylindrical shells the flexural waves appear along the longitudinal axis and around the circumference.

Recent analytical work by Veletsos [22, 29] and others [6, 20, 25, 28, 33] have proved the significance of tank flexibility for earthquake response to horizontal ground motions. Experimental work has been conducted by Clough [26] and Niwa [27]. All these studies, along with those for rigid tanks [17, 22, 32], indicate that prediction of seismic slosh responses, which occur at very low frequencies, can be predicted with reasonable accuracy with simple analytical model concepts, such as the one proposed by Housner [17]. However, the more dominant seismic pressure responses are of an inertial nature, so that they occur at higher frequencies, and may be strongly influenced by tank wall flexural responses. Thus the design of flexible containers from a limiting stress point of view still suffers considerable uncertainty. The more detailed and straightforward method that includes the tank flexibility effects is the one proposed by Veletsos.

Veletsos' Approach. In Ref. 22, Veletsos presents a simple procedure for evaluating the so-called impulsive forces induced by the horizontal component of an earthquake motion in liquid-filled cylindrical tanks of circular cross section fixed at the base and having a free liquid surface, giving due consideration to the effect of tank flexibility. The convective forces must be determined separately and combined appropriately with the impulsive forces computed by Veletsos' procedure. As the sloshing of the liquid originating the convective forces is characterized by oscillations of much longer periods than those characterizing the impulsive forces, they cannot be influenced by the tank flexibility. Thus they can be determined by the procedure developed for rigid tanks [17, 23, 39].

Veletsos' procedure is based on the assumption that the tank-fluid system behaves as a single-degree-of-freedom system and the fluid is incompressible. The effective masses for the system, the magnitudes and distribution of the hydrodynamic forces and the base shear and moments induced by these forces, are evaluated for several different assumed modes of vibrations $\psi_{(z)}$ (Fig. 18). The results are summarized in tables in a form convenient for design applications.

The basic step in this procedure is the selection of the mode of vibration $\psi_{(z)}$. For a given tank, the appropriate configuration depends on the relative magnitudes of the flexural and shearing deformations for the structure. These magnitudes depend in turn on the dimensions of the tank and on relative weights of the roof system and of the contained liquid. Veletsos [22] has recommended the following procedure for selecting $\psi_{(z)}$.

(1) Assume a trial configuration $\psi_{(z)}$: for convenience it may be taken equal to one of the functions considered in Fig. 18.

(2) Compute the resulting inertia and hydrodynamic forces, taking for simplicity as the acceleration relative to ground $\ddot{W}(t) = g$. (Veletsos gives graphs that permit direct determination of these forces).

(3) Compute the deflection of the tank due to forces determined in step 2, considering the effects of both flexural and shearing deformations.

(4) The desired $\psi_{(z)}$ is the deflection determined in step 3 normalized with respect to the deflection value computed at $z = H$.

The "fundamental" circular natural frequency of the system, ω, may then be determined using Rayleigh's method.

In Ref. 22 Veletsos summarizes the principal steps of his method. From later studies by Veletsos and Yang [29] using the simple beam-type analysis as well as a more refined one in which the tank is analyzed by use of a shell theory, these researchers concluded:

> For thin tanks with values of H/R in the range bewteen about 0.2 and 1, reasonable upper-bound estimates of the peak values of the impulsive wall pressures and of the associated base shear may be obtained from the corresponding solutions for a rigid tank, merely by replacing the maximum ground acceleration in the expressions for these quantities with the spectral value of the pseudo-acceleration corresponding to the fundamental natural frequency of the tank-fluid system. The natural modes of interest involve a single sine wave in the circumferential direction. The same type of conclusion also is expected to be valid for other

response quantities, but this remains to be verified by the results of additional studies which are currently in progress.

Veletsos and Yang [29] also provided information which facilitates the evaluation of the "fundamental" natural frequency of tank-fluid systems. From their studies they concluded that the fundamental frequency can be estimated with good accuracy using Dunkerley's approximation. Application of Veletsos' procedure to the analysis of practical tanks reveals that the seismic effects in flexible tanks may be significantly greater than those induced in similarly excited rigid tanks.

COMPARISON OF RESULTS USING DIFFERENT METHODS

To analyze the performance of wine reservoirs during the 1977 San Juan Earthquake, the writer has conducted a series of analysis of the different tanks located in San Juan during the Earthquake. The different methods discussed above were used and Table 1 summarizes the main results obtained in the analyses of the 1 Ml tank shown in Fig. 5. The dynamic analyses were carried out using a smooth linear elastic response spectrum corresponding to a peak ground acceleration of 0.33 g. This response spectrum was obtained by modifying the one proposed by Newmark and Rosenblueth [21], according to the guidelines of the Nuclear Regulatory Commission, NRC Guide 1.60, of October 1973.

From the results presented in this table, the importance of the tank flexibility in the estimation of the impulsive forces is evident. The appropriate configuration, $\psi_{(z)}$ for the tank analyzed was the $\psi_{A(z)} = \sin \frac{\pi}{2} \frac{Z}{H}$ which gives an impulsive force which is 90 percent higher than the value obtained using the rigid tank approach. Because the convective forces are very small for this relatively slender tank $(\frac{H}{R} = 2)$, the use of a rigid tank approach will underestimate the base shear in more than 43 percent and the base bending moment (which controls the shell buckling) in more than 49 percent.

Comparing the values obtained using UBC 76 with those obtained using flexible tanks, it can be seen that even when a K = 2 is used, the code values underestimate the seismic forces in about 61 percent, and the bending moment at the base in more than 62 percent. This is because code does not require consideration of the two horizontal components of ground motions acting simultaneously. However, even if the code values are multiplied by $\sqrt{2}$, the resultant values would underestimate the base shear and bending moment at the base in more than 46 percent.

All the above methods are concerned with the analysis of the effects of only the horizontal components of ground motions, neglecting the effects of the vertical translational components and rotational components. Furthermore, analytical methods assume that the tank has a truly circular cross section and fixed base. A discussion follows of the present knowledge regarding possible effects of vertical excitation, movement of tank base, and cross section distortions.

EFFECTS OF THE VERTICAL COMPONENT OF THE GROUND MOTION

Lately some attention has been paid to the effects of the vertical component of ground motions. Kana and Dodge [32] in 1975 investigated the effects of both horizontal and vertical ground motions on the tank response. Marchag in 1978 [41] and Ricaudeau in 1981 [37] conducted studies on the effect of vertical excitations and concluded that the "breathing response" to vertical excitations can significantly increase the hoop tension in the shell and create serious problems because it may lead to compressive stresses corresponding to internal depression with respect to atmospheric pressure, for which buckling limits are usually considered to be very low. Although these last two authors have presented some numerical evaluations of the vertical effects, no reliable analytical method has yet been developed to predict these effects.

EFFECTS OF TANK BASE MOVEMENTS

Anchored (fixed base) tanks require a substantial and expensive foundation to resist any uplift tendency in the shell. The forces associated with the uplift tendency (Fig. 16) are difficult to predict and seismic design of unanchored tanks that might uplift is complex. Clough [26] has estimated the effects that the rocking displacement of the tank has on the axial stress in the shell. He developed some curves that permit comparison of stresses predicted for anchored and unanchored shells of typical proportions, and from this comparison, the relative amplification of shell stress which occurs in unanchored tanks becomes clear.

A recent study by Langer and Prager [38] of an existing water refueling tank of 9.2 m diameter and 22 m high, with wall thickness varying from 2.0 to 1.5 cm, founded on a slab 10.3 m diameter and 1 m thick at depth 0.9 m in a gravel layer, shows that soil structure interaction effects had to be considered because the fixed base fundamental period of the structure was increased by about 30 percent.

Housner and Haroun [31] conducted theoretical and experimental investigations of the dynamic behavior of cylindrical liquid storage tanks seeking to improve the design to resist earthquakes. Their experimental studies concerning vibration tests of full-scale tanks with different types of foundations were to assess the influence of support conditions. Ambient and forced vibration tests were carried out to determine the natural frequencies and mode shapes. From these tests it was found that the foundation flexibility had a noticeable influence on the dynamic characteristics of liquid storage tanks. The observed rocking motions reduced the natural frequencies of the beam-type modes, while the interaction of the out-of-round deformation with the foundation was found to be insignificant. To account for the soil deformability, a simple mathematical model was included in the analysis to represent the foundation soil. Field measurements of the natural frequencies and mode shapes showed good agreement with the computed values. Also of interest is that Housner and Haroun concluded that the roofs have a noticeable influence on the dynamic characteristics of these tanks. Due to their usual high in-plane rigidity, roof systems do not allow significant in plane deformation to occur at their levels as the tank vibrates. This constraint specially affects the mode shapes.

TANK CROSS SECTION DISTORTIONS: OUT-OF-ROUND DEFORMATIONS

As has been discussed above, most of the analytical procedures available for predicting the response of thin walled, cylindrical liquid storage tanks have been developed under the assumption that the cross section of the tank is truly circular. Under this assumption the hydrodynamics pressure exerted by the liquid on the tank wall and the associated radial displacements, when subjected to a horizontal component of ground shaking, are proportional to cos θ (Fig. 18). Based on these predictions, typical design procedures assume that the critical seismic response mechanism is a quasi-static bending moment effect without distortion of the circular cross section. However, the results of experimental investigations conducted recently at Berkeley [26, 27, 42], have revealed that a significant out-of-round deformation response is induced in addition to cantilever beam type response. The measured circumferential distribution of the radial displacements were not proportional to cos θ, but instead were dominated by functions proportional to cos 3θ and cos 4θ. These results raise serious questions as to the correctness of the present design methods. From the Berkeley studies it has been concluded that resonant out-of-round deformations do significantly contribute to seismic axial stresses in anchored tanks. Thus stress predictions, even including the shell flexibility, are not always conservative.

The observed distortions might result from the hydrodynamic pressures interacting with initial geometric eccentricities. It is evident that more refined analysis procedure, which takes into account coupled liquid-shell interaction and section distortion vibrations, must be developed if a rational approach to design of thin-shell tanks is to be achieved.

Recently Veletsos and Turner [30] have studied the effects of initial out-of-roundness of tanks and concluded that the response of high-order circumferential distribution determined in the Berkeley test program must have been due to an initial out-of-roundness with wave length of 1/9 the circumference or less. The measured response is believed to have been due to the reduction of the initial out-of-roundness due to filling of the tank with water and its subsequent partial, localized recovery during shaking.

Housner and Haroun [31], during their vibration tests of full-scale tanks, clearly observed the out-of-roundness or "circumferential" mode shapes. Therefore, it is clear that plate-type modes do respond to base excitations and should be considered. A comparison between measured and computed circumferential mode shapes is given in Fig. 19.

The above discussion presents a summary of the present knowledge regarding the actual behavior of cylindrical liquid storage tanks and of the analytical methods available to predict the demands. As pointed out, although some of the available methods have been presented as design procedure, they are actually analyses procedures rather than a complete design procedure.

Recently a series of seismic design provisions have been prepared which are proposed to be included as an appendix in API Standard 650, Welded Steel Tanks for Oil Storage. A brief discussion of these provisions, which are the most complete to date, follows.

SEISMIC DESIGN PROVISIONS FOR WELDED STEEL OIL STORAGE TANKS

Wozniak and Mitchell in 1978 [12] have discussed the basis of these design provisions, which include the state of the art in this field up to 1978. Detailed requirements are included to assure stability of the tank shell against overturning and to preclude buckling due to longitudinal compression for the highest level of earthquake ground motion expected at the tank site during the life of the tank.

No provisions are included to check the shell against hoop tension because it is claimed that the seismic response in hoop tension does not govern the design of the shell for the maximum level of earthquake ground motion proposed in API Standard 650. However, Wozniak and Mitchell recognize that when tanks are designed for higher levels of earthquake ground motion, increased hoop tension should be investigated, and they offer guidelines for estimating the corresponding increase.

A requirement is included to provide suitable flexibility in piping attached to the shell or bottom of the tank. Additional information is presented for calculating the height of sloshing of the liquid content (to minimize or avoid overflow and/or damage to the roof) and for designing roof support columns to resist forces caused by sloshing. Because the calculation of the fundamental period required by the Veletsos method is somewhat complex for tanks which do not experience uplift, and unknown for those that do, and because ground flexibility is not included, Wozniak and Mitchell have justified the use of the following approach in computing the design overturning moment, M, at the bottom of the shell.

$$M = ZI\ (C_1W_sX_s + C_1W_rH_t + C_1W_1X_1 + C_2W_2X_2) \tag{15}$$

Where Z and I are the seismic zone coefficient and essential facility factor respectively, and have been defined by UBC [8]. C_1 and C_2 are the respective lateral force coefficients for the impulsive and convective forces and W_1 and W_2 are the corresponding weights of the effective masses of the tank content (Fig. 17). W_s and W_R are the total weight of tank shell, and total weight of tank roof plus portion of snow load, if any, respectively. The X_s, X_1 and X_2 are the height from bottom of tank shell to centroids of shell, and of impulsive and convective earthquake forces respectively. H_t is the total height of the tank.

In view of the difficulties pointed out above, these two authors justified the use of a constant value for C_1, which represents the maximum amplified ground motion. The proposed value is $C = 0.24$, which is claimed to be consistent with the UBC maximum value for structures other than buildings ($K = 2.0$), considering that S (see Eq. 1) can be taken as 1 for estimation of the effect of the impulsive forces.

The above authors consider that $C_1 = 0.24$ is a very high value, which is true when compared with the usual value for building based on the UBC approach (Eq. 1), but as shown in Table 1, it is not so when the actual dynamic seismic response is considered. They justify the use of this apparent high value because of the low damping (Wozniak and Mitchell suggest a value of damping coefficient of 2 percent of critical) inherent for storage tanks, the lack of nonstructural load bearing elements, and the lack of ductility of the tank shell in longitudinal compression. The above authors also consider that for some tanks, taking C_1 as the maximum amplified ground motion, i.e., $C_1 = 0.24$, may be overconservative. Thus for very rigid tanks which are anchored, it may be desireable to calculate the fundamental period and use a lower spectral acceleration value. Analysis of results obtained by applying Veletsos' method (see Table 1) shows that the value of $C_1 = 0.24$ proposed for API Standard 650 may be quite unconservative, in certain cases.

For the value of C_2 it is recommended that it be determined as a function of the natural period of the first mode sloshing, T, and the soil conditions at the tank site.

When T is less than 4.5 sec

$$C_2 = \frac{0.30\ S}{T}$$

When T is greater than 4.5 sec

$$C_2 = \frac{1.35\ S}{T^2}$$

Where S, the site amplification factor, varies from 1. to 1.5 depending on the soil profile type--1.0 for rock-like soils and 1.5 for soft to medium-stiff soils. These amplification factors correspond to those recommended by ATC-3 [43].

The lateral forces coefficients, C_1 and C_2, are applicable for the areas of highest seismicity, Z = 1, and for tanks not required to be functional for emergency post earthquake operations, i.e., I = 1. The proposed API Standard 650 has tables permitting selection of the appropriate values for Z and I for seismicity zones and functional requirements respectively, i.e., for oil storage and power generating facilities which are essential for emergency post earthquake operations, I should be taken as 1.5.

The above proposed design procedure is quite straight-forward to apply. It provides equations for determining the maximum longitudinal compression force at the bottom of the tank shell for tanks which experience uplift, as well as the bottom uplift length, recommending that this length be limited to 7 percent of the tank radius.

SUMMARY OF THE STATE OF THE ART

Significant research has been conducted towards solving problems involved in predicting seismic response of cylindrical liquid storage tanks. Although most of this research has been concentrated on isolated aspects rather than on the whole design problem, significant advancement has been made in understanding, at least qualitatively, the seismic response of this structural system. Although there are several aspects of the real response (such as the effects of vertical excitations ("breathing" modes); out-of roundness distortions; state of stress developed in the bottom and shell plates when uplift occurred in unanchored tanks; and the prediction of the shell critical buckling stress under this complex state of stress) for which reliable predictions have not been developed, at least the problems have been identified.

Summarizing, present knowledge does not allow formulation of a straightforward numerical optimal design procedure for the design of this type of tank. Furthermore, recognizing the larger uncertainties that are at present involved in predicting seismic response of even the most simple structural systems and the above mentioned inherent problems of the liquid-tank-foundation system, it is believed that the possibility of developing such optimal numerical design procedure in the near future, is very slim. However, most of the problems involved in the design of such systems have been identified and it is believed worthwhile to present these to the designers through a comprehensive qualitative (conceptual) design process, which will force designers to recognize these problems in design. Such a design process is summarized below.

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COMPREHENSIVE SEISMIC RESISTANT DESIGN PROCEDURE FOR CYLINDRICAL LIQUID STORAGE TANKS

INTRODUCTORY REMARKS

Although several attempts have been made to provide a comprehensive design procedure for the seismic resistant design of this type of tank, perhaps the most complete is the one proposed for welded steel tanks for oil storage, API Standard 650 [12]; certain aspects of this design procedure, however, are still open to question. Most of the studies conducted up to now have emphasized the estimation of the demands; very little attention has been paid to improving the prediction of the supplies.

The following presents a comprehensive design procedure in which emphasis is placed on the concepts (conceptual design) rather than on developing a simplified formula to facilitate the numerical design. The suggested improved procedure follows the methodology used by the writer in the seismic resistant design of buildings. The main aspects of this design procedure are summarized in the flow chart of Fig. 20. In developing this suggested procedure, the author has considered the two sides of the general design Eq. (7), keeping in mind the tremendous uncertainties involved in the numerical prediction of demands and supplies. In this respect the designer should consider the following factors:

(1) No one knows for certain what the intensity and main dynamic characteristics of future earthquakes will be since our technical knowledge is based on too short an historical span of earthquake observation.

(2) Many uncertainties exist regarding the effects of soil-tank interactions and of soil foundation failure.

(3) Modelling techniques and available methods for structural analysis offer satisfactory results for predicting the base shear force and bending moment when considering a tank of truly circular cross sections, fixed at its base and subjected only to the horizontal component of ground motions. Very little, however, is known regarding the effects of the vertical component of ground motion, of the out-of-roundness distortions, and of movements (uplift, rocking and sliding) of its base or foundation; thus no general mathematical or mechanical model to include these effects has been developed.

(4) Besides the problem of how to combine properly the different effects, many uncertainties exist regarding the prediction of the actual state of stress at the most critical region of the tank shell due to local effects at connections with the bottom plate and with the roof, and the effects listed above under item (3).

(5) Many uncertainties exist regarding the prediction of deformation demands, which are very important for the design of connecting pipes.

(6) Many uncertainties exist regarding the prediction of the strength and stiffness supplied to the thin wall shell of the tank, because these are controlled by buckling which is sensitive to the real state of stress and actual deformed geometry.

(7) The detailing and constructional aspects, particularly those concerning: workmanship in welding of the plates; connecting the shell wall to the bottom plate, to the roof, and to the connecting pipes; anchoring the tank; and in constructing the foundation, play an important role in the seismic behavior of these tanks.

(8) Maintenance, particularly regarding the connections and the foundations, also can affect the response.

In view of the multitude of uncertainties involved in the above factors, designers would do well to try to avoid or minimize those problems. and particularly those combinations which could contribute to a failure. Thus designers must attend to the physical behavior of this type of tank during earthquakes, rather than blindly applying empirical formulas. The different aspects of the comprehensive design process, summarized in Fig. 20, are briefly discussed below.

DEFINITION OF STRUCTURAL ENVIRONMENT AND ESTABLISHMENT OF DESIGN EXCITATIONS AND DESIGN CRITERIA

The establishment of design excitations and criteria requires the definition of the structural environment. Besides the permanent forces created by the gravity field and live loads such as snow and wind, of particular concern to seismic resistant design is the local and global geology of regions in which the tank will be located, the site suitability analysis considering all possible types of ground failure, the risk analysis of damage due to direct and indirect earthquake effects [44], including fire potential, particularly in cases of combustible liquid tanks. All these factors may affect the exact location of the storage tank.

Risk and Safety Analysis. Reference 40 discusses in detail the basic information which must be developed to properly consider the risk and safety of a proposed tank installation. The information needed is classified into two groups: first, the recurrence intervals at various levels of ground movements which must be generated by the seismologist; and second, the performance criteria. Miles [40] summarized the minimum performance criteria for water and sewage reservoirs.

Site Selection: Geological Factors and Site Suitability Analysis. Miles [40], after discussing the different geological factors which must be considered during the tank site location process, gives the following general advice.

(1) Seek as much geological information as you can reasonably obtain consistent with the risk involved. Geological information is relatively inexpensive compared with the consequences of locating a reservior at an inappropriate site. Extensive geological information is often required in any case for the environmental impact analysis, permits, public hearings and board meetings.

(2) Discourage the tendency of each member of the design team to apply a safety factor to his work. The project engineer should know where his factor of safety is applied and its magnitude. Normally, safety factors are reserved for the structural design and should not be applied by the geologist or the seismologist.

(3) Start the detailed design only after all the aspects of site location have been carefully evaluated. The risk and safety analysis, and the alternative site geological evaluations offer more opportunities to save time and money than any other stage of the project.

Selection of Design Earthquakes. As in the case of design of buildings for preliminary design; the most convenient way of specifying the design earthquake is by specifying a smooth response spectra which can be considered as the smooth average of all the response spectra corresponding to all available records obtained on sites similar to the site under consideration as far as their local and global geological conditions are concerned. Response spectra usually available for building design might not be relevant for fluid-tank interaction. First the values of damping ratio, ξ, for which the design response spectra for buildings is usually plotted is equal or larger than 2 percent. As will be discussed later, the value of ξ for real liquid tank system vibration is smaller than 2 percent, and for the sloshing of the liquid, is less than 0.5 percent. Furthermore, as pointed out by Sogabe et al., [6] and Clough [26], in studies of the dynamic effect of sloshing (convective pressure), because the first fundamental period of vibration is usually higher than 3 seconds [6], typical strong motion earthquake records may not accurately represent low-frequency, low-amplitude shaking. For the same reason the NRC Guide 1.60 of October 1973 advises that design response spectra be modified for application to the prediction of the sloshing response. The required modification entails artificially maintaining a constant spectral displacement amplitude for periods in excess of 4.0 seconds.

SELECTION OF THE TANK CONFIGURATION AND STRUCTURAL SYSTEM

Since we are concerned with just cylindrical tanks, the main parameters that remain to be selected for the overall configuration are the height of the tank-to-tank ratio, i.e., H_s/R or what are closely related to it, the liquid height-to-tank ratio H/R (Fig. 18), the free board and the type of roof. Regarding the structural system, the only decision that remains is between an anchored or unanchored support system.

Selection of H/R Ratio. The impulsive force in a flexible tank depends on this ratio since the pseudo acceleration of the impulsive mass depends on the fundamental period of vibration of the liquid tank system. This period in turn depends on the H/R ratio. Increasing H/R increases T. For a given H/R ratio and thus a given T, the value of the acceleration for the impulsive mass will depend on the shape of the response spectra. If the resulting acceleration is too high it may be more practical to change the H/R ratio of the tank by changing its relative dimensions rather than to try to live with a high acceleration for the impulsive mass. For the U.S. west coast type of earthquake it is usually necessary to reduce the H/R ratio to 0.5 or less to avoid the highest peak of the pseudo acceleration for the impulsive mass in the response spectra. Clough [26] has estimated the importance of varying H/R from 5 to 0.29, i.e., from very tall slender tanks to very broad ones whose bases are anchored. For all the anchored tanks considered by Clough, the most important of the contribution to the forces and moments are those due to the impulsive mass. Thus proper selection of H/R is of utmost importance because, as is pointed out later, this ratio is a major parameter in determining the tank's earthquake vulnerability to damage. For tanks of small capacity characterized by relatively large H/R, the critical stresses will be generated by global effects--overturning bending moment due to lateral seismic force. In large capacity tanks characterized by low H/R ratios and, therefore, less sensitive to overturning moments, critical stresses are expected to arise through the local response mechanism, which relates circumferential stress in the shell to dynamic pressure changes at the shell liquid surface.

Selection of Freeboard. In large capacity tanks most of the seismic losses have generally comprised roof damage due to wave action, and failure of external piping connections. Roof damage can be avoided or minimized by proper selection of the freeboard. Epstein [23] and Wozniak and Mitchell [12] have developed simple equations to estimate the sloshing wave height.

Selection of Roof. Tank roofs may be flat, conical, or spheroidal, and may be structurally supported or self-supported. Structural supported roofs are usually conical for drainage with the minimum practical slope being 3/4 in. in 1 ft. A greater slope is preferable for sites where a large accumulation of snow is possible. In Ref. 45, guidelines for the design of the different types of roof are given--guidelines for extra safety of supported roofs are offered later.

Selection of Anchored or Unanchored Tanks. In the selection of the most efficient configuration and structural system, it is necessary to consider the way that tanks will be supported. Anchored tanks require a substantial and expensive foundation to resist any uplift tendency in the shell. The forces associated with that uplift tendency are difficult to predict. Furthermore, improperly detailed "tie downs" can cause considerable damage in the shell. Thus designers usually are more inclined to select unanchored tanks in which the shell is supported on a simple ring-wall foundation adequate to prevent differential settlements. Seismic design of such structures is complex because the axial buckling of the shell is accentuated by the diaphragm action of the bottom plate and by the initial bulging created by the large inward radial forces which are applied by the bottom plate and the weight of the contained liquid acting on the part of this plate when the shell lifts off the foundation. Usually these effects are completely neglected in typical tank designs.

Clough [26] has estimated the effects that the rocking displacement of the tank has on the axial stresses in the shell and developed curves that allow comparison of stresses predicted for anchored and unanchored shells of typical proportions. From this comparison it is clear the relative amplification of shell stress which occurs in unanchored tanks. This amplification is much larger in small-capacity tanks, with small radius and large H/R ratio, than it is in large capacity tanks with small H/R ratio. From the studies conducted, it appears that the H/R ratio is a major determinant of a tank's earthquake vulnerability. Typical cylindrical tanks of relatively small capacity are most susceptible to seismic damage. Tanks of large radius, greater capacity, and correspondingly lower height-to-radius ratio will be less affected by lateral earthquake forces. Clough shows that typical anchored tanks with radius in the range of 2.15 m to 10.67 m, and typical unanchored tanks with radius up to about 14.33 m, will buckle under action of the postulated earthquake unless their shells are thickened beyond the value required to resist hydrostatic forces. On the other hand, unanchored tanks of about 14.33 m radius or larger, designed in consideration of hydrostatic stress only, have sufficient inherent buckling resistance to sustain these seismic loads without damage.

PREDICTION OF MECHANICAL BEHAVIOR OF THE TANK

In this step we have to consider: Modelling, Structural and Stress Analysis, and Preliminary Design (i.e., preliminary proportioning and detailing). To conduct any analysis it is necessary to have a preliminary design; consequently the following procedure should be followed after the overall configuration and support system have been selected.

1. First Preliminary Design of the required shell thickness along the height of the tank. This can be done based on the membrane stresses induced by the hydrostatic pressure and considering possibility of needs for increasing these thickness based on the effects of the hydrodynamic pressures that have already been discussed. Standards from the American Water Works Association (AWWA) and the American Petroleum Institute (API) can be used in such preliminary design [45].

2. Modelling of the Tank. This is usually based on the equivalent mechanical models shown in Fig. 17 that have been suggested for the case of rigid tanks [23], but considering the effects of tank flexibility as recommended by Veletsos and Yang [29] (Fig. 18). For final analysis and/or design it is convenient to consider the flexible tank model (Fig. 18) as well as the soil-structure interaction effects. There are cases in which soil-structure interaction effects can considerably affect the fixed base fundamental period of the structure [31, 38].

3. Structural Analyses: Determination of the Dynamic Characteristics of the Fluid-Tank System and of the Internal Forces.

(1) Effective Masses: The effective masses corresponding to the impulsive and the convective forces are computed separately. The effective impulsive masses can be computed using the expressions developed by Veletsos [22] or his table, developed for different values of the ratio within total height of tank and height of liquid and for constant thickness of the tank wall. This is the most consistent mehtod. However, these effective impulse masses can be computed by using the expressions summarized by Epstein [23] or using the graph included in the API Standard 650 [12]. The effective convective mass can be computed using the Epstein table [23] or the graph included in the API Standard 650 [12].

(2) Vibrational Modes and Natural Frequencies of the Liquid-Filled Tank and of the Oscillating Liquid (Sloshing). To find the free vibrational characteristics of the tanks filled with liquid, it is recommended using the method suggested by Veletsos [22] considering both the bending and shear deformations. In the first attempt to avoid the computation of these characteristics, a very conservative approach will be to assume the maximum pseudo acceleration corresponding to the design response spectra for the damping value discussed below. The fundamental sloshing periods can be estimated using formulas given in Refs. 12 or 23. The periods for the higher modes can be determined from expressions given in Ref. 6.

(3) Damping Ratio for Liquid-Filled Tanks. Very little data is available on the appropriate values of damping ratio for liquid-filled tanks. Although Fujita and Shiraki [25] reported that from tests conducted on a 2 m^3 capacity-reduced scale model of a cylindrical tank, the damping during the lateral vibration of the liquid-tank system increased as the water depth was increased, varying from 1 percent to a maximum of 5 percent when the tank was completely filled, it is likely that the damping coefficient corresponding to viscosity effects confined in a boundary layer decrease with tank dimension increases. Thus for real tanks whose capacity is over 100 m^3 or over 1000 m^3, the value could be less than 2 percent. As far as the damping ratio for the sloshing of the liquid, Fujita and Shiraki [25] have measured very small values, less than 0.5 percent. Lev and Jain [46] reported liquid damping in the order of 1/2 percent. Rigaudeau [37] reported that standard values of damping coefficients, which are included in the

building response spectra or determined by model testing, may not be relevant for fluid-structure interaction. He suggested that for real tanks the damping ratio for the lateral vibration of the liquid tank system should be less than 2 percent. For sloshing, the damping ratio can be smaller than 1/2 percent. For water reservoirs, Miles [40] has suggested using 1 percent for anchored tanks and 3 to 5 percent for unanchored tanks. In view of the low value of the damping ratio for the impulsive actions, as well as for the sloshing vibration, several authors have taken a conservative approach, suggesting neglecting damping, i.e., working with response spectra corresponding to zero damping [46].

(4) Participation Factor. The participation factor, C, is determined as the ratio of the effective mass of the system for the rigid body component of motion and the effective mass of the system for the motion specified by $\psi_{(z)}$ (see Fig. 18).

(5) Effective Forces and Moments. From the design response spectrum (or the response spectrum applicable to the particular ground motion under consideration in case of analysis) corresponding to the selected damping ratio, the values of the pseudo accelerations for the impulsive mass, A_o, and the convective masses, A_k, can be determined using the natural periods determined in step 3(2). For a tank with different thickness along its height, it is necessary to estimate the pseudo acceleration at the levels at which the thickness is changed. The pseudo acceleration of a general section located at distance z from the base of the tank will be given by $\psi_{(z)} \times CA_o$, where C is the participation factor and A_o is the pseudo acceleration. The maximum forces induced by the impulsive forces in the structure can be determined either by differentiation of the selected deflection function or by first computing the equivalent static forces corresponding to the computed maximum deflection and by evaluating the desired effects of these equivalent static forces. The determination of these forces can be done using the schemes proposed by Veletsos [22]. The advantage of using these schemes is that then the internal forces at any section of the tank can be computed. If the only interest is to compute the maximum base shear and the maximum bending moment at the base of the shell as well as the maximum overturning moment on the foundation, then these values can be easily obtained by using expressions developed by Veletsos [22] or other authors like Epstein [23] and Wozniak and Mitchell [12]. In using the expression suggested by Epstein, caution should be taken that these expressions have been derived for a rigid tank. Thus in the computation of the contribution of the impulsive forces, the maximum ground acceleration should be replaced with the spectral value of the pseudo accelerations corresponding to the fundamental natural period of the flexible tank-liquid system. In evaluating the maximum pressures, forces, and moments induced by the earthquake ground motions in tanks by using the above methods of equivalent static forces there are two problems whose solutions require designer judgment.

The first problem is how should the contribution of the two different masses, the impulsive and the convective, be combined? Obviously the greatest response is produced when the physical conditions of the real tank is such that these two masses produce their maximum contribution simultaneously. However, usually this will not be the case, particularly in large tanks with low H/R ratios, in which the impulsive and convective forces and moments will not occur simultaneously. Then in general the direct summation of the maximum impulsive and convective forces will be conservative.

The second problem is related to the fact that actual earthquake ground motions consist of three translational and three rotational components. Even if it is assumed that the effects of the rotational as well as those of the vertical translational components are insignificant, there are still two horizontal components that act simultaneously on the tank foundation. The response spectrum are usually prepared for each of the recorded components separately. Thus the only thing that can be done is to compute the maximum response for each of these components. Now the question is, how to combine them? A conservative approach would be to add vectorially, or if the same response spectra is used for the two directions, to multiply the results obtained in one direction by $\sqrt{2}$.

(6) Internal Forces. Once the equivalent external static forces have been determined, the internal forces at any tank section can be computed by statics.

4. Stress Analysis. In this step, stress demands at the critical sections are determined. Usually the shear stresses are small and they do not control the design, whereas either the hoop stresses or the longitudinal compressive stresses do (because they can induce buckling of the thin wall shell). Where the wall thickness is constant throughout the height of the shell (tank) the critical section will be close to the bottom (base) and, therefore, the maximum bending moment at the base computed in step 3(6) can be conservatively combined with the axial stress produced by gravity forces. The critical hoop stresses are obtained by superimposing the hoop stress due to the hydrodynamic pressure to that induced by the hydrostatic pressure. The longitudinal or axial compressive stress and the hoop stress demands should be estimated at all levels where the thickness of the tank wall changes.

In using these estimated stress demands it should be kept in mind that the actual values of these stresses are considerably affected by local variations of pressure and change in geometry (out of roundness) of the shell. Considerable increase and decrease in the above computed hoop stresses can be induced by the "breathing" modes which respond to vertical excitations. These modes, together with the modes that can be excited due to initial out-of-round of the tank, can induce significant changes in the local state of stress in the wall shell. In recent tests conducted by Niwa [27] on slender tanks, it was found that stresses computed on the basis of a rigid tank model grossly underestimate the actual pressures by factors ranging from two to nearly five at various heights.

5. Check of Preliminary Design. This requires determination of the available (or supplied) strength and its comparison with the required demands estimated in step 4.

(1) Hoop Tension. The available or supplied strength per inch of shell height can be obtained by multiplying the thickness of the shell by the allowable tensile stress of the metal used in the fabrication of the tank. The question is, how much should be the allowable tensile stress? The normal allowable design stress, or increased, or the yielding tensile stress or strength? The answer depends on how the demands have been computed. If the hoop stress demands have been determined assuming that the maximum impulsive and convective responses occur simultaneously, and superimposing the maximum values obtained from the two horizontal components of the earthquake, and the effect of vertical excitations has also been con-

sidered, then the supplied strength should be computed on the basis of the yielding, or 90 percent of yielding of the shell material [40]. Wozniack and Mitchell [12] have suggested that the increase in hoop tension due to the hydrodynamic effects (impulsive plus convective) be divided by a ductility factor of two and then added to the hoop tension due to the hydrostatic pressure, and compared with the normal allowable design tensile stresses.

(2) Allowable Buckling Stress. The value of this allowable stress varies depending on the standard available and how the demand on the axial compressive stresses has been determined. If the demands have been obtained in the conservative way described in steps 3 and 4, then the safety factor against buckling that is usually assumed in allowable stress design can be reduced. The method considered in the design provisions proposed to be included in API Standard 650 [12] appears to be the most adequate of all existing provisions. According to these proposed provisions it is necessary to distinguish between unanchored and anchored tanks. Formulas are given to estimate the stress demand as well as the supplied buckling strength for these two cases of supports.

FINAL DETAILING AND CONSTRUCTION ASPECTS

As discussed in Ref. 44, seismic response of any structure depends on its state at the moment that the earthquake occurs. Thus proper numerical analysis and design are necessary but not sufficient. Proper detailing and careful construction and maintenance are of utmost importance. Miles [40] has pointed out that "the good intentions of the design engineer can be negated unless special attention is given to the details." Some guidelines regarding special detailing and construction practice that improve seismic performance of tanks are described below.

Unanchored Tanks. In general designers try to use unanchored tanks because they are more economical. If the shell lifts off the foundation the actual state of stress in the shell, as well as in the tank bottom plate, is very complex to predict accurately. In this case because of the uncertainties involved in the estimation of the demands, it is desirable to:

(1) Increase somewhat the thickness of the bottom plates beyond those obtained from computations.

(2) Increase the thickness of the shell to increase safety against buckling.

However, it is recommended [12] that the thickness of the bottom plate should not exceed the thickness of the bottom shell course to limit the secondary building stress in the shell. Usually it is recommended that the bottom plate thickness not exceed 50 percent of the thickness of the shell.

To provide extra margin of safety, Miles [40] recommends:

(1) Provide butt-welded annular bottom plates under the unanchored shell. Minimum width for the bottom annular ring is given in Ref. 12.

(2) Use tough, weldable steel with the highest possible yielding strength.

(3) Require 100 percent radiograph of all welded joints.

If the tank cannot be proportioned to prevent shell buckling without increasing the bottom plate thickness over the recommended limit, or the required resistance to overturning cannot be provided by the tank shell and internal content without exceeding the maximum value permitted for the uplift length (7 percent of R), then an anchored design must be considered.

Anchored Tank. Shell compressive stresses are easier to analyze and lower than for unanchored tanks of the same size. Because a thinner shell can be used and because of the way that anchor bolts have been attached to the shell, past performance shows a potential for tearing of the shell. Furthermore, in many cases the bolts were pulled out of the foundation. To improve the performance of anchor bolts, Miles [40] recommends using the details shown in Fig. 21. The anchor bolt assemblies should be designed so that the bolts yield before there is an anchorage or shell attachment failure. The use of a square shear plate at the bottom of a relatively short anchor bolt provides better anchorage than hook-type anchor bolts (less relative bond slippage). Anchor bolts may be used as a device to dissipate energy in cases where a larger than expected earthquake occurs. Large, curved washers are recommended for use under the retaining nuts of anchor bolts, i.e., energy can be dissipated when the washers flatten out under the uplift tendency of the tank.

Design of Roof. To avoid or minimize the damage to the roof and upper part of the walls due to sloshing, it is recommended estimating the sloshing wave height and providing a freeboard in the tank somewhat higher than the computed one. In cases where rafters are used, these rafters should be provided with lateral bracing and when roof supporting columns are used, the columns should be designed to resist the forces caused by sloshing of the liquid content. Reference 12 describes a procedure for the analysis of the seismic induced loads on these columns.

Design of Attached Piping. As described previously, many of the observed tank failures are due to failures of the attached pipes. To avoid this type of failure it is necessary to provide suitable flexibility between the tank and attached piping. Although this provision has been mandatory for unanchored tanks subject to bottom uplift [12], it is also highly recommended for anchored tanks. As pointed out by Miles [40], the designer should also question whether the piping itself will move relative to the reservoir due to poor soil conditions. The special provisions for piping connected to the bottom of unanchored tanks are given in Ref. 12. Devices which provide flexibility based on use of sleeve-type couplings or flexible ball points are discussed in Ref. 40.

COST OF SEISMIC PROTECTION FOR TANKS

Miles [40] has analyzed the extra cost due for an increased level of seismic protection, using costs derived from water reservoir projects, all of ratios $H/R < 0.5$. In his comparative cost analyses he plotted the extra cost vs. the design base acceleration and concluded that while there is a definite increase in cost, the increase is reasonable. For example, to offer protection against a base acceleration of 0.30 g, the cost increase is less than 6 percent. For a design base acceleration of 0.50 g, the maximum increase was: 12.5 percent for 11.5 Ml tanks; and less than 8 percent for tanks with capacity smaller than 4.5 Ml. The main extra cost items were for thicker shells to accomodate hydrodynamic pressures, extra freeboard, annular bottom plates, extra nondestructive testing, and flexibility for attached piping. Therefore, by proper selection of H/R ratio, it is possible to economically resist high levels of seismic accelerations.

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SUMMARY, CONCLUSIONS AND RESEARCH RECOMMENDATIONS

SUMMARY AND CONCLUSIONS

Analysis of performance of cylindrical liquid storage steel tanks shows that this type of industrial facility is very vulnerable to the effects of moderate or severe earthquake ground shakings. The observed damages have been classified into several modes, the most common and severe due to: shell buckling, roof buckling or roof collapse, breaking of the welded joint of shell to bottom plate, and the breaking or pulling apart of external tank connections (pipes and anchor bolts). The performance of these tanks indicates that the current design criteria in use should be reviewed. The state of the practice reveals that most of these tanks were designed and constructed according to standard designs which do not take into account the true forces generated by earthquake ground motions. Present UBC seismic design provisions lead to designs that are not safe for moderate or severe earthquake ground motion.

Recent recommended design procedures which have begun to be used in practice, are based on the use of the rigid tank equivalent mechanical (dynamic) model suggested by Housner, in which the hydrodynamic effects are evaluated approximately as the sum of two parts: an impulsive and a convective. A review of the state of the art reveals that while the convective pressures can be accurately predicted by such a model, the impulsive forces are significantly underestimated. To improve predictions of the effects of the impulsive pressures, it is necessary to consider the effects of tank flexibility. Simple procedures accounting for the effects of such flexibility have been developed by Veletsos.

Comparing the values of forces and moments obtained according to UBC and rigid tank approaches, with the one including tank flexibility, clearly shows the unconservatism of the first two approaches, particularly of the UBC approach, in predicting strength demands.

Analysis of some recent available analytical and experimental data on seismic response of these tanks shows that the following parameters can have significant effects on the response: the vertical component of the ground motion (breathing modes); the tank base movements due to base flexibility, and particularly due to uplift; and tank cross section distortions (out-of-round deformations). Although the problems have been identified, no general methods for modelling and/or estimating numerically these effects have been developed.

Although significant research has been conducted toward solving the problems involved in predicting the seismic behavior of this type of tank, most of the research has concentrated on isolated aspects rather than on the whole design problem. Most of these efforts have been devoted to predicting the demands, while very little has been done to determine accurately the supplied buckling strength of the shell under the complex state of stress that usually occurred in the bottom of tanks, particularly in cases of unanchored tanks. Thus no straightforward numerical optimal design procedure for these tanks has yet been developed. The most complete design procedure developed is the one proposed to be included in API Standard 650, Welded Steel Tanks for Oil Storage.

Now that the main problems involved in the seismic behavior of these tanks have been identified, a comprehensive design process which emphasizes conceptual design rather than numerical analysis, has been proposed.

The proposed design process forces the designer to consider these problems, reviewing the significant aspects of the design and construction of these tanks. In view of the multitude of uncertainties involved in establishing the seismic design forces and in the proper modelling of these tanks and, therefore, in predicting the demands, emphasis is put on the proper selection of the tank configuration, and in the detailing and construction aspects. It is shown that proper selection of the height-to-tank ratio, H/R, is a major parameter in determining the tank's earthquake vulnerability to damage. Another important parameter is the selection of the type of support: anchored or unanchored; guidelines for proper selection of the support are offered. If the H/R ratio is properly selected, it is possible to overcome the uncertainties by proper detailing and careful workmanship, and to provide extra margin of safety at a minimum cost. A series of provisions is recommended to provide for such extra safety.

RECOMMENDATIONS FOR FUTURE RESEARCH

Integrated analytical and experimental research should be conducted with the ultimate objective of developing a reliable optimal design procedure for cylindrical liquid storage tanks. To achieve this objective, reliable data is needed on:

(1) The effects of vertical excitations: "breathing" modes.

(2) The reasons for the significant out-of-roundness response under horizontal ground motions, and the importance of the initial out-of-roundness and other assymetries in this response.

(3) The effects of base flexibility and of tank movements (sliding and uplift).

(4) The uplift resistance when a large amount of uplift occurs in unanchored tanks.

(5) The actual state of stress developed in the shell and bottom plates where uplift occurs.

(6) The critical buckling strength of the shell under the real conditions existing in the seismic response of liquid-filled anchored and unanchored tanks, i.e., to find the proper values of the knockdown factor.

(7) The effective damping ratio for the liquid-tank system, as well as for the sloshing liquid.

(8) The effects of different types of roofs and of their connections to the shell.

(9) The torsional buckling of the shell: prediction of demands as well as the available strength.

Analytical and experimental studies should be continued regarding the reliability of present methods for estimating and combining the effects of impulsive and convective hydrodynamic pressures under the two components of horizontal ground motions, particularly considering the great uncertainties that exist in the definition of design earthquakes.

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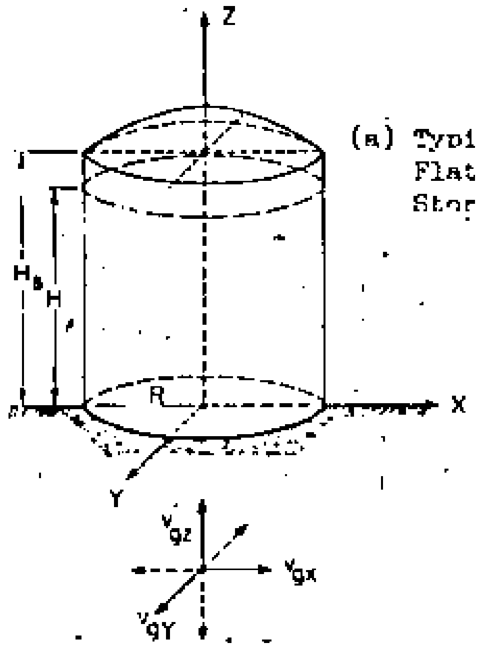
TABLE 1: COMPARISON OF RESULTS OBTAINED USING DIFFERENT METHODS OF ANALYSES OF A 1×10^6 LITER STEEL TANK

METHOD OF ANALYSIS BASED ON	VALUES OF INTERNAL FORCES AT BASE OF TANK						OVERTURNING MOMENT AT FOUNDATION (KNm)
	SHEAR (KN) DUE TO			BENDING MOMENT (KNm) DUE TO			
	IMPULSIVE	CONVECTIVE	TOTAL	IMPULSIVE	CONVECTIVE	TOTAL	TOTAL
UBC 76 (K=2)			2547			13449	
RIGID TANK	3200	429	3629	14672	3416	18088	20743
FLEXIBLE TANK $\psi_A(Z) = \sin(\frac{\pi Z}{2H})$	6077	429	6506	32424	3416	35840	39524
$\psi_C(Z) = 1-\cos(\frac{\pi Z}{2H})$	3499	429	3928	21178	3416	24594	26080



(a) Typical Cylindrical Flat Bottom Liquid Storage Tank

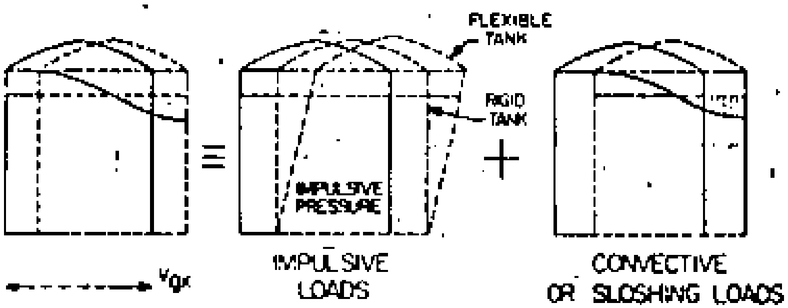
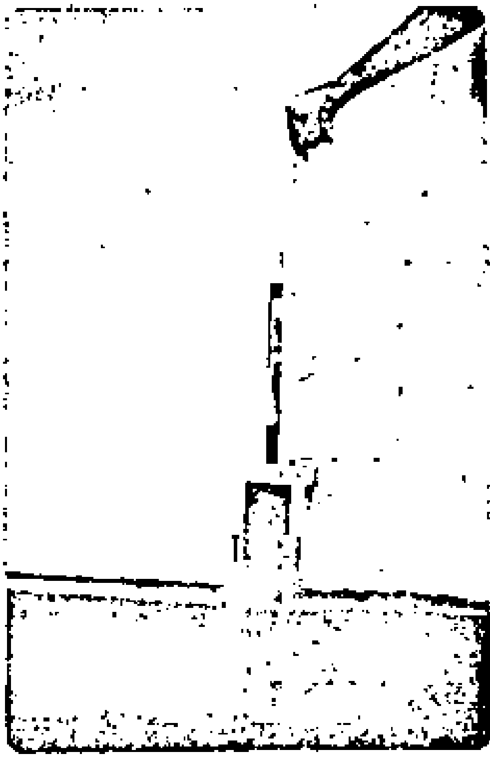


FIG. 1 TYPE OF TANK CONSIDERED AND FLUID PRESSURES AND FORCES ORIGINATED BY HORIZONTAL ACCELERATION OF GROUND

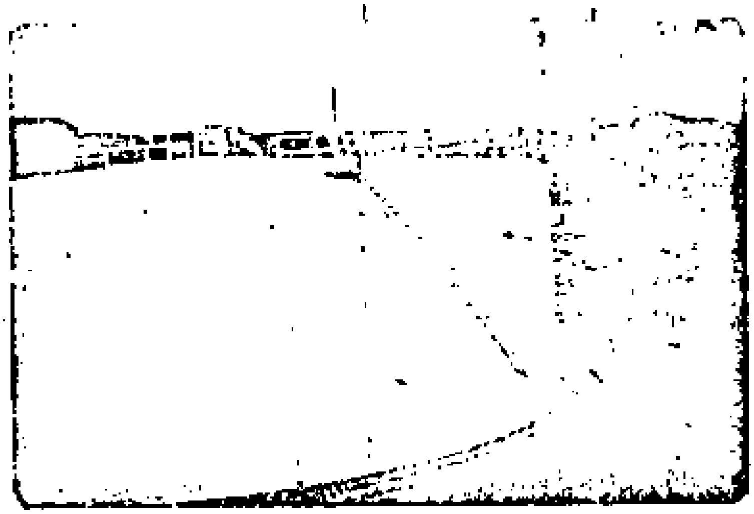
(a) Uncontrolled Fire

(b) Buckling of Tank Due to Fire

FIG. 2(a) 1964 NIIGATA EARTHQUAKE FIRE ORIGINATED BY FAILURE OF TANKS AND (b) TANK DAMAGE DUE FIRE [illegible]

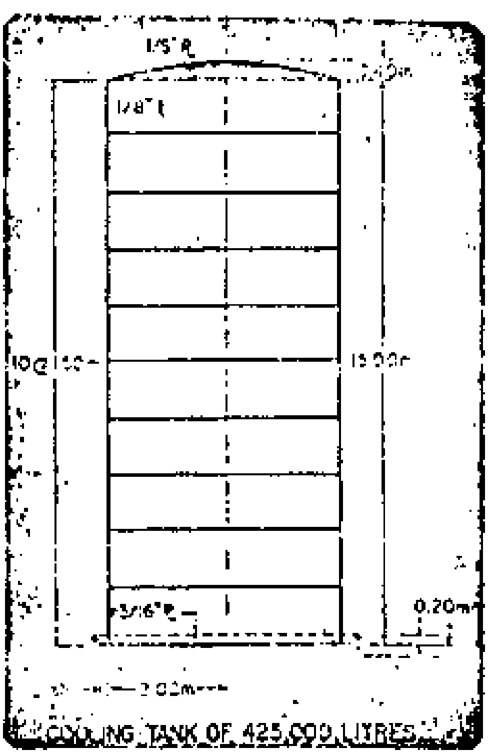


(a) Pull Out of Anchor Bolt

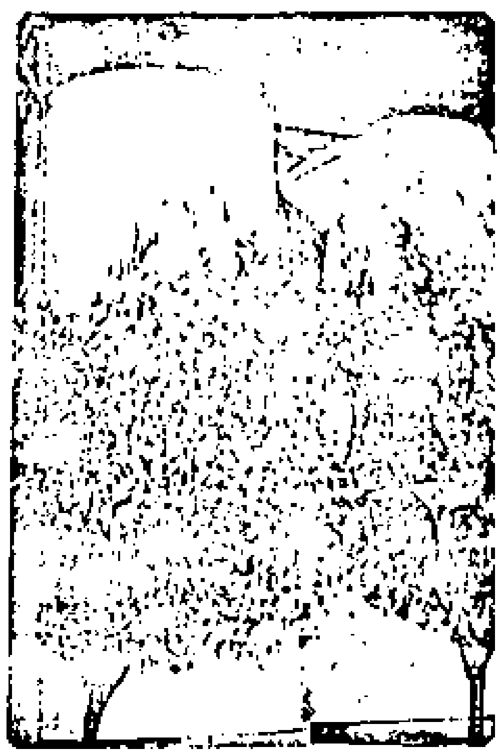


(b) Buckling of Shell Upper Part

FIG. 3 1971 SAN FERNANDO EARTHQUAKE: DAMAGE TO WATER TANK



(a) Dimensions



(b) Damage

FIG. 4 1977 SAN JUAN EARTHQUAKE: WINE COOLING TANK RESERVOIRS

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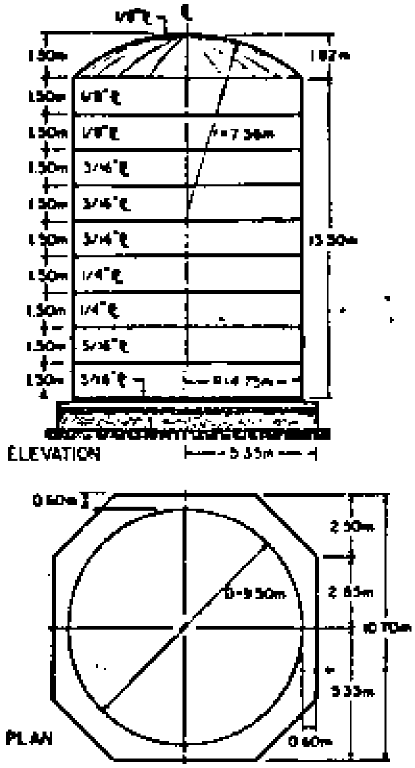
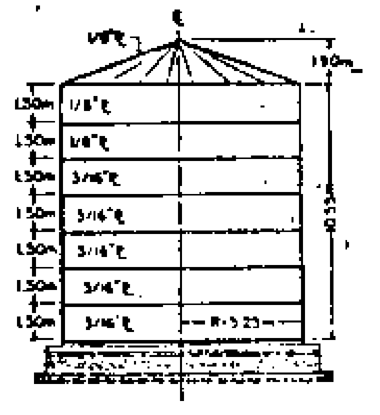
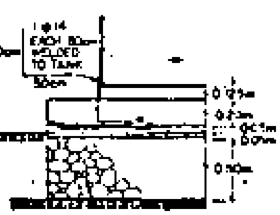
FIG. 5 1977 SAN JUAN EARTHQUAKE
DETAILS OF 1 and 1.1
MILLION LITRE TANKS

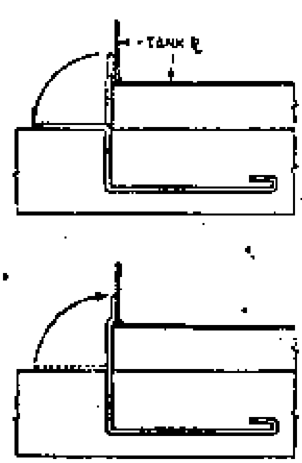




(a)



(b)



(c)

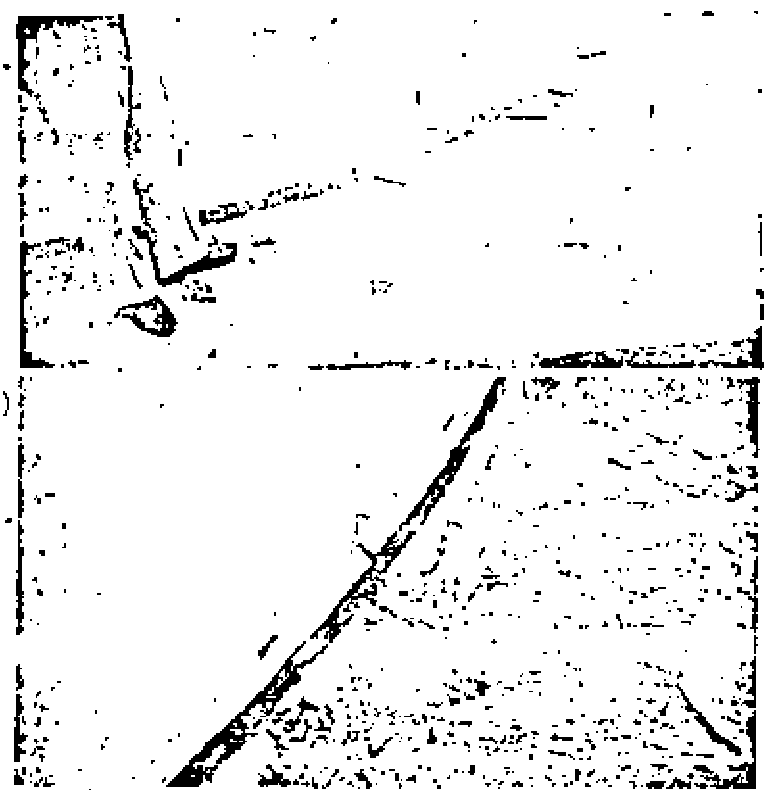


FIG. 6 DETAILS OF FOUNDATION AND ANCHORAGE OF TANKS SHOWN
IN FIG. 5 AND DAMAGE TO ANCHOR RODS.

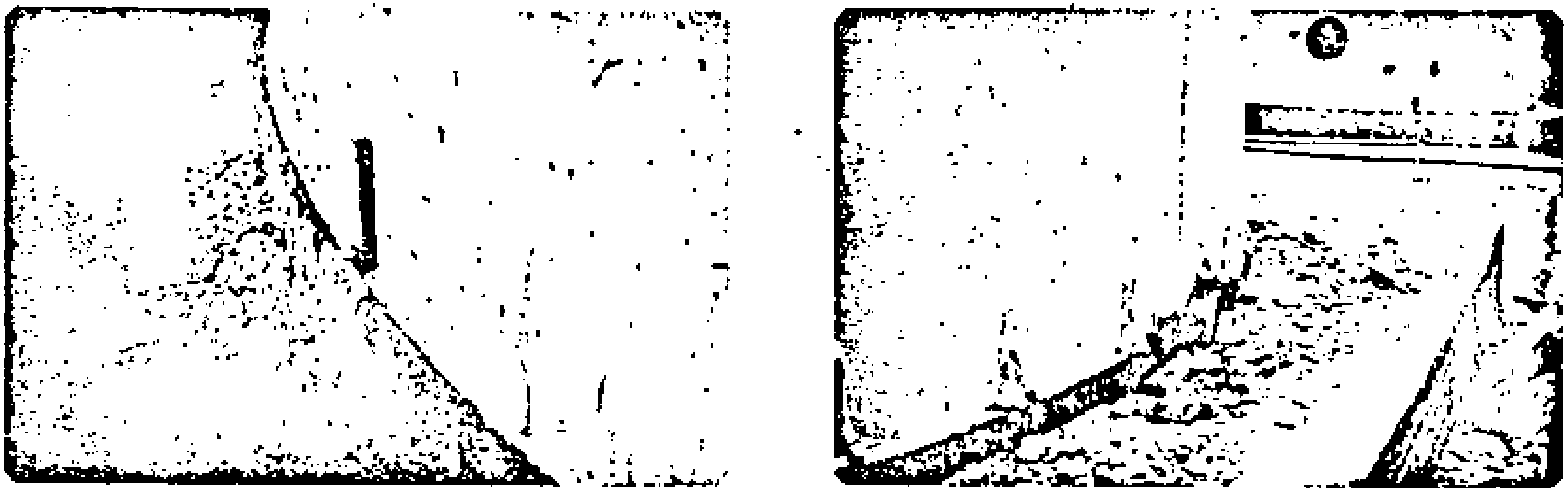


FIG. 7 1977 SAN JUAN EARTHQUAKE: FAILURE OF ANCHOR ROD AT THE KINK REQUIRED TO FACILITATE ITS ATTACHMENT (WELDING) TO THE SHELL

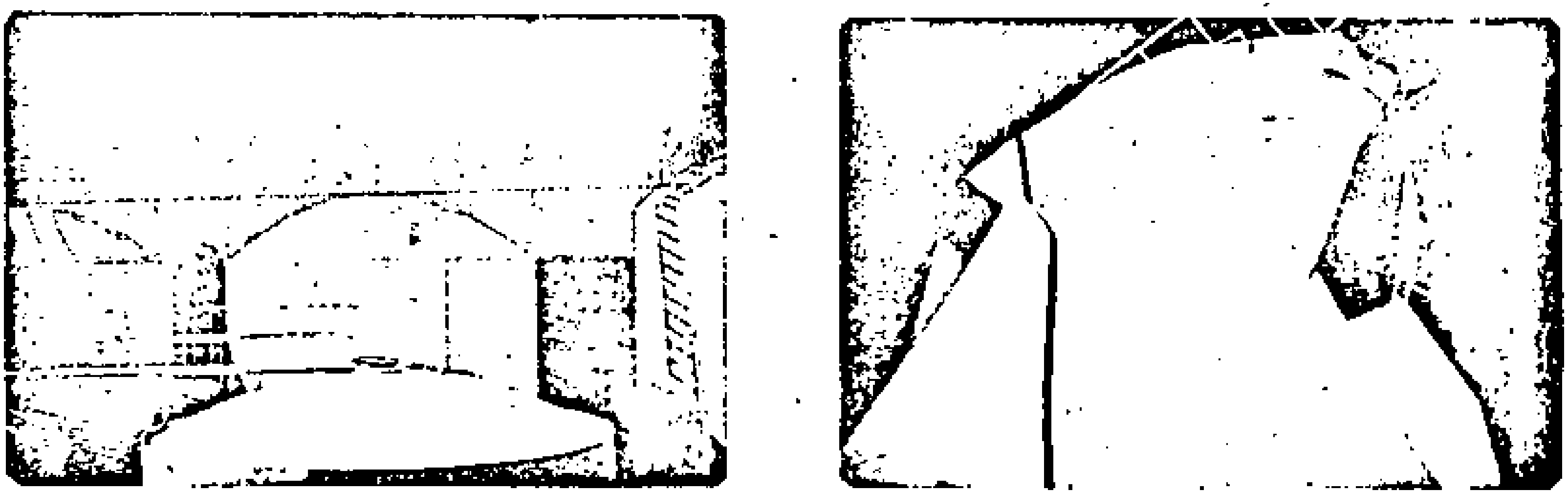


FIG. 8 1977 SAN JUAN EARTHQUAKE: DAMAGE AND FAILURE OF ROOF DUE TO SLOSHING ACTIONS AND/OR SUCTION

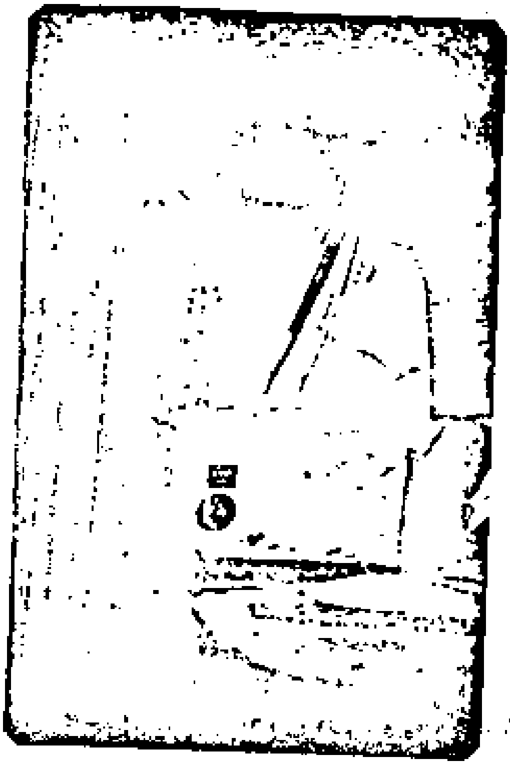


FIG. 9 1977 SAN JUAN EARTHQUAKE: TORSIONAL BUCKLING OF SHELL

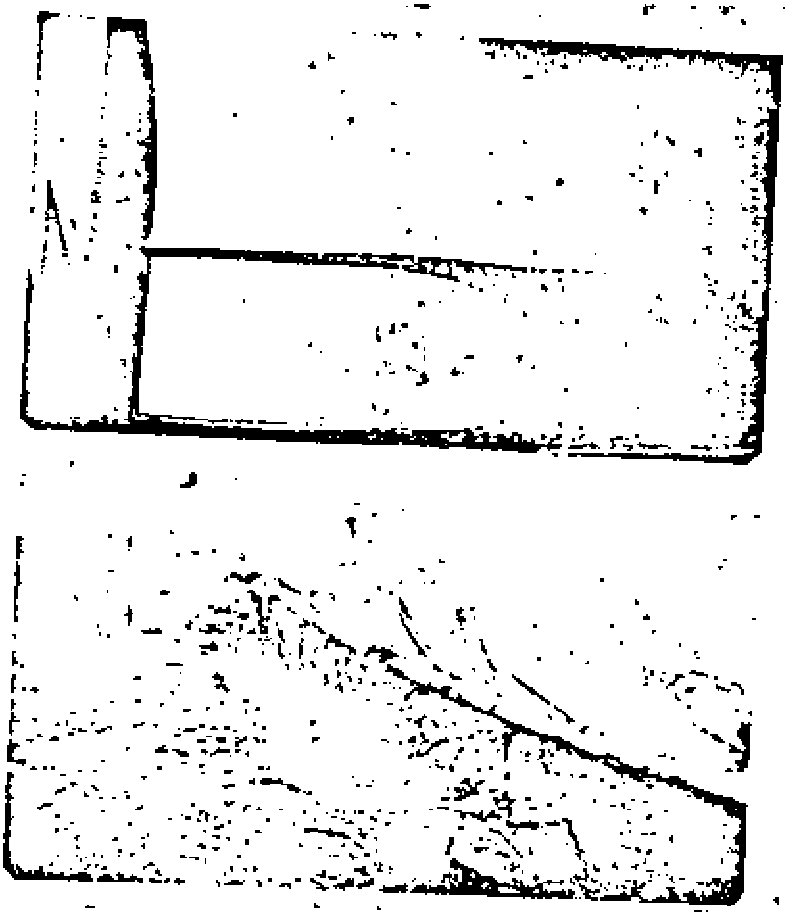


FIG. 10 1977 SAN JUAN EARTHQUAKE: BUCKLING OF SHELL (ELEPHANT'S FOOT TYPE)

FIG. 11 1977 SAN JUAN EARTHQUAKE: WELDING FAILURE AND TIER OFF OF SHELL

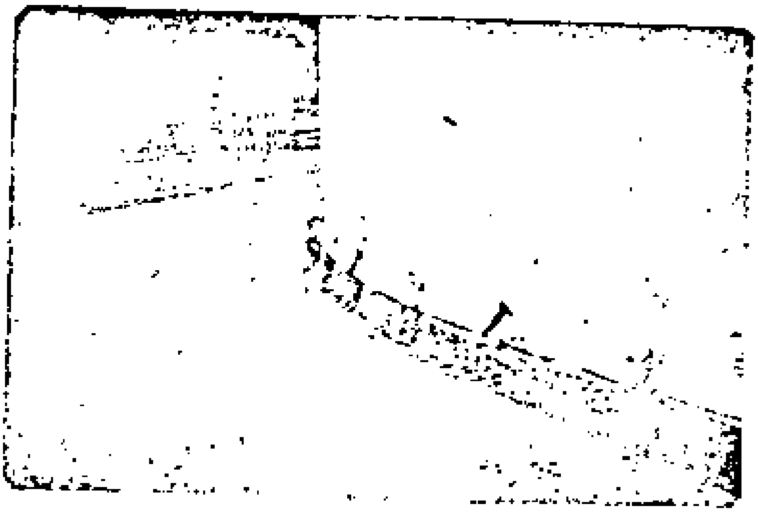
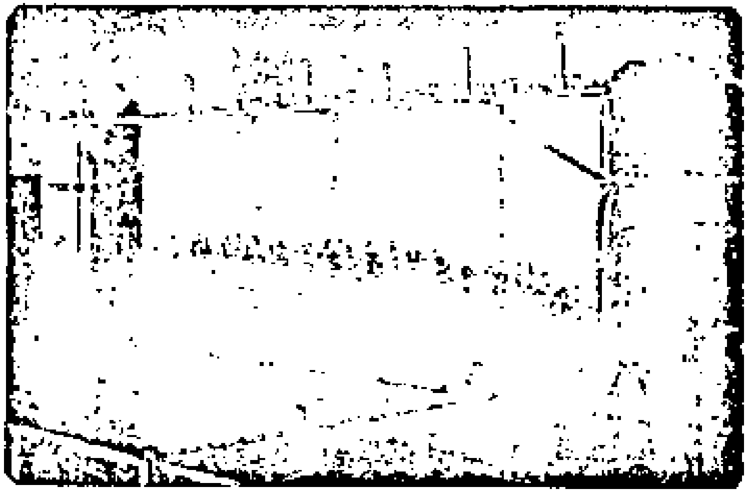


FIG. 12 1977 SAN JUAN EARTHQUAKE: SLIDING OF TANK



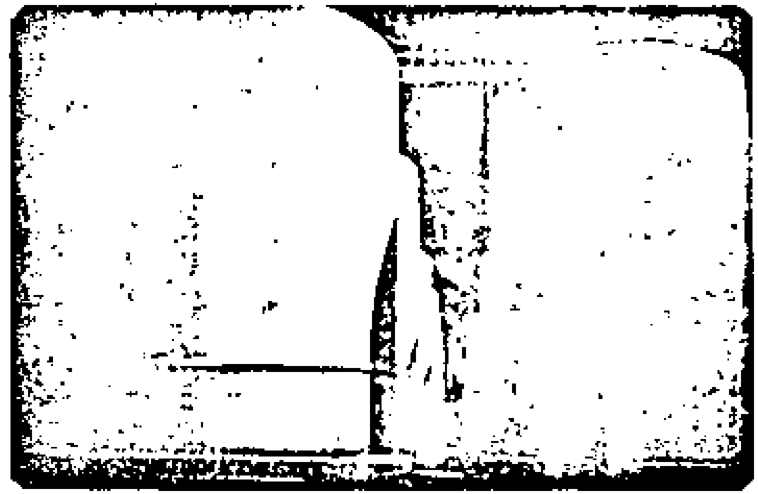


FIG. 13 1977 SAN JUAN EARTHQUAKE: INCLINATION OF TANKS DUE TO FOUNDATION FAILURE.

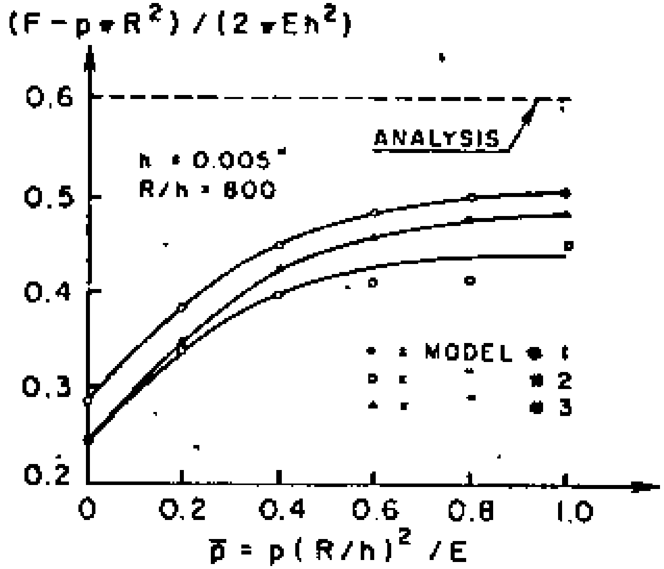


FIG. 14 CRITICAL BUCKLING STRESS VS. INTERNAL PRESSURE: TEST RESULTS OBTAINED BY SHIH AND BABCOCK [9]

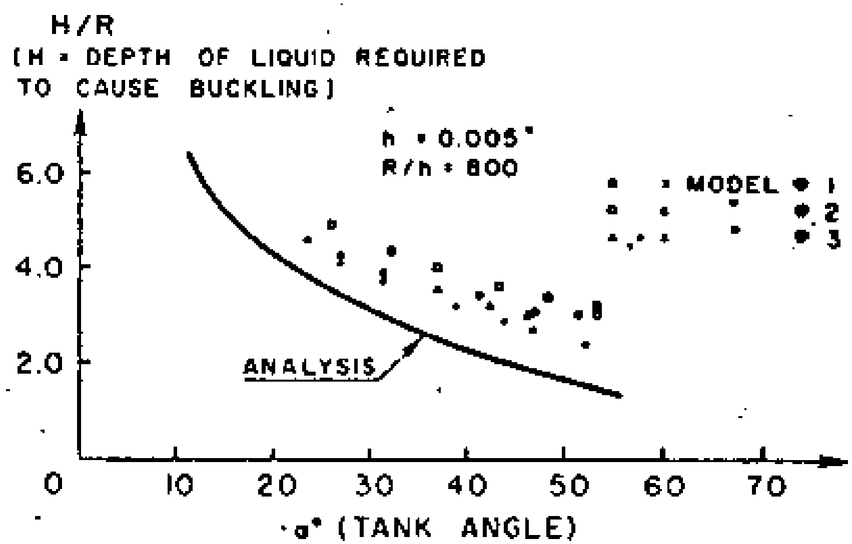


FIG. 15 COMPARISON OF INCLINED TANK TEST RESULTS WITH ANALYTICAL PREDICTED VALUES BASED ON [illegible]

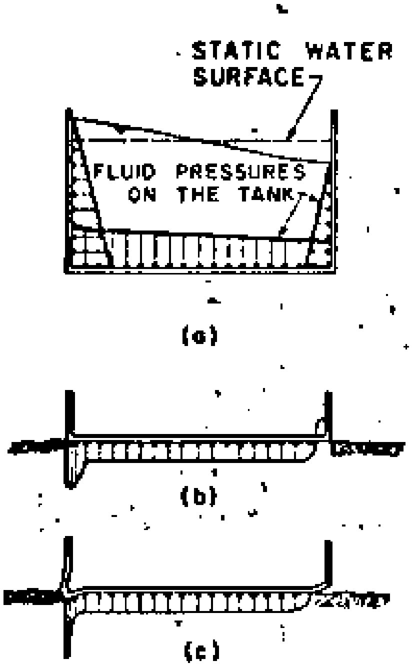


FIG. 16 CONVECTIVE PRESSURES DUE SLOSHING FLUID AND DISTRIBUTION OF PRESSURES AT THE FOUNDATION (AFTER HANSON [1])

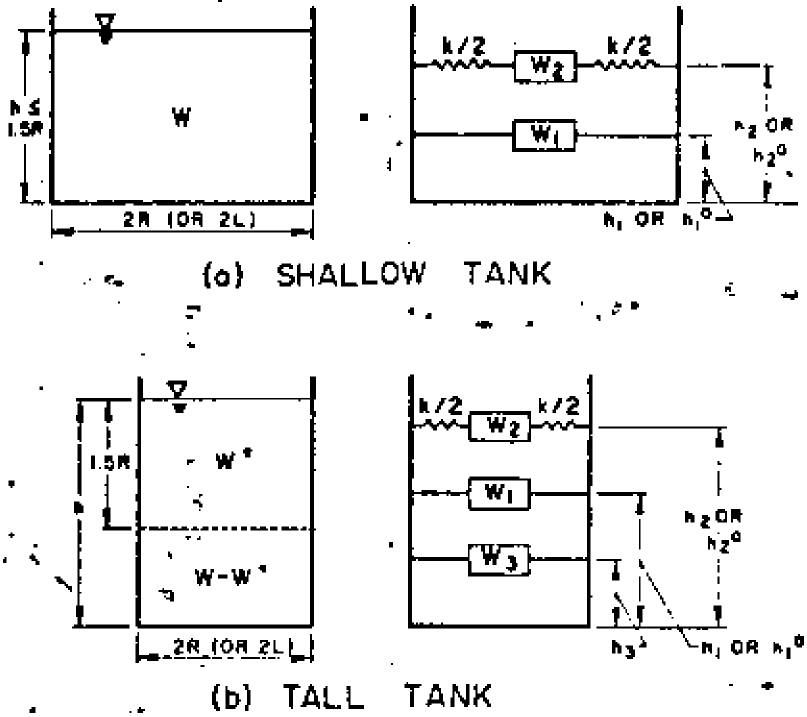


FIG. 17 EQUIVALENT MECHANICAL MODELS [23]

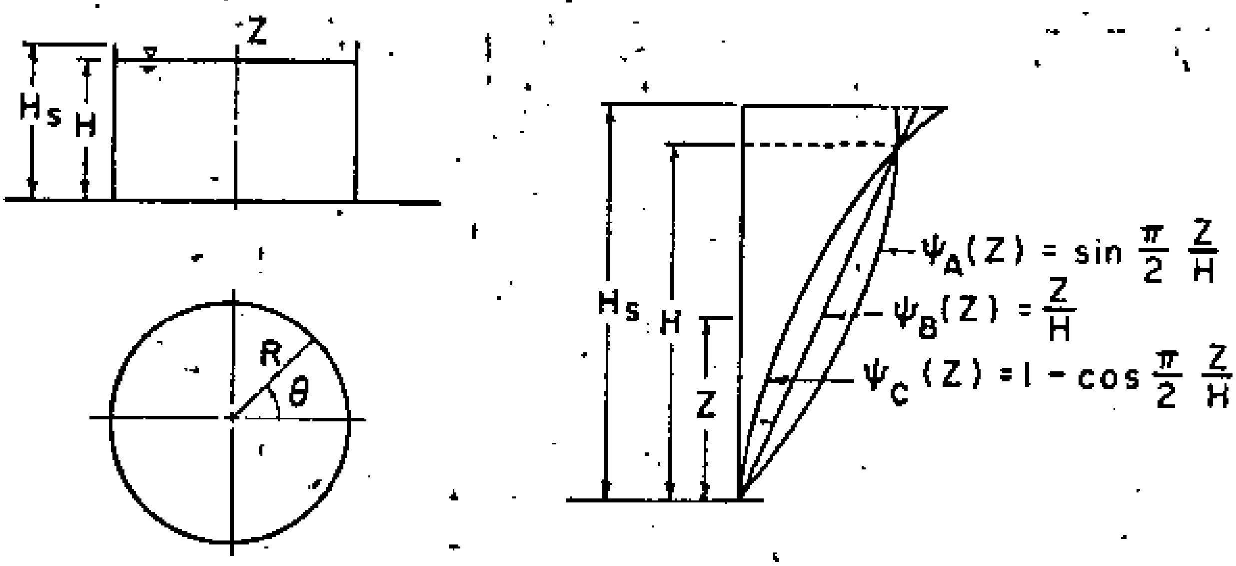


FIG. 18 LIQUID-TANK MODEL CONSIDERED BY VELETSOS [22]

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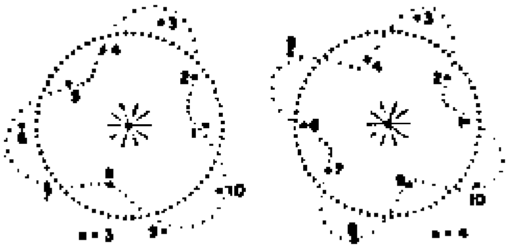

FIG. 19 COMPARISON BETWEEN MEASURED AND COMPUTED CIRCUMFERENTIAL MODE SHAPES [31]

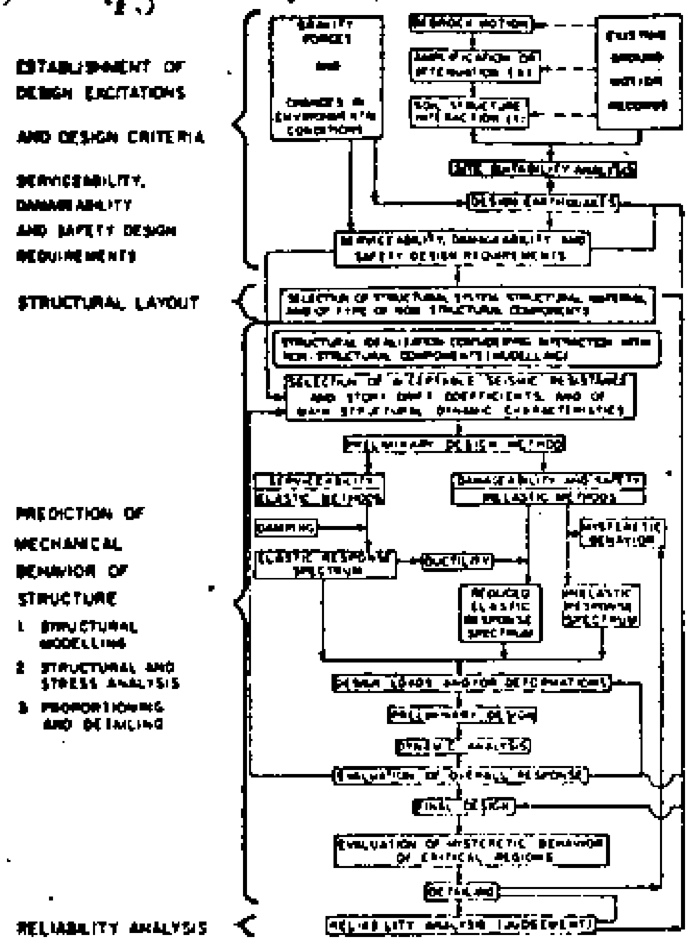

FIG. 20 FLOW DIAGRAM OF GENERAL ASPECTS INVOLVED IN COMPREHENSIVE SEISMIC RESISTANT DESIGN

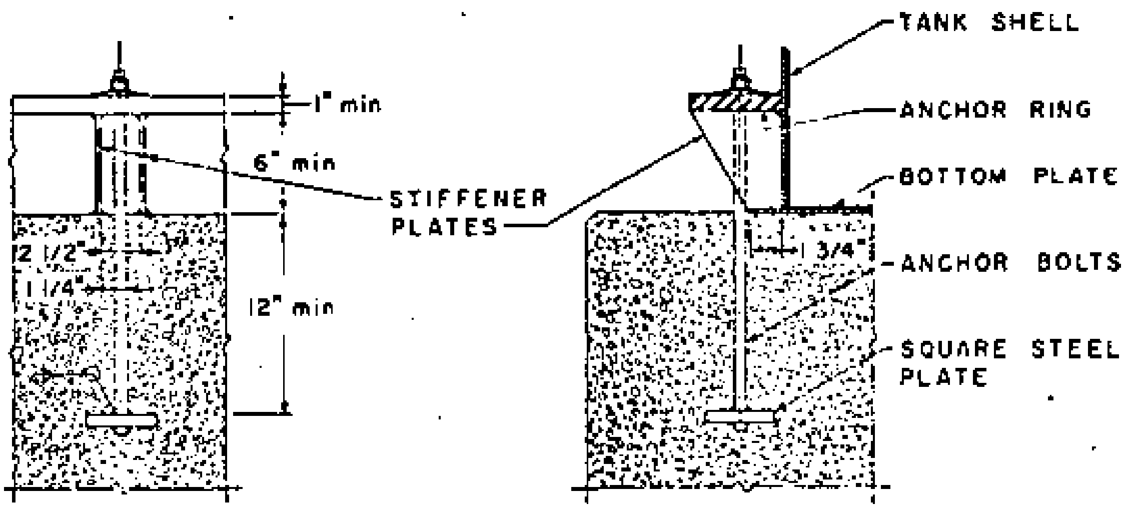

FIG. 21 ANCHORED TANKS--RECOMMENDED DETAILS FOR IMPROVEMENT OF ANCHORAGE [40]

ANALISIS HIDRODINAMICO DE TANQUES

Jesús Iglesias J. (I)
José de la Cera A. (II)

RESUMEN

Se presenta la solución matemáticamente rigurosa para el com-
portamiento hidrodinámico de tanques de paredes rígidas con el objeto
de establecer una comparación con el método analógico propuesto por
Housner cuyo uso se encuentra ampliamente extendido en la actuali-
dad. Los resultados obtenidos permiten recomendar el uso de la solu-
ción rigurosa mediante tablas o gráficas que faciliten su manejo.

INTRODUCCION

El estudio del comportamiento hidrodinámico de tanques bien pue-
de remontarse a la década de los treinta cuando Hoskins (1) y Ruge
realizaron los primeros trabajos de carácter experimental con mo-
tivo de la numerosas fallas ocurridas en este tipo de estructuras du-
rante el sismo de 1933 en Long Beach, California. Con estos estu-
dios se evidenció la necesidad de usar criterios de diseño dinámico
en vez de recurrir a la suposición de una carga estática equivalente
de diseño actuando en el centro de masa del tanque, tal como se ha
venido haciendo.

Posteriormente a fines de los años cuarenta, el enfoque analítico
del problema recibe un gran impulso con las investigaciones realiza-
das por Arias (3) y Jacobsen (4), quienes formulan la teoría de la ma-
sa reducida que permite llevar a cabo el análisis del tanque modelado
por una serie de osciladores hidrodinámicos. Estos conceptos han
sido ampliamente desarrollados por Graham (5) y Moran (6) para tan-
ques de forma tanto rectangular como cilíndrica.

En 1957 Housner (7) publica un desarrollo teórico del problema ba-
sado en una analogía mecánica que no satisface el equilibrio dinámico
del fluído (9), y que conduce a la obtención de modos senoidales in-
dependientemente de la forma del recipiente, todo esto a diferencia de

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los trabajos ya mencionados, que parten de la búsqueda del potencial
de velocidades del líquido que cumpla con la ecuación de Laplace y
las condiciones de frontera.

La presentación eminentemente práctica de los artículos de
Housner (7,8) y la gran difusión que han tenido, son la causa de que
hayan sido tomados como base para el diseño dinámico de tanques por
varias normas (10,11), desechando la aparente complicación que im-
plica el manejo de los modelos matemáticamente rigurosos. Esta si-
tuación hace sentir la necesidad de analizar con detenimiento y desde
un punto de vista tanto teórico como de aplicación la conveniencia de
la elección hecha a favor del método de Housner.

DETERMINACION DEL POTENCIAL DE VELOCIDADES

La solución del movimiento irrotacional de un fluido homogéneo e
incompresible, se reduce a encontrar el potencial de velocidades ϕ
que satisfaga la ecuación de continuidad representada por la ecuación
de Laplace,

$$\nabla^2 \phi = 0 \qquad (1)$$

y las condiciones de frontera correspondientes (12).

Para el caso de un tanque de paredes rígidas, las condiciones de
frontera en el fondo y las paredes están dadas por la consideración de
que el fluído adyacente sólo tiene respecto a ellas velocidad tangen-
cial. En la superficie libre, si se suponen desplazamientos de oleaje
pequeños, la condición de frontera es la de Poisson (12),

$$g \frac{\partial \phi}{\partial z} + \frac{\partial^2 \phi}{\partial t^2} = 0 \qquad (2)$$

donde g es la aceleración de la gravedad, t la variable temporal y
el sistema de referencia es el indicado en las Figs 1 y 3.

TANQUE RECTANGULAR

Si se considera que el desplazamiento inicial ocurre en la direc-
ción X (Fig 1), la ecuación de continuidad será,

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial z^2} = 0 \qquad (3)$$

las condiciones de frontera en paredes y fondo,

$$\left.\frac{\partial \phi}{\partial x}\right|_{x=\pm a} = 0 \quad ; \quad \left.\frac{\partial \phi}{\partial z}\right|_{z=-h} = 0 \tag{4}$$

y la condición de frontera en la superficie libre

$$\left. g\,\frac{\partial \phi}{\partial z} + \frac{\partial^2 \phi}{\partial t^2} = 0 \right|_{z=0} \tag{5}$$

Proponiendo un potencial del tipo

$$\phi = A\, X(x)\, Z(z)\, T(t) \tag{6}$$

donde A es una constante, X, Z y T son funciones de x, z y t respectivamente y después de sustituir en la ecuación de Laplace, integrar e introducir las condiciones de frontera, dicho potencial queda definido en función de los n modos del movimiento como

$$\phi = \sum_{n=1}^{\infty} \phi_n = \sum_{n=1}^{\infty} A_n \sin(k_n x)\cosh\left[k_n(h+z)\right]\cos(\sigma_n t + \varepsilon_n) \tag{7}$$

donde A_n y ε_n son constantes y

$$\sigma_n^2 = g\, k_n \tanh(k_n h) \tag{8}$$

y

$$k_n = \left(\frac{n-1/2}{a}\right)\pi \tag{9}$$

Puesto que el perfil del líquido en la superficie está definido (12) por

$$\eta = \left.\frac{1}{g}\,\frac{\partial \phi}{\partial t}\right|_{z=0} \tag{10}$$

se observa que la forma de los modos es senoidal (Fig 2)

$$\eta_n = -A_n \sin(k_n x)\cosh(k_n h)\cos(\sigma_n t + \varepsilon_n) \tag{11}$$

Esta coincidencia en la forma modal con el método de Housner, no se presenta para otros tipos de tanques como se verá más adelante.

Una vez obtenido ϕ, es posible hallar la energía cinética y el

impulso del fluído para un modo dado (12) mediante

$$T_n = \frac{\rho}{2}\int_V \bar{q}_n^{\,2}\, dV \quad , \quad I_n = \rho \int_V \bar{q}\, dV \tag{12}$$

donde

$$\bar{q}_n = \left\{ \frac{\partial \phi_n}{\partial x},\ \frac{\partial \phi_n}{\partial y},\ \frac{\partial \phi_n}{\partial z} \right\} \tag{13}$$

Efectuando el cociente

$$\frac{I_n^2}{2T_n} = \frac{(M_n q_n)^2}{2\left(\frac{1}{2} M_n q_n^2\right)} = M_n \tag{14}$$

se obtiene la masa del oscilador hidrodinámico correspondiente al modo n

$$M_n = M\left[\frac{2\tanh\left(\Pi_n \frac{h}{a}\right)}{\frac{h}{a}\,\Pi_n^3}\right] = M R_n \tag{15}$$

donde M es la masa total del líquido y

$$\Pi_n = \pi\left(n - \frac{1}{2}\right) \tag{16}$$

En consecuencia, el factor de masa reducida queda determinado por

$$R_0 = 1 - \sum_{n=1}^{\infty} R_n \tag{17}$$

Graham (5), trabajando el potencial de velocidades a través de la ecuación de presiones llega a obtener los mismos resultados, además de determinar las posiciones de las masas equivalentes y de sus correspondientes resortes (Fig 6)

$$M_n = M R_n \quad ; \quad M_0 = M R_0$$

$$H_n = \left[1 - \frac{2 \tanh\left(\frac{\pi_n h}{2a}\right)}{\pi_n \frac{h}{a}} \right] h$$

$$H_0 = \left[\frac{1}{2} - \frac{1}{h M_0} \sum_{n=1}^{\infty} M_n H_n \right] h \qquad (18)$$

$$K_n = \frac{2 M g \tanh^2\left(\pi_n \frac{h}{a}\right)}{h \pi_n^2}$$

TANQUE CILINDRICO

En este caso la ecuación de Laplace en coordenadas cilíndricas (Fig .) es

$$\frac{1}{r^2}\frac{\partial^2 \phi}{\partial \theta^2} + \frac{\partial^2 \phi}{\partial r^2} + \frac{1}{r}\frac{\partial \phi}{\partial r} + \frac{\partial^2 \phi}{\partial z^2} = 0 \qquad (19)$$

y las condiciones de frontera son

$$\left.\frac{\partial \phi}{\partial r}\right|_{r=a} = 0 \quad ; \quad \left.\frac{\partial \phi}{\partial z}\right|_{z=-h} = 0 \quad ; \quad \left.\frac{\partial \phi}{\partial \theta}\right|_{\theta=0,\pi} = 0$$

$$\left. g\frac{\partial \phi}{\partial z} + \frac{\partial^2 \phi}{\partial t^2} = 0 \right|_{z=0} \qquad (20),$$

A partir del potencial

$$\phi = A\, R(r)\, \Theta(\theta)\, Z(z)\, T(t) \qquad (21)$$

Sustituyendo en la ecuación de Laplace, integrando e introduciendo las condiciones de frontera, es posible definir dicho potencial como

$$\phi = \sum_{n=1}^{\infty} \phi_n = \sum_{n=1}^{\infty} A_n J_1\left(\frac{\lambda_n r}{a}\right) \cosh\left(\lambda_n \frac{h+z}{a}\right) \cos(p_n t + \varepsilon_n) \cos\theta \qquad \ldots(22)$$

donde J_1 es la función de Bessel de 1a. especie de orden 1, λ_n son las raíces de J_1' y

$$p_n^2 = \frac{g}{a} \lambda_n \tanh\left(\lambda_n \frac{h}{a}\right) \qquad (23)$$

Ahora los modos ya no son senoidales, sino que adquieren la forma de funciones J_1 de Bessel (Fig 4)

$$\eta_n = -A_n p_n J_1\left(\lambda_n \frac{r}{a}\right) \cosh\left(\lambda_n \frac{h}{a}\right) \operatorname{sen}(p_n t + \varepsilon_n) \cos\theta \qquad (24)$$

con lo cual surge una diferencia irreconciliable con el método de Housner.

De manera similar al caso anterior es posible obtener las masas equivalentes como

$$M_n = \frac{I_n^2}{T_n} = M \frac{2 \tanh\left(\lambda_n \frac{h}{a}\right)}{\lambda_n (\lambda_n^2 - 1) \frac{h}{a}} = M R_n \quad ; \quad R_0 = 1 - \sum_{n=1}^{\infty} R_n \qquad \ldots(25)$$

Aunque con un desarrollo más complicado, Moran (8) encontró también los resultados anteriores e inclusive las expresiones para la posición de las masas y las rigideces de los resortes hidrodinámicos (Fig 5)

$$M_n = M R_n \quad ; \quad M_0 = M R_0$$

$$H_n = \frac{h}{R_n}\left(R_n - S_n + S_n'\right)$$

$$H_0 = \frac{h}{R_0}\left(R_0 + 2S_0 - \frac{1}{2}\right) \qquad (26)$$

$$K_n = \frac{2 M g \tanh\left(\lambda_n \frac{h}{a}\right)}{h \lambda_n (\lambda_n^2 - 1)}$$

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$$S_n = \frac{2\left[1-\operatorname{sech}\left(\lambda_n \frac{h}{a}\right)\right]}{\lambda_n^2\left(\lambda_n^2-1\right)\left(\frac{h}{a}\right)^2}$$

$$S_n^1 = \frac{2\operatorname{sech}\left(\lambda_n \frac{h}{a}\right)}{\lambda_n^2\left(\lambda_n^2-1\right)\left(\frac{h}{a}\right)} \;;\quad S_0 = \sum_{n=1}^{\infty} S_n$$

COMPARACION NUMERICA

Se llevó a cabo un análisis numérico para comparar los resultados obtenidos en R_1 y R_0 tanto con las expresiones de Housner tomadas de (10), como con las analíticas considerando los primeros 20 modos para el cálculo de R_0.

Tanque Rectangular. Aquí se aprecia (Fig 6) una pequeña diferencia en cuanto al factor de reducción de masa del primer modo, que se justifica porque en ambos casos es senoidal la forma de los modos, sin embargo en lo que respecta a la masa adherida, su factor de reducción presenta hasta un 10% de diferencia.

Tanque Cilíndrico. Para este tipo de tanque, en el que como se vió anteriormente existe una discrepancia en las formas modales, se presentan diferencias en el cálculo de la masa del oscilador hidrodinámico del primer modo hasta de 17% y en el de la masa adherida hasta de 7%.

CONCLUSION

Teniendo en cuenta el aspecto teórico del problema no es difícil aceptar la mayor solidez en el planteamiento riguroso seguido al principio de este trabajo, en relación a la analogía mecánica propuesta por Housner, hecho que se ve confirmado por el trabajo experimental (13).

En cuanto al aspecto práctico, si bien las expresiones analíticas son complicadas pues el cálculo de R_0 requiere de la evaluación de una serie infinita y en el caso del tanque cilíndrico es necesario trabajar con las funciones de Bessel, el recurso de elaborar tablas o gráficas para obtener los valores requeridos en función de la relación de aspecto del tanque elimina esta dificultad.

En base a lo anterior y teniendo en cuenta que el uso de la analogía de Housner puede conducir a errores de magnitud apreciable, se concluye que es más adecuado e igualmente práctico el uso de los resultados matemáticamente rigurosos.

Reconocimiento. Los autores agradecen al Profesor A. Arias las sugerencias y comentarios al presente trabajo, además de su personal contribución al desarrollo del estudio del comportamiento hidrodinámico de tanques, fruto de una vida ejemplar de dedicación científica.

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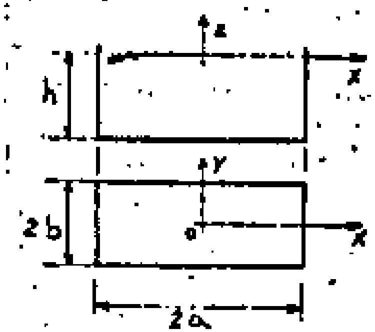


Fig 1. Sistema coordenado del tanque rectangular

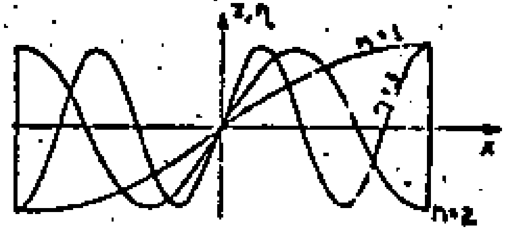


Fig 2. Formas modales en la superficie del tanque rectangular (senoidales).

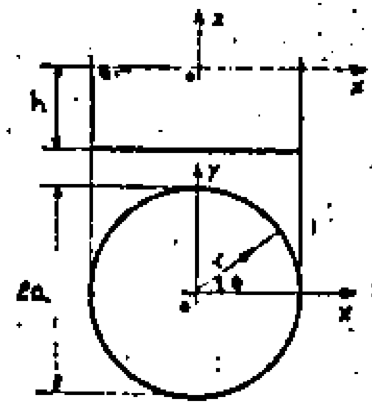


Fig 3. Sistema coordenado del tanque cilíndrico

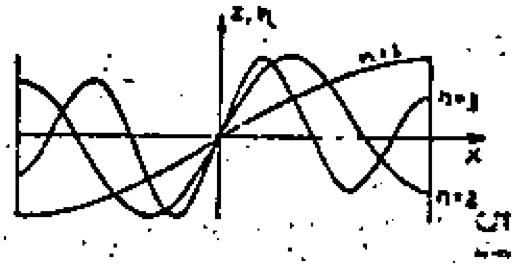


Fig 4. Formas modales en la superficie del tanque cilíndrico.

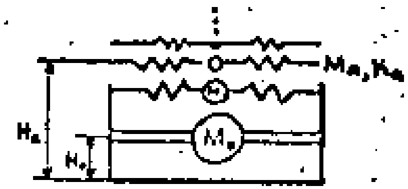
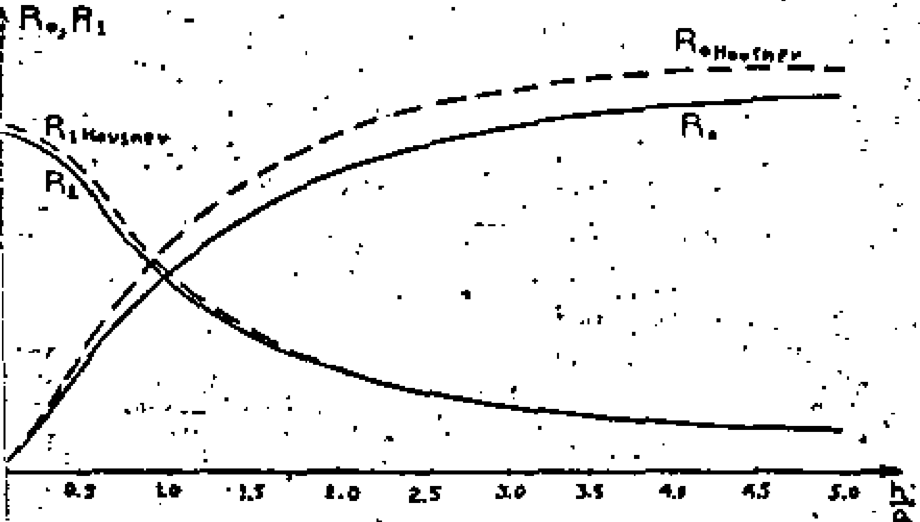
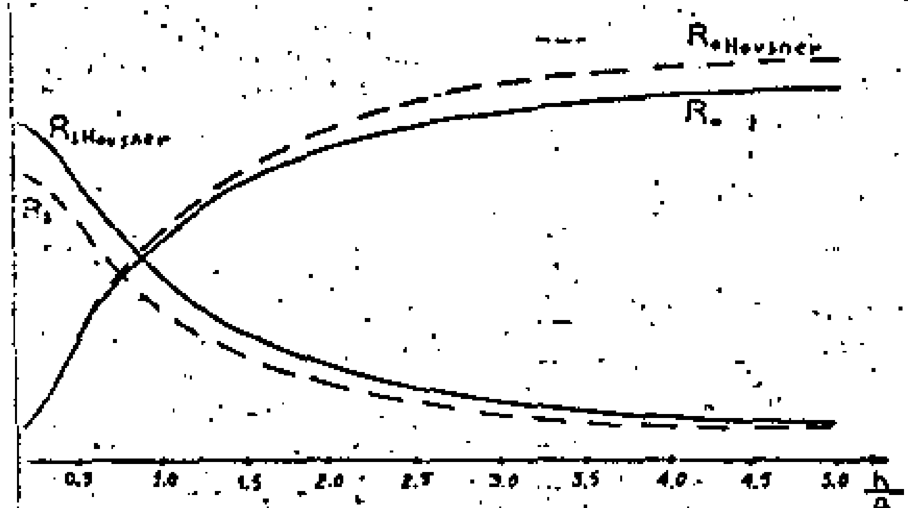


Fig 5. Osciladores hidrodinámicos y masa adherida.



3. Factores de reducción de masa del tanque rectangular.



. Factores de reducción de masa del tanque cilíndrico.

PRESENTED AT THE SESSION ON
ADVANCES IN STORAGE TANK DESIGN
API, REFINING
43RD MIDYEAR MEETING
SHERATON CENTRE
TORONTO, ONTARIO, CANADA
TUESDAY, MAY 9, 1978

BASIS OF SEISMIC DESIGN PROVISION FOR WELDED STEEL OIL STORAGE TANKS

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Standard Oil Company of California
San Francisco, California

CBT—5359

BASIS OF SEISMIC DESIGN PROVISIONS FOR WELDED STEEL OIL STORAGE TANKS

R. S. Wozniak[1] and W. W. Mitchell[2]

ABSTRACT

Recommended design provisions are described for the seismic design of flat bottom storage tanks which are proposed to be included in API Standard 650. The basis for establishing design loads is presented including seismic zone coefficients and the essential facilities factor. The design procedure is based on the approximate method of Professor Housner except that amplification of ground motion is recognized in determining the impulsive response. The derivation of curves is presented which simplify the calculation of the weights of the effective masses of tank contents, their centers of gravity, and the period of vibration of the sloshing mode. The basis of the design lateral force coefficients is given.

Resistance to overturning for unanchored tanks is provided by the tank shell and a portion of the tank contents which depends on the width of bottom annular ring which may lift off the foundation. The basis for determining this width is presented. A curve is included in the provisions for calculating the maximum longitudinal compression force in the shell for unanchored tanks. The derivation of this curve is presented and an approximate formula for the curve given. The formulas for maximum longitudinal compression force in the shell for anchored tanks and required anchorage resistance are explained. The basis is given for establishing the maximum allowable shell compression which takes into account the effect of internal pressure due to the liquid contents.

Supplemental information is presented for calculating the height of sloshing of the liquid contents, for designing roof support columns to resist forces caused by sloshing, and to calculate the increase in hoop tension in the shell due to seismic forces.

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INTRODUCTION

Reports of damage from major earthquakes within the last few decades cite cases of damage to flat bottom welded steel storage tanks [1, 2, 3, 4][3]. Damage to the tanks falls in four general categories:

1. Buckling of the bottom of the tank shell due to longitudinal compressive stresses resulting from overturning forces. This buckling is most frequently in the form of an outward bulge in the bottom foot or two of the tank shell extending partly or completely around the tank termed an "elephant's foot bulge." Damage of this type has generally been limited to unanchored tanks ranging between 10 and 100 feet in diameter. Loss of contents has resulted in some of the more severely buckled tanks.

2. Damage to the roof and upper shell of the tank and to internal roof support columns due to sloshing of the tank contents.

3. Damage to piping and other appurtenances connected to a tank due to movement of the tank.

4. Damage resulting from failure of the supporting ground, notably from liquefaction, washout due to broken piping, and slope failure due to high edge loads.

The damage reports have led to increasing interest in the seismic design of tanks to be located in seismically active areas. Tank builders, owners, and, in some instances, regulatory agencies have developed their own criteria for seismic design. To provide uniform guidelines, recommended design provisions have been prepared which are proposed to be included as an appendix (Appendix P) in API Standard 650, Welded Steel Tanks for Oil Storage. Similar provisions are being developed by the American Water Works Association for water storage tanks.

SCOPE OF DESIGN PROVISIONS

The proposed Appendix P to API Standard 650 covering seismic design of storage tanks is included in Appendix 1 to this paper. Detailed requirements are included to assure

[3] References are listed at the end of the text.

stability of the tank shell against overturning and to preclude buckling of the tank shell due to longitudinal compression for the level of earthquake ground motion which has a reasonable likelihood of not being exceeded during the life of the tank in the region in which the tank will be located.

A requirement is included to provide suitable flexibility in piping attached to the shell or bottom of the tank. Additional items which the tank purchaser may wish to consider to minimize or avoid overflow and damage to the roof and upper shell and to roof support columns are noted. These latter items normally do not pose a risk to life safety or to the safety of surrounding facilities, but may be considered from the standpoint of economic risk to the tank itself. Guidelines are included later in this paper for the design to accomplish these objectives.

The response of tanks to earthquake ground motion also includes an increase in hoop tension in the shell. This has led to rupture of the shell in the past for riveted tanks. The shells of welded tanks, however, have substantial ductility in hoop tension and can absorb energy resulting from earthquake ground motion through yielding. Current practice for the seismic design of welded steel tanks for hoop tension takes this ductility into account. When this is done, seismic response in hoop tension does not govern the design of the shell for the maximum level of earthquake ground motion proposed for API Standard 650. Consequently, no provisions are included for hoop tension. When tanks are designed for higher levels of earthquake ground motion, increased hoop tension should be investigated. Guidelines for this are included later in this paper.

The proposed Appendix P does not address soil stability since this does not affect the design of the tank. However, it is important that soil conditions at prospective tank sites in seismically active areas be investigated for potential instability including liquefaction during an earthquake.

DESIGN LOADING

The design procedure presented in Appendix P is based on the simplified procedure developed by Professor G. W. Housner [5] and included in Chapter 6 and Appendix F of ERDA TID 7024 [6] with modifications as suggested by Professor A. S. Veletsos [7]. As noted in the Introduction to Appendix P, the procedure considers two response modes of the tank and its contents: the response of the tank shell and roof together with a portion of the contents which moves in unison with the shell, and the fundamental sloshing mode of the contents. The forces associated with these modes are normally termed the impulsive force and the convective force respectively. The design overturning moment at the bottom of the shell resulting from these forces is given by the following formula in Section P.3.1:

$$M = ZI(C_1W_sX_s + C_1W_rH_t + C_1W_1X_1 + C_2W_2X_2) \qquad (1)$$

In this formula, C_1 and C_2 are the respective lateral force coefficients for the impulsive and convective forces and W_1 and W_2 are the corresponding weights of the effective masses of the tank contents. Curves for determining W_1 and W_2 as a ratio to the total weight of tank contents, W_T are given in Figure P-2 of Appendix P for various ratios of tank diameter, D, to maximum filling height, H. These curves are based on the formulas developed by Housner and presented in TID 7024. For the weight of the liquid contributing to the impulsive force, W_1, Housner presents the following formula for tanks where the ratio of filling height to radius is less than 1.5 (D/H greater than 1.333):

$$\frac{W_1}{W_T} = \frac{\tanh 0.866\frac{D}{H}}{0.866\frac{D}{H}} \qquad (2a)$$

Where the ratio of filling height to radius is greater than 1.5 (D/H less than 1.333), Housner's procedure considers the liquid contents in the lower part of the tank below a depth equal to 1.5 times the radius to respond as a rigid body as far as impulsive forces are concerned. The effective weight of the upper portion of the contents is determined from formula (2a) using D/H = 1.333. The total effective weight is determined by adding the full weight of the lower portion of the contents to the effective weight of the upper portion. This leads to the formula:

$$\frac{W_1}{W_T} = 1.0 - 0.218\frac{D}{H} \qquad (2b)$$

The formula for the weight of the effective contents used to determine the convective force, which is based on Housner's corrected version of TID 7024, is as follows:

$$\frac{W_2}{W_T} = 0.230\ \frac{D}{H}\tanh\left(\frac{3.67}{D/H}\right) \qquad (3)$$

The heights X_1 and X_2 from the bottom of the tank shell to the centroids of the lateral seismic forces applied to W_1 and W_2, respectively, as ratios to the maximum filling height, H, are given in Figure P-1 of Appendix P for various D/H ratios. Again, these are based on the work of Housner. For tanks where the ratio of filling height to radius is less than 1.5 (D/H greater than 1.333), the formula for the height to the centroid of the impulsive force is:

$$\frac{X_1}{H} = 0.375 \qquad (4a)$$

Where the ratio of filling height to radius is greater than 1.5 (D/H less than 1.333):

$$\frac{X_1}{H} = 0.500 - 0.094\frac{D}{H} \qquad (4b)$$

The formula for the height to the centroid of the convective force is:

$$\frac{X_2}{H} = 1.0 - \frac{\cosh\left(\frac{3.67}{D/H}\right) - 1.0}{\frac{3.67}{D/H}\sinh\left(\frac{3.67}{D/H}\right)} \qquad (5)$$

As noted at the end of Paragraph P.3.1, the overturning moment calculated in accordance with formula (1) is that applied to the bottom of the shell. The total overturning moment applied to the foundation can be determined by substituting X_1' and X_2' from the following formulas for X_1 and X_2, respectively, in formula (1):

$\frac{D}{H} > 1.333$:

$$\frac{X_1'}{H} = 0.375\left[1.0 + 1.333\left(\frac{0.866\frac{D}{H}}{\tanh 0.866\frac{D}{H}} - 1.0\right)\right] \qquad (6a)$$

$\frac{D}{H} < 1.333$:

$$\frac{X_1'}{H} = 0.500 + 0.060\frac{D}{H} \qquad (6b)$$

$$\frac{X_2'}{H} = 1.0 - \frac{\cosh\left(\frac{3.67}{D/H}\right) - 1.937}{\frac{3.67}{D/H}\sinh\left(\frac{3.67}{D/H}\right)} \qquad (7)$$

Tanks on the ground are inherently rigid. In his work, Housner considered the tank to be infinitely rigid so that the motion of the tank shell and roof together with that portion of the contents that moves in unison with the shell coincides with ground motion. In reality, tanks are not infinitely rigid. Storage tanks typically have natural periods of vibration in the range of 0.10 to 0.25 seconds. Veletsos, in this study of thin wall flexible tanks, concludes that the impulsive force can be reasonably well estimated from the solutions derived for a rigid tank except replacing the maximum ground acceleration with the spectral value of the pseudo-acceleration corresponding to the fundamental natural frequency of the tank-fluid system. Since the calculation of the fundamental period is complex for tanks which do not experience uplift and unknown for those that do, a constant value is proposed in Appendix P for C_1 which represents the maximum amplified ground motion. The value of 0.24 is consistent with the Uniform Building Code maximum value for structures other than buildings (K = 2.0) excluding any soil factor. The soil factor does not appear appropriate for structures with a very low natural period of vibration. The high value of C_1 in comparison with buildings is appropr because of the low damping inherent for storage tanks, the lack of nonstructural load bearing elements, and the lack of ductility of the tank shell in longitudinal compression.

For some tanks, taking C_1 as the maximum amplified ground motion may be overconservative. For very rigid tanks which are anchored, it may be desirable to calculate the fundamental period and use a lower spectral acceleration value.

The period of the first sloshing mode is relatively long and the corresponding value of spectral acceleration falls in the region of maximum spectral velocity or displacement. The formula presented for C_2 is based on a maximum spectral velocity of 1.5 to 2.3 ft/sec and a maximum spectral displacement of 1.1 to 1.65 feet, depending on soil type.

The calculation of C_2 requires the determination of the natural period of the first sloshing mode and the site amplification factor, S. The period can be determined from the expression:

$$T = kD^{\frac{1}{2}} \tag{8}$$

where k is obtained from Figure P-4 for various D/H ratios. This comes from the formula:

$$T = \frac{2\pi\sqrt{D}}{\sqrt{3.67g \tanh\left(\frac{3.67}{D/H}\right)}} \tag{9}$$

Substituting g = 32.2 ft/sec^2, k is then:

$$k = \frac{0.578}{\sqrt{\tanh\left(\frac{3.67}{D/H}\right)}} \tag{10}$$

The site amplification factor, S, is determined from Table P-2 and varies from 1.0 for rock-like soils to 1.5 for soft to medium stiff soils. These amplifications factors correspond to those recommended in the Final Review Draft of

Recommended Comprehensive Seismic Design Provisions for Buildings [3] prepared by the Applied Technology Council in a study (Project ATC-3) sponsored by the National Science Foundation and the National Bureau of Standards.

The lateral force coefficients, C_1 and C_2, are applicable for the areas of highest seismicity and for tanks which are not required to be functional for emergency post earthquake operations. The zone coefficient, Z, and the essential facilities factor, I, are included in formula (1) for the design overturning moment to provide an adjustment for tanks located in less seismically active areas and for tanks which are required to be functional for emergency operations after an earthquake. The value of the zone coefficient, Z, is obtained from Table P-1 for the various zones defined in Figure P-1. The values of Z correspond to those specified in the Uniform Building Code. The zone maps are based on peak ground motion acceleration contour maps included in the Final Review Draft of the ATC-3 Project. For the 48 contiguous states, the map used is that where the accelerations are a measure of effective peak velocity so to be appropriate for the long period convective force as well as the short period impulsive force. The ATC-3 map depicts contours of approximately equal seismic risk and is considered to be an improvement over the zone map included in the Uniform Building Code which is based on historic earthquake damage levels. The ATC-3 map is for use in establishing ultimate design loads and the acceleration values depicted should be reduced for application in working stress design procedures. The relationships between the zones shown on the maps included in Appendix P and the contour ranges shown on the ATC-3 maps are as follows:

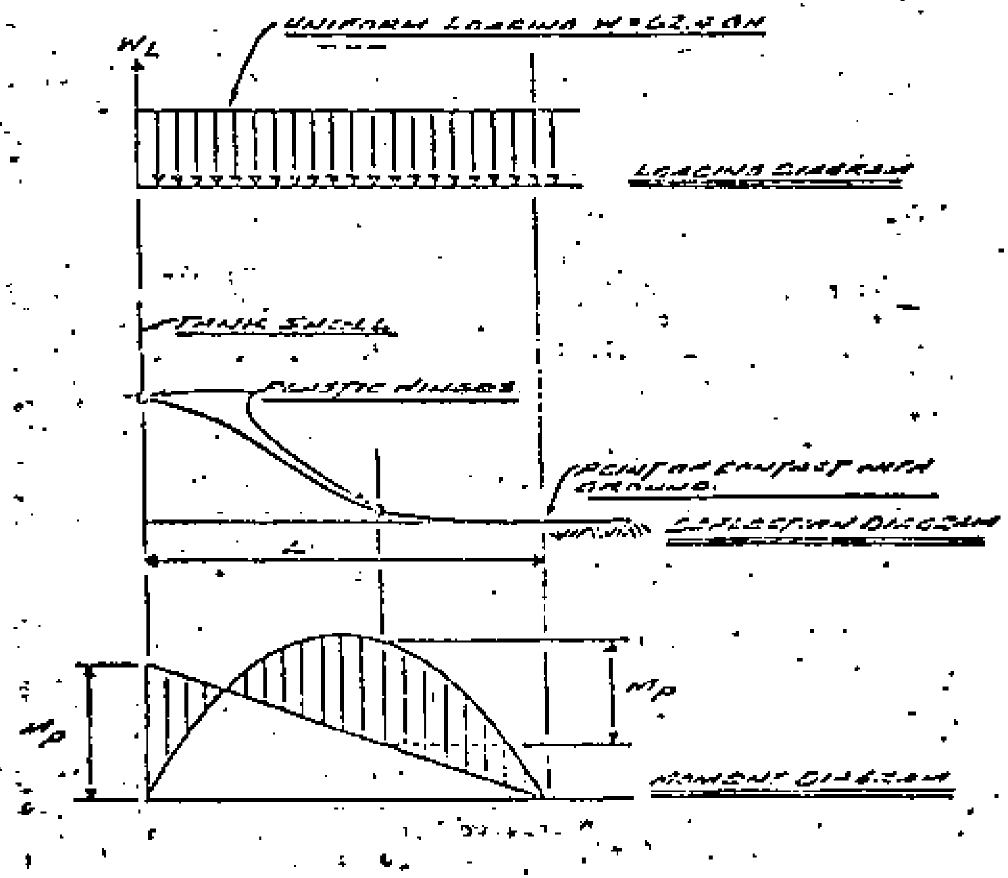
Appendix P Seismic Zone	ATC-3 Map Contour Ranges
4	Over 0.4
3	0.2 to 0.4
2	0.1 to 0.2
1	0.05 to 0.1
0	Under 0.05

The essential facilities factor, I, corresponds to the Occupancy Importance Factor specified in the Uniform Building Code. The UBC requires that "structures or buildings which must be safe and usable for emergency purposes after an earthquake in order to preserve the health and safety of the general public" be designed for a factor of 1.5. For oil storage, this should apply to tanks such as those storing

...el for power generating facilities which are essential for ...ergency postearthquake operations.

...SISTANCE TO OVERTURNING

The factors which may contribute to resistance of the ...erturning moment are noted in paragraph P.4. The weight of ...e contents which may be utilized to resist overturning is ...sed on the calculated reaction at the tank shell of an ...emental strip of the bottom plate perpendicular to the ...ell which can be lifted off the ground. The calculation is ...sed on small deflection theory and assumes the development ... two plastic hinges, one at the junction to the shell and ...e other at some distance inward from the shell. The ...sumed loading, deflection and moment diagram are shown ...low:



The equilibrium solution leads to the following relationships:

$$w_L = 7\sqrt{w M_p} \tag{11}$$

and

$$L = 1.707 \frac{w_L}{w} \tag{12}$$

Substituting $M_p = \frac{F_{by} t_b^2}{4}$ and $w = 62.4GH$

$$w_L = 7.90 t_b \sqrt{F_{by} GH} \tag{13}$$

and

$$L = 0.0274 \frac{w_L}{GH} \tag{14}$$

Practice has been to limit the uplift length, L, to 6 to 7 percent of the tank radius [9]. The limitation of w_L to 1.25 GHD limits L to about 6.8% of the radius. Recent shaking table model tests of tanks [10] show significant changes in the response characteristics which are not accounted for by current design procedures when greater amounts of uplift occur.

The above procedure to establish the maximum resistance of the liquid contents to overturning of the tank is conservative since it does not take into account membrane stresses which will develop in the bottom upon uplift. Further studies need to be undertaken to better determine the uplift resistance and to account for the changes in response when large amounts of uplift occur.

SHELL COMPRESSION

Methods for determining the maximum longitudinal compression force, b, at the bottom of the tank shell are

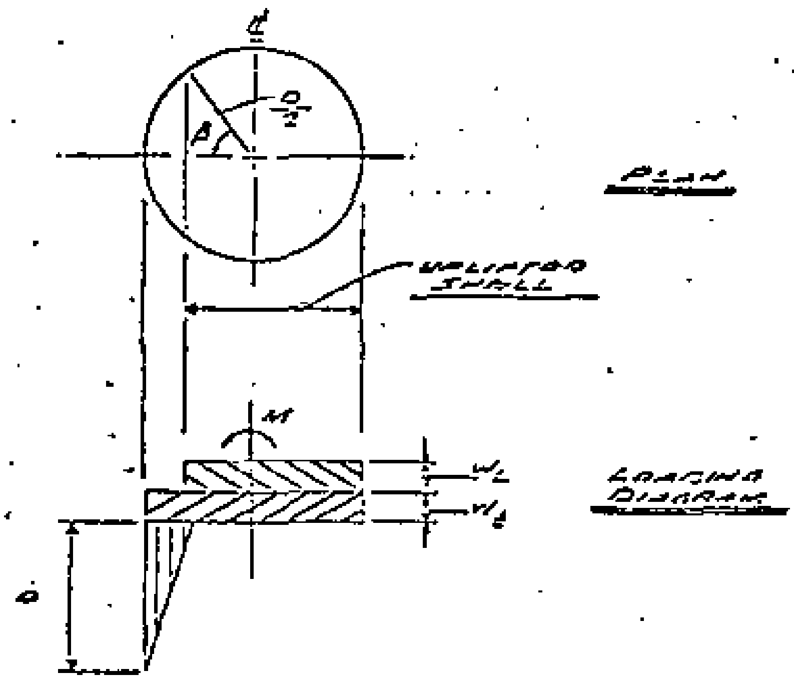
12

given in Paragraphs P.5.1 and P.5.2. For unanchored tanks where there is no uplift and for anchored tanks the compression force can be determined readily from the formula:

$$b = w_1 + \frac{4M}{\pi D^2} = w_1 + \frac{1.273M}{D^2} \qquad (15)$$

which assumes that the force varies directly with the distance from the centerline of the tank in the direction of the lateral loading. For tanks which experience uplift, b can be determined from the value of the compressive force parameter obtained from Figure P-5 as a function of the overturning moment parameter.

The curve in Figure P-5 is derived from the following assumed load distribution around the shell of the tank:



From the summation of vertical forces and overturning moments, the following expressions are obtained:

$$\frac{b + w_L}{w_1 + w_L} = \frac{\pi(1 - \cos\beta)}{\sin\beta - \beta\cos\beta} \qquad (16)$$

$$\frac{M}{D^2(w_1 + w_L)} = \frac{\pi}{8} \cdot \frac{2\beta - \sin 2\beta}{\sin 2\beta - \beta\cos\beta} \qquad (17)$$

By substituting values of β from 0 to π radians in these expressions, the relationship between the two parameters on the lefthand side of the expressions is obtained. This relationship applies for values of the moment parameter from $\pi/4$ where uplift commences to $\pi/2$ where the shell becomes unstable. The relationship may be approximated with good accuracy up to a value of the moment parameter of 1.54 with the following formula:

$$\frac{b + w_L}{w_t + w_L} = \frac{1}{0.8762 - 0.15567\left[\frac{M}{D^2(w_t + w_L)}\right]^{2.2}} \qquad (18)$$

Formulas for determining the maximum allowable longitudinal compression in the shell are given in paragraph P.5.1. These were established to provide a safety factor against buckling of about 1.5. Excluding the effect of internal pressure, the critical stress for very thin wall shells was established as 0.125 Et/R where E is the modulus of elasticity, t the shell thickness, and R the shell radius. This is based on the work of C. D. Miller [11] of Chicago Bridge & Iron Company. Using E = 29,000,000 psi and a safety factor of about 1.5, this leads to an allowable stress of:

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$$\frac{400{,}000t}{D} \quad (19)$$

where t is in inches and D is the diameter in feet.

Lo, Crate and Schwartz (12) determined through theoretical analysis and experimental tests that the critical buckling stress in compression for thin wall cylinders increases with internal pressure. Theoretically, with sufficient internal pressure the critical buckling stress will reach the classical limit of 0.6 Et/R. However, only limited tests have been made to date. The tests made by Lo, Crate and Schwartz showed a doubling of the critical buckling stress as the nondimensional parameter $\frac{P}{E}\left(\frac{P}{t}\right)^2$ increased from zero (no internal pressure) to a value of 0.1028. This value is reached when the value of GHD^2/t^2 is about 300,000. Based on this, the allowable longitudinal compressive stress for thin wall tanks for values of GHD^2/t^2 greater than 200,000 was established as:

$$F_a = \frac{800{,}000t}{D} \quad (20)$$

The tests of Lo, Crate and Schwartz showed a nearly linear increase in critical buckling stress with internal pressure up to the limits of their tests. Thus, for values of GHD^2/t^2 less than 200,000 the allowable compressive stress was established as:

$$F_a = \frac{400{,}000t}{D} + \frac{2GHD}{t} \quad (21)$$

Formula (21) will normally apply only to very small tanks where the shell thickness is established by minimum values rather than by hoop stress.

As the thickness of the shell in proportion to the diameter of the tank becomes relatively large, formulas (20)

and (21) are no longer applicable. Miller presents formulas for critical buckling of shells without internal pressure for intermediate and thick shells leading to a maximum value of critical buckling stress equal to the yield stress. The limit of $F_a = 0.5\ F_{ty}$ is established in Appendix P to maintain an adequate safety factor throughout the intermediate range of thickness to diameter ratio. This limit will normally apply only to small diameter tanks (under 15 ft diameter)

The longitudinal compressive buckling stress of tank shells is a subject which needs further study. The presence of internal pressure and the radial restrain provided by the bottom leads to a different form of buckling than has been experienced in most experimental work on the buckling of cylinders under axial and bending loads.

ANCHORAGE OF TANKS

Generally, tanks need not be anchored when the required resistance to overturning can be provided by the tank shell and internal contents without exceeding the maximum value permitted for w_L. When anchorage is required, careful attention should be given to the attachment of the anchors to the shell to avoid the possibility of tearing the shell. The specified anchorage resistance given in paragraph P.6 for anchored tanks provides a factor of safety in that the resistance provided by the weight of the tank shell is not considered.

SLOSHING WAVE HEIGHT

In some cases it may be desirable to provide freeboard in the tank above the maximum filling height to minimize or avoid overflow and damage to the roof and upper shell due to sloshing of the liquid contents. The height of the sloshing wave may be determined from the following formula based on Housner's corrected version of TID 7024:

$$d = 1.124ZIC_2T^2 \tanh\left(4.77\sqrt{\frac{H}{D}}\right) \quad (22)$$

ROOF SUPPORTING COLUMNS

When it is desired to design roof supporting columns to resist the forces caused by the sloshing of the liquid contents, these forces may be determined as described in Appendix 2 of this paper.

HOOP TENSION

When it is desired to analyze the tank shell for increased hoop tension due to earthquake ground motion, the increased hoop tension P_E per inch of shell height can be obtained from the following expression:

$$P_E = P_1 + P_2 \qquad (23)$$

where P_1 is the tension due to the impulsive force and P_2 is that due to the convective force.

For tanks where D/H is greather than 1.333, P_1 may be determined from the following formula:

$$P_1 = 4.5\, ZIC_1\, GDH \left[\frac{Y}{H} - \frac{1}{2} \left(\frac{Y}{H} \right)^2 \right] \tanh \left(0.866\, \frac{D}{H} \right) \qquad (24)$$

where Y is the distance in feet from the liquid surface to the point under consideration. As can be seen, P_1 is zero at the surface and maximum at the bottom (Y = H). Where D/H is less than 1.333, P_1 may be determined as follows:

$Y < 0.75D$:

$$P_1 = 2.77\, ZIC_1\, GD^2 \left[\frac{Y}{0.75D} - \frac{1}{2} \left(\frac{Y}{0.75D} \right)^2 \right] \qquad (25a)$$

$Y > 0.75D$:

$$P_1 = 1.386\, ZIC_1\, GD^2 \qquad (25b)$$

The convective hoop tension, P_2 may be determined from the following formula:

$$P_2 = 0.9752 I C_2 G D^2 \frac{\cosh\left(3.68 \frac{H-Y}{D}\right)}{\cosh\left(3.68 \frac{H}{D}\right)} \quad (26)$$

The increased hoop tension due to earthquake ground motion should be added to the hoop tension due to hydrostatic pressure. The hydrodynamic portion of the stress, P_E, should be divided by a ductility factor of 2.0 for application in the design at normal allowable design tensile stresses.

CONCLUSION

The basis has been presented for the seismic design provisions for oil storage tanks which have been proposed as an appendix (Appendix P) to API Standard 650. The formulas have been given for the curves included in the proposed revision to facilitate design calculations. Supplemental information has been presented to determine the sloshing wave height, the forces on roof supporting columns caused by sloshing and the increased hoop tension due to earthquake ground motion for use when it is desired to take these factors into consideration in the seismic design.

It has been seen that the design provisions are based on the simplified procedure developed by Housner for rigid tanks except that the maximum ground acceleration is replaced with the spectral value of the pseudo-acceleration corresponding to the fundamental natural frequency of the tank-fluid system as suggested by Veletsos. Provisions are included to insure stability of the tank shell against overturning and to preclude buckling of the tank shell due to longitudinal compression; however, further study of these effects are recommended.

NOMENCLATURE

- b = maximum longitudinal shell compression force, lbs/ft shell circumference
- C_1, C_2 = lateral earthquake coefficients for impulsive an[d] convective forces, respectively
- d = height of sloshing wave above mean depth, ft
- D = tank diameter, ft
- E = modulus of elasticity, psi
- F_a = maximum allowable longitudinal compressive stress in tank shell, psi
- F_{by} and F_{ty} = minimum specified yield strength of bottom annular ring and tank shell, respectively, psi
- g = acceleration due to gravity = 32.2 ft/sec^2
- G = specific gravity (1.0 for water)
- H = maximum filling height of tank, ft
- H_t = total height of tank shell, ft
- I = essential facilities factor
- k = parameter for calculating T, (sec^2/ft)$^{1/2}$
- L = bottom uplift length, ft
- M = overturning moment applied to bottom of tank shell, ft-lbs
- M_p = plastic bending moment in bottom annular ring, in.-lbs/in.
- p = internal pressure, psi
- P_1, P_2, and P_E = increased hoop tension in tank shell due [to] impulsive, convective, and total earthqua[ke] force, respectively, lbs/in.
- R = tank radius, in.
- S = site amplification factor

t = thickness of cylindrical shell, in. When used in formulas for tank design applies to thickness of bottom shell course excluding corrosion allowance.

t_b = thickness of bottom annular ring, in.

T = sloshing wave period, sec

w = unit weight on tank bottom, lbs/sq ft

w_L = maximum weight of tank contents which may be utilized to resist shell overturning moment, lbs/ft of shell circumference.

w_t = weight of tank shell, lbs/ft of shell circumference

W_r = total weight of tank roof plus portion of snow load, if any, lbs

W_s = total weight of tank shell, lbs

W_T = total weight of tank contents, lbs

W_1 and W_2 = weight of effective masses of tank contents for determining impulsive and convective lateral earthquake forces, lbs

X_s = height from bottom of tank shell to center of gravity of shell, ft

X_1 and X_2 = height from bottom of tank shell to centroids of impulsive and convective lateral earthquake forces, respectively, for computing M, ft

X_1' and X_2' = height from bottom of tank shell to centroids of impulsive and convective lateral earthquake forces, respectively, for computing total overturning moment on foundation, ft

y = vertical distance from liquid surface to point on shell being analyzed for hoop tension, ft

Z = seismic zone coefficient

= central angle between axis of tank in the direction of earthquake ground motion and point on circumference where shell uplift commences, radians.

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APPENDIX 1
PROPOSED APPENDIX P TO API STANDARD 650

SEISMIC DESIGN OF STORAGE TANKS

P.1 SCOPE

This appendix establishes recommended minimum basic requirements for the design of storage tanks subjected to seismic load as specified by purchaser. These requirements represent accepted practice for application to flat bottom tanks. However, it is recognized that other procedures and applicable factors or additional requirements may be specified by the purchaser or jurisdictional authorities. Any deviation from the requirements herein must be by agreement between purchaser and manufacturer.

P.2 INTRODUCTION

The design procedure considers two response modes of the tank and its contents: (1) the relatively high frequency amplified response to lateral ground motion of the tank shell and roof together with a portion of the liquid contents which moves in unison with the shell, and (2) the relatively low frequency amplified response of a portion of the liquid contents in the fundamental sloshing mode. The design requires the determination of the hydrodynamic mass associated with each mode and the lateral force and overturning moment applied to the shell resulting from the response of the masses to lateral ground motion. Provisions are included to assure stability of the tank shell against overturning and to preclude buckling of the tank shell due to longitudinal compression.

No provisions are included regarding the increase in hoop tension due to seismic forces since this does not affect shell thickness for the lateral force coefficients specified herein taking into account generally accepted increased allowable stress and ductility ratios.

P.3 DESIGN LOADING

P.3.1 Overturning Moment

The overturning moment due to seismic forces applied to the bottom of the shell shall be determined as follows:

$$M = ZI(C_1W_sX_s + C_1W_rH_t + C_1W_1X_1 + C_2W_2X_2)$$

M = Overturning moment in foot pounds applied to bottom of tank shell.

Z = Zone coefficient from Figure P-1 and Table P-1.

I = Essential facilities factor. $I = 1.5$ for tanks which must be functional for emergency post earthquake operations and 1.0 for all other tanks.

C_1 and C_2 = Lateral earthquake force coefficients determined per paragraph P.3.3.

W_s = Total weight in pounds of tank shell.

X_s = Height in feet from bottom of tank shell to center of gravity of shell.

W_r = Total weight in pounds of tank roof plus portion of snow load, if any, as specified by purchaser.

H_t = Total height in feet of tank shell.

W_1 = Weight in pounds of effective mass of tank contents which moves in unison with tank shell, determined per paragraph P.3.2(a).

X_1 = Height in feet from bottom of tank shell to centroid of lateral seismic force applied to W_1, determined per paragraph P.3.2(b).

W_2 = Weight in pounds of effective mass of first mode sloshing contents of tank, determined per paragraph P.3.2(a).

X_2 = Height in feet from bottom of tank shell to centroid of lateral seismic force applied to W_2, determined per paragraph P.3.2(b).

Note: The overturning moment determined per this paragraph is that applied to the bottom of the shell only. The tank foundation is subjected to an additional overturning moment due to lateral displacement of the tank contents which may need to be considered in the design of some foundations such as pile supported concrete mats.

P.3.2 Effective Mass of Tank Contents

a. The effective mass W_1, and the effective mass W_2, may be determined by multiplying W_T, by the ratios W_1/W_T and W_2/W_T, respectively, obtained from Figure P-2 for the ratio D/H.

Where:

W_T = Total weight in pounds of tank contents (product specific gravity specified by purchaser).

D = Tank diameter in feet.

H = Maximum filling height of tank in feet from bottom of shell to top of top angle or overflow which limits filling height.

b. The heights from the bottom of the tank shell to the centroids of the lateral seismic forces applied to W_1 and W_2, X_1 and X_2, may be determined by mutiplying H, by the ratios X_1/H and X_2/H, respectively, obtained from Figure P-3 for the ratio of D/H.

c. The curves in Figures P-2 and P-3 are based on a modification of the equations presented in ERDA Technical Information Document 7024*. Alternatively, W_1, W_2, X_1 and X_2 may be determined by other analytical procedures based on the dynamic characteristics of the tank.

P.3.3 Lateral Force Coefficients

a. The lateral force coefficient C_1 shall be taken as 0.24.

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* Technical Information Document 7024, Nuclear Reactors and Earthquakes, prepared by Lockheed Aircraft Corporation, and Holmes & Narver, Inc., for the U.S. Atomic Energy Commission, August 1963.

b. The lateral force coefficient C_2 shall be determined as a function of the natural period of the first mode sloshing, T, and the soil conditions at the tank site.

When T is less than 4.5:

$$C_2 = \frac{0.30\ S}{T}$$

When T is greater than 4.5:

$$C_2 = \frac{1.35\ S}{T^2}$$

Where:

S = Site amplification factor from Table P-2.

T = Natural period in seconds of first mode sloshing. T may be determined from the following expression:

$$T = kD^{\frac{1}{2}}$$

k = Factor obtained from Figure P-4 for the ratio D/H.

c. Alternatively, C_1 and C_2 may be determined from response spectra established for the specific site of the tank and for the dynamic characteristics of the tank. The spectrum for C_1 should be established for a damping coefficient of 2% of critical and scaled to a maximum amplified acceleration of 0.24 times the acceleration of gravity. The spectrum for C_2 should correspond to the spectrum for C_1 except modified for a damping coefficient of 0.5% of critical.

RESISTANCE TO OVERTURNING

Resistance to the overturning moment at the bottom of the shell may be provided by the weight of the tank shell and by the weight of a portion of the tank contents adjacent to the shell for unanchored tanks or by anchorage of the tank shell. For unanchored tanks, the portion of the contents which may be utilized to resist overturning is

dependent on the width of the bottom annular ring which lifts off the foundation and may be determined as follows:

$$w_L = 7.9 t_b \sqrt{F_{by} GH}$$

except that w_L shall not exceed 1.25GHD.

Where:

w_L = Maximum weight of tank contents in pounds per foot of shell circumference which may be utilized to resist the shell overturning moment.

t_b = Thickness of bottom annular ring in inches.

F_{by} = Minimum specified yield strength in pounds per square inch of bottom annular ring.

G = Design specific gravity of contents as specified by purchaser.

b. The thickness of the bottom annular ring, t_b, shall not exceed the thickness of the bottom shell course, or ¼ inch, whichever is greater. Where the bottom annular ring is thicker than the remainder of the bottom, the width of the annular ring in feet shall be equal to or greater than:

$$0.0274 \frac{w_L}{GH}$$

P.5 SHELL COMPRESSION

P.5.1 Unanchored Tanks

The maximum longitudinal compression force at the bottom of the shell may be determined as follows:

When $\frac{M}{D^2(w_t+w_L)}$ is equal to or less than 0.785:

$$b = w_t + \frac{1.273\ M}{D^2}$$

When $\frac{M}{D^2(w_t+w_L)}$ is greater than 0.785:

b may be computed from the value of the parameter $\frac{b+w_L}{w_t+w_L}$ obtained from Figure P-5.

Where:

b = Maximum longitudinal shell compression force in pounds per foot of shell circumference.

w_t = Weight of tank shell in pounds per foot of shell circumference.

2 Anchored Tanks:

The maximum longitudinal compression force at the bottom of the shell may be determined as follows:

$$b = w_t + \frac{1.273\ M}{D^2}$$

3 Maximum Allowable Shell Compression

The maximum longitudinal compressive stress in the shell, $\frac{b}{12\ t}$, shall not exceed the maximum allowable stress, F_a, determined as follows:

When the value of $\frac{GHD^2}{t^2}$ is greater than 200,000:

$$F_a = \frac{800,000\ t}{D}$$

When the value of $\frac{GHD^2}{t^2}$ is less than 200,000:

$$F_a = \frac{400,000\ t}{D} + \frac{2\ GHD}{t}$$

Except that in no case shall the value of F_a exceed 0.5 F_{ty}.

Where:

t = Thickness in inches, excluding corrosion allowance, of the bottom shell course.

F_a = Maximum allowable longitudinal compressive stress in the shell in pounds per square inch. The above formulas for F_a take into account the effect of internal pressure due to the liquid contents.

F_{ty} = Minimum specified yield strength of the shell in pounds per square inch.

P.6 ANCHORAGE OF TANKS

Anchorage of tanks shall be designed to provide a minimum anchorage resistance in pounds per foot of shell circumference of:

$$\frac{1.273\ M}{D^2}$$

The stresses due to anchor forces in the tank shell at the points of attachment of the anchors shall be investigated.

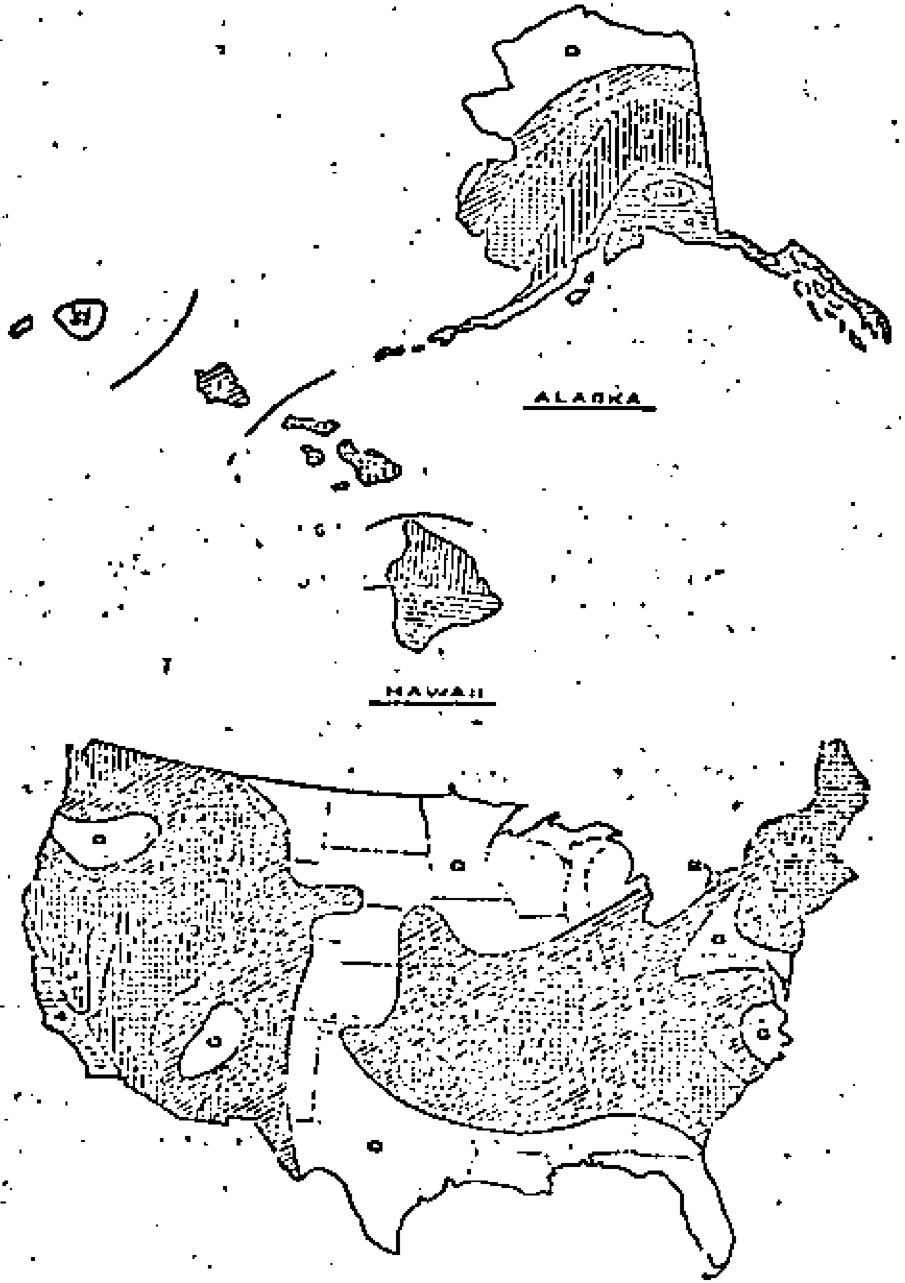
P.7 PIPING

Provisions for suitable flexibility in all piping attached to the shell or bottom of the tank shall be considered. On unanchored tanks subject to bottom uplift, piping connected to the bottom shall be free to lift with the bottom or shall be located so that the horizontal distance measured from the shell to the edge of the connecting reinforcement shall be the width of the bottom hold down as calculated in paragraph P.4(b) plus 12 inches.

P.8 ADDITIONAL CONSIDERATIONS

a. The purchaser shall specify any freeboard desired to minimize or avoid overflow and damage to the roof and upper shell due to sloshing of the liquid contents.

b. The base of the roof supporting columns shall be restrained to prevent lateral movement during earthquakes. When specified by the purchaser, the columns shall be designed to resist the forces caused by the sloshing of the liquid contents.



SEISMIC ZONE MAP

FIGURE P-1

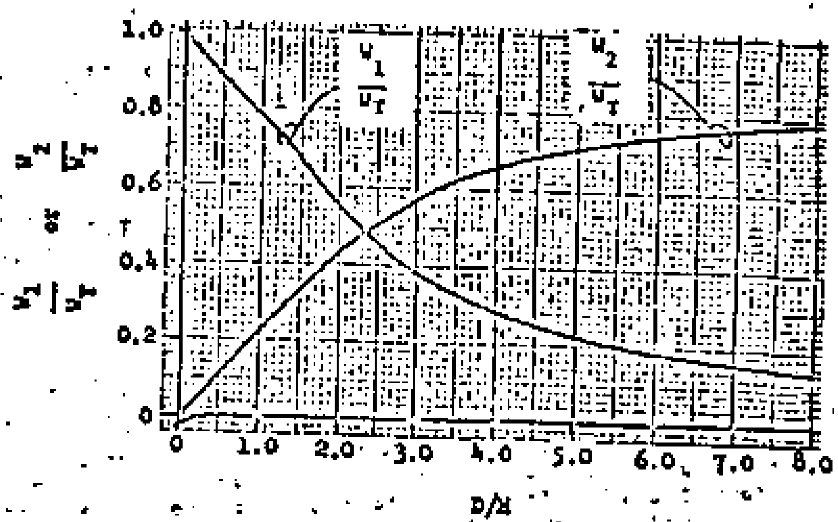


FIGURE P-2

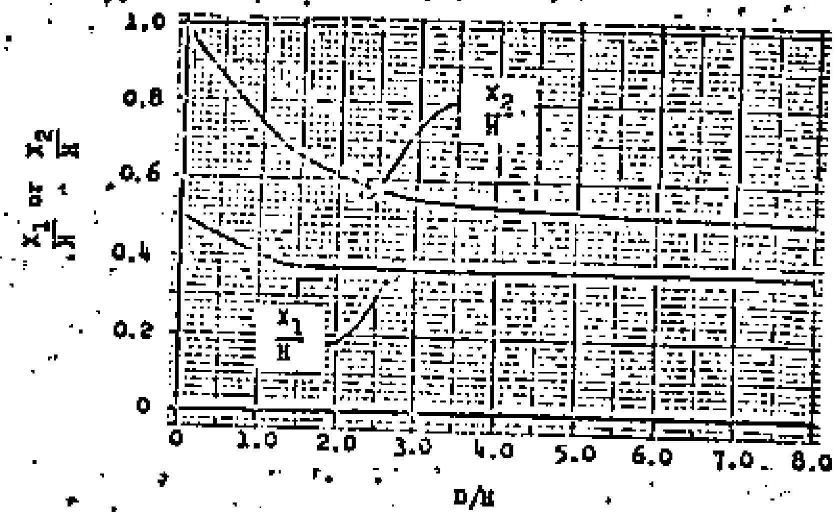


FIGURE P-3

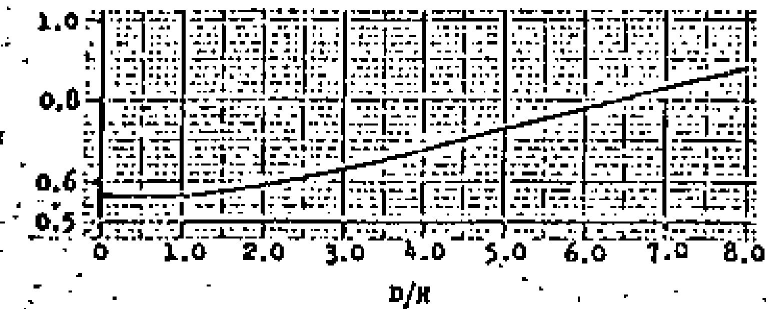


FIGURE P-4

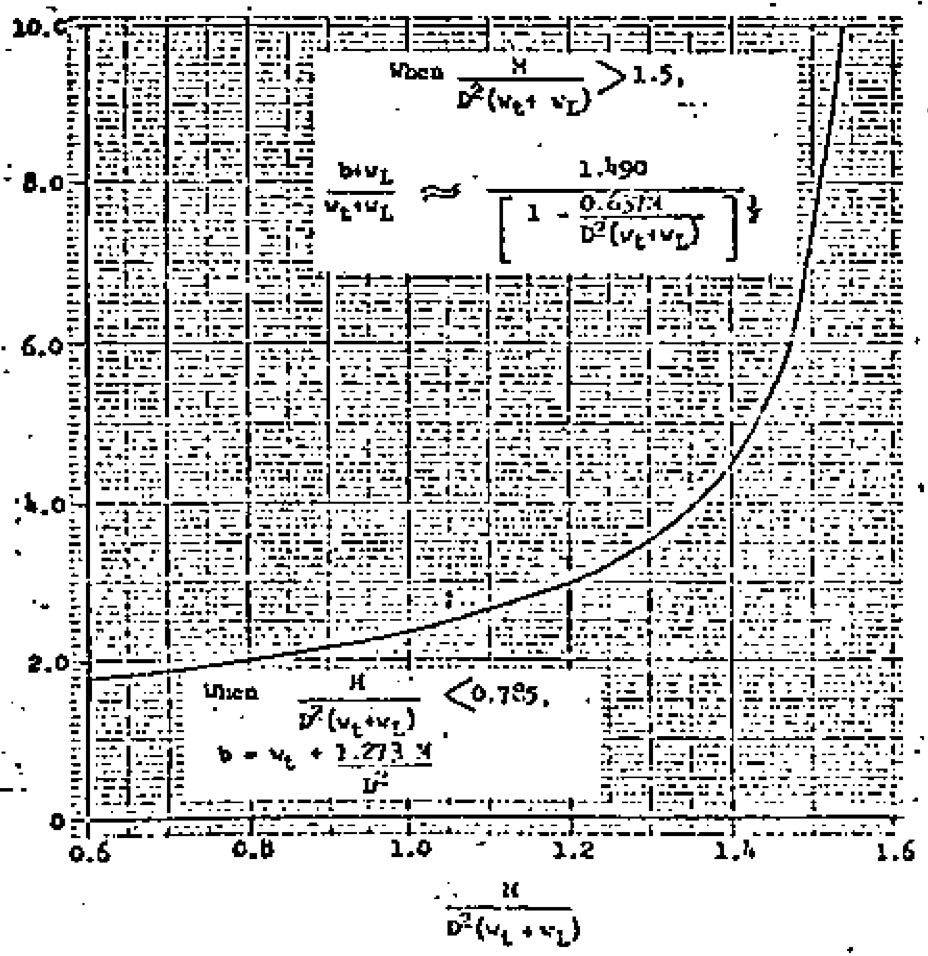


FIGURE P-5

ZONE COEFFICIENT				
	Seismic Zone Per Figure P-1			
	1	2	3	4
Z	0.1875	0.375	0.75	1.0

No earthquake design required for Zone 0.

TABLE P-1

SOIL PROFILE COEFFICIENT			
	Soil Profile Type		
	A	B	C
S	1.0	1.2	1.5

SOIL PROFILE TYPE A is a profile with:

1. Rock of any characteristic, either shale-like or crystalline in nature. Such material may be characteriz by a shear wave velocity greater than 2,500 feet per second, or
2. Stiff soil conditions where the soil depth is less than 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

SOIL PROFILE TYPE B is a profile with deep cohesionless or stiff clay conditions, including sites where the soil depth exceeds 200 feet and the soil types overlying rock are stabl deposits of sands, gravels, or stiff clays.

SOIL PROFILE TYPE C is a profile with soft-to-medium-stiff clays and sands, characterized by 30 feet or more of soft-to-medium-stiff clay with or without intervening layers of sand or other cohesionless soils.

In locations where the soil profile type is not known in sufficient detail to determine the soil profile type, Soil Profile C shall be assumed.

TABLE P-2

APPENDIX 2

HORIZONTAL FORCES ON COLUMNS CAUSED BY SLOSHING OF FLUID IN CYLINDRICAL TANKS

The following presentation is considered a reasonable approximation for the determination of seismic induced loads on columns.

The total horizontal force acting per foot of column length includes the drag force, inertial force, acceleration force of the column mass, and the acceleration force of an effective column of water. The acceleration force of the column and its effective water mass are functions of the seismic factor. The drag and inertial forces are functions of the fluid velocity, u, and acceleration, $\dot{u}$:

$$u = \frac{2dgT}{D} \; \frac{\cosh \pi\left(\frac{H-Y}{D}\right) \cos 2\pi \frac{t}{T} \cos \pi \frac{X}{D}}{\cosh \pi \frac{H}{D}} \qquad (1)$$

$$\dot{u} = \frac{4\pi dg}{D} \; \frac{\cosh \pi\left(\frac{H-Y}{D}\right) \sin 2\pi \frac{t}{T} \cos \pi \frac{X}{D}}{\cosh \pi \frac{H}{D}} \qquad (2)$$

The average force per foot of column is:

$$\overline{FT} = \frac{1}{H}\int_0^H dF_d + \frac{1}{H}\int_0^H dF_i + ZIC_I\left(m_c + m_w\right)$$

where: $F_d = C_D \rho D_c \frac{u|u|}{2}$

$$F_i = C_M \rho \frac{\pi D_c^2}{4} \dot{u}$$

Equation (3) may be applied to circular and rectangular shaped interior columns. For circular columns, D_c, is the maximum dimension of the member cross-section as shown in Figure 1. The analysis for rectangular columns is based on an equivalent circular column with diameter D_c. The drag factor is corrected for rectangular column to account for the additional resistance to flow.

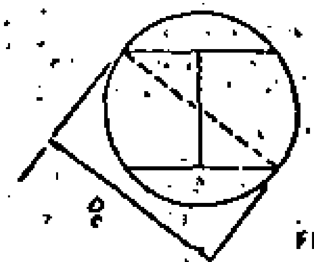


FIG 1

Substituting Equation (1) and (2) into Equation (3) and integrating yields the following average force per foot of column:

$$\overline{FT} = \frac{C_D \rho D_c (dgT)^2}{2\pi HD} \; \frac{\cos 2\pi \frac{t}{T} \cos \pi \frac{X}{D} \left|\cos 2\pi \frac{t}{T} \cos \pi \frac{X}{D}\right|}{\cosh^2\left(\pi \frac{H}{D}\right)} \left(\sinh 2\pi \frac{H}{D} + 2\pi \frac{H}{D}\right)$$

$$+ \frac{\pi C_M \rho D_c^2 dg}{H} \; \frac{\sin 2\pi \frac{t}{T} \cos \pi \frac{X}{D} \sinh \pi \frac{H}{D}}{\cosh \pi \frac{H}{D}} + ZIC_I\left(m_c + m_w\right) \qquad (4)$$

The solution of Equation (4) is a function of time and location of the column in the tank. An iteration with time over the period of the sloshing wave is necessary to search out the maximum column load.

For simplicity, the design of the column for combined beam-column action is made assuming the seismic load acts uniformly over the full height of the column rather than the fluid height. It is recommended that AISC primary column allowables be used in the beam-column design since secondary column allowables have safety factors too low to allow an additional increase for the seismic load.

NOMENCLATURE FOR APPENDIX 3

The following defines terms used in Appendix 3 only. For other terms, see the Nomenclature following the main body of the paper.

C_D = drag coefficient. A value of 1.0 is recommended for round columns and 1.6 for wide flange structural shapes.

C_M = mass coefficient. A value of 2.0 is recommended.

D_c = maximum cross-section dimension of column, ft.

F_d and F_i = drag and inertia force on column, lb/ft.

ΣF = average total force on column, lb/ft.

m_c = column weight, lb/ft.

m_w = weight of effective column of water, lb/ft. $m_w = \frac{\pi D^2}{4} \rho g$

u = fluid particle velocity, ft/sec.

$\dot{u}$ = fluid particle acceleration, ft/sec.^{2}

x = horizontal distance in direction of earthquake force from center of tank to center of column, ft.

ρ = mass of fluid, lb-sec^2/ft.^{4}

τ = time from beginning of wave cycle, sec. τ varies from 0 to T.

DIVISION DE EDUCACION CONTINUA
FACULTAD DE INGENIERIA U.N.A.M.

IX CURSO INTERNACIONAL DE INGENIERIA SISMICA

DISEÑO SISMICO DE ESTRUCTURAS ESPECIALES

T U B E R I A S

Dr. Francisco C. Aguilar López de Nava
Julio, 1983

METODOS ANALITICOS

En la Tabla 1, se muestran las cargas , propiedades de los materiales y tipo de análisis estructural que es necesario considerar en el diseño de tubería y sus componentes para fallas de formaciones y cargas sobre el equipo que interconectan.

Los objetivos de llevar a cabo el análisis de flexibilidad de un Sistema de tubería son :

1. Verificar que ninguna de las componentes del sistema esté sobre esforzada en ninguna de las condiciones de carga que es posible esperar durante la vida útil del sistema.
2. Revisar que los elementos mecánicos, impuestos por la expansión de la tubería a las boquillas del equipo interconectado no sean mayores a las permisibles.

Las cargas que normalmente se consideran son :

a) Peso propio de las componentes, líquido que conduce y aislamiento.
b) Cambios de longitud de la tubería, debidas a cambios de temperatura.
c) Movimientos del equipo interconectado.
d) Viento y/o sísmo.
e) Excitaciones inducidas por el equipo interconectado.

El análisis de un sistema de tubería depende tanto de su trazo geométrico, como del tipo y localización de los soportes; sin embargo, a su vez la información para el diseño de estos elementos se obtiene como resultado del análisis, por lo que en general el procedimiento es de tipo interactivo.

Tradicionalmente el análisis estructural de sistemas de tuberías se ha denominado " Análisis de Flexibilidad de Tuberías ", sin embargo, debe hacerse notar que ésto se debe a que originalmente solo se utilizó el método de las flexibilidades por este tipo de cálculos; mientras que en la actualidad el método de las rigideces es la más popular.

En la referencia de J.E. Brock,se encuentra una revisión amplia sobre el análisis de sistema de tubería, incluyendo 265 referencias al respecto. En la misma referencia se presenta la historia del desarrollo de los programas de computadora en existencia, hasta 1966.

Hasta 1970, la mayoría de este tipo de análisis se hacía en comportamiento elástico lineal, considerando al sistema de tubería como un conjunto de vigas rectas y curvas, incluyendo factores de intensificación de esfuerzos y flexibilidad para los tramos curvos.

Por lo general, no se consideraba factores de incremento de flexibilidad de otras componentes que en algunos casos pueden alterar las deformaciones de todo el sistema.

La solución analítica del problema general del análisis estructural en tres dimensiones, considerando restricciones intermedias impuestas por los diferentes tipos de apoyos, aunque básicamente es sencilla, involucra una gran cantidad de cálculos, efectuados de acuerdo a una secuela cuidadosa, aún para configuraciones simples.

Esto motivó que se desarrollaran diversos métodos simplificados para hacer práctico el análisis de tuberías. El uso extensivo de las computadoras digitales y el desarrollo explosivo de los métodos matriciales, ha venido a facilitar el análisis elástico-lineal de sistemas de tuberías, eliminando la necesidad de soluciones simplificadas.

En la actualidad, existen principalmente en Estados Unidos una multitud de programas para este fin, variando en detalles menores, tales como : máximo número de ramales, número de " loops ", tipo de restricciones intermedias, etc.

Uno de los más ampliamente usados es el desarrollado por la Marina de los Estados Unidos, que se designa como M.E.C./21. El tamaño máximo de problema que puede manejar es de 99 ramales y/o 999 puntos nodales.

El tiempo de máquina empleado por elemento en una computadora IBM-7094, es de 0.05 minutos. El reporte de Griffin describe la aplicación del programa y sirve como manual del usuario. Este programa es manejado por Los Alamos Cientific Laboratory, Los Alamos New Mexico.

Otro programa bastante utilizado es el PIPE que distribuye Argonne National Laboratory, cuyas limitaciones son 100 nodos, 20 " loops ", 25 cargas externas y 10 conjuntos de propiedades de material.

El " Service Bureau Corporation " es otro organismo que proporciona servicio de análisis de flexibilidad de tuberías, el programa que ofrecen tiene la ventaja de permitir una codificación sencilla, aún para configuraciones complejas de tubería.

La mayor parte de las compañías de Ingeniería Norteamericanas, tales como Bechtel Co., C.F. Braun Co., Electric Boat Division of General Dynamics, ESSO Research and Engineering, Fluor Corporation, M.W. Dellog Co., Arthur D. Little, etc.; que se dedican a realizar ingeniería de proyecto, tiene sus propios programas de computadora que, por lo general, utilizan únicamente en forma interna.

TABLA I

Factores involucrados en el diseño de componentes de tubería

Requerimientos de Diseño - Evitar Fallas por:

A) Ruptura debida a:

1. Carga única de corto tiempo (incluyendo fallas frágil)
2. Cargas repetidas (fatiga)
3. Carga prolongada a alta temperatura (ruptura por creep)
4. Combinaciones de las cargas anteriores

B) Deformación excesiva que conduzca a:

1. Fugas en asientos de válvulas
2. Atascamiento de mecanismos de válvulas
3. Fugas en juntas bridadas

C) Cargas excesivas en equipo conectado que produzcan:

1. Ruptura del equipo
2. Sobrecarga en chumaceras de equipo rotatorio
3. Desalineamiento y modificaciones a los claros libres de partes rotatorias con posible daño a éstas.

Cargas

1. Presiones internas (operación y prueba
2. Fuerzas de expansión térmica
3. Peso propio y del fluído
4. Gradientes térmicos
5. Vibración forzada (viento, sismo o equipo rotatorio)
6. Cargas en juntas bridadas
7. Cargas concentradas (válvulas)
8. Golpe de ariete

Cont. Tabla I

Propiedades de material

1. Módulo de elasticidad
2. Relación de Poisson
3. Resistencia última
4. Esfuerzo de fluencia
5. Esfuerzo de "creep"
6. Resistencia por fatiga
7. Ductilidad
8. Resistencia a la ruptura bajo cargas de larga duración

EFECTOS DINAMICOS EN SISTEMAS DE TUBERIA

Introducción

La intención de este capítulo es presentar un resumen de las bases de la teoría de vibraciones aplicables a sistemas de tubería para auxiliar al diseñador a lograr prácticas de diseño que minimizen la aparición de vibraciones objetables o dañinas en condiciones de operación.

Los efectos dañinos de las vibraciones normalmente no son interpretados adecuadamente, ya que han ocurrido fallas debidas a vibración que se han atribuído a otras causas; mientras que por el contrario, oscilaciones de amplitud perceptible, pero no dañinas, han dado lugar a alarmas excesivas.

Entre los efectos indeseables que debe considerar el diseñador de tubería están:

a) Las pulsaciones de flujo que pueden producir una operación ruidosa y una turbulencia excesiva que a su vez genere mayor transferencia de calor.
b) Daño o fuga de juntas críticas y sellos
c) Efectos perjudiciales en equipo interconectado
d) Corrosión, erosión
e) Efectos psicológicos en las personas
f) Falla por fatiga
g) Propagación de grietas a partir dedefectos en la tubería
h) Transmisión de vibraciones a estructuras de soporte

Se ha publicado relativamente poco sobre vibración de tubería, sin embargo, hay una gama muy amplia de material general sobre vibraciones mecánicas que es directamente aplicable a las oscilaciones estructurales de tubería.

Los libros de texto de S. Timoshenko "Vibration Problems in Engineering" y de J. Den Hartog "Mechanical Vibrations" son los más conocidos por su tratamiento ingenieril de los fundamentos de las vibraciones mecánicas y estructurales.

Definiciones

1. Período de vibración, T, (en segundos) es el tiempo que tarda un sistema en efectuar una oscilación completa.
2. Frecuencia de oscilación, f, (en ciclos por segundo) es igual al recíproco del período de vibración.
3. La frecuencia angular, W, (en radianes por segundo) es la frecuencia en radianes.
4. Grados de libertad es el número de cantidades independientes que definen la posición de un sistema.
5. Modo principal de vibración es la "forma" o configuración que adopta un sistema al vibrar a una frecuencia definida. El número de modos es igual al número de grados de libertad.
6. Frecuencia natural es la frecuencia menor, se conoce como fundamental.
7. Amortiguamiento es una fuerza proporcional a la velocidad de vibración que tiende a reducir las amplitudes de vibración.
8. Resonancia es la amplificación de la amplitud de vibración producida por una coincidencia entre alguna frecuencia natural W, y la frecuencia de excitación externa.
9. Factor de amplificación es la relación entre la máxima amplitud de vibración y la deflección estática y es función del cociente de la frecuencia de excitación y la natural, así como del amortiguamiento.

Fuentes de excitación.

Debe distinguirse cuidadosamente entre los tres tipos de vibración existente:

a) libre
b) forzada
c) autoexcitada.

En vibración libre un sistema vibra sin fuerzas externas, la excitación está proporcionada por condiciones iniciales de desplazamiento y/o velocidad.

En vibración forzada un sistema oscila bajo la acción externa de una fuerza perturbadora periodica. Una fuente primaria de excitación puede ser el desbalanceo de maquinaria rotatoria (motores eléctricos, turbinas, compresores, bombas, etc.)

Otras fuentes de vibraciones forzadas de tubería son la variación periodica de presiones en el fluído o "pulsaciones" y la aceleración de masas en un mecanismo reciprocante.

Las vibraciones autoexcitadas son un fenómeno algo complejo, ya que el sistema vibra aún sin fuerzas externas periodicas y la vibración persiste aún en presencia de amortiguamiento, ya que su orígen proviene de fuentes de energía interna.

En sistemas de tubería, la vibración de este tipo, normalmente se ha encontrado asociada a inestabilidades de flujo que, por lo general, se deben a una mala operación de equipos rotatorios interconectados.

La maquinaria rotatoria constituye la mayor fuente de vibración mecánica, debido al inevitable desbalanceo de masa que existe en las partes rotatorias del equipo, por lo que a menos que el equipo se balancee muy cuidadosamente o se apoye sobre una cimentación provista de aisladores de vibración, cabe esperar la existencia de vibraciones forzadas con frecuencia igual a la de rotación del equipo en la tubería interconectada y las estructuras cercanas.

Si la velocidad de rotación está en la cercanía de alguna frecuencia natural de la tubería, las amplitudes de vibración pueden llegar a ser muy considerables y producir fallas de la tubería o sus componentes, por lo general, a mediano y largo plazo.

Un compresor del tipo reciprocante es una fuente de variación periodica de la presión a una frecuencia igual a la velocidad de rotación, multiplicado por el número de cilindros de acción simple o por el doble del número de cilindros para acción doble.

Si esta frecuencia está cercana a la frecuencia acústica del sis tema de tubería conectado, aparecerán variaciones periodicas grandes de la presión, ésto se conoce como resonancia acústica, la -- cual puede tener efectos adversos sobre la maquinaría y la tubería.

Otra fuente de excitación periodica es la producida por el viento. Si éste incide perpendicular al eje de un cilindro de diámetro D (en pies) a una velocidad constante U (pies/seg), se producen fuerzas periodicas de excitación a una frecuencia f (en ciclos/seg)

$$f = SU/D$$

en donde S es el número de Strouhal, que vale aproximadamente -- 0.18 para un cilindro.

Estas fuerzas aerodinámicas son debidas a los movimientos de los vórtices de Von Karman alrededor del cilindro y actúan a 90° de - la dirección del viento.

Su magnitud, por lo general, es pequeña, pero si alguna frecuencia natural se encuentra en la cercanía de esta frecuencia perturbadora, se puede producir una resonancia de la tubería.

La eliminación de las fuentes de vibración es indudablemente el - método más deseable de solución de un problema de vibración, sin embargo, ésto no siempre es posible en la práctica; por lo que - frecuentemente se recurre únicamente a aislar y a controlar la - vibración.

Debido al número tan grande de variables y condiciones que hay - que tomar en cuenta para determinar un trazo de tubería, no se - recomienda que en todos los casos se efectúe un análisis dinámico detallado.

Sin embargo, se justifica emplear algún tiempo en estimar la frecuencia fundamental de una tubería o tramo de tubería en aquellos casos que la fuerza de excitación es evidente.

Una adecuada selección y espaciamiento de soportes, guías y restricciones puede permitir alejar la frecuencia natural de un sis tema de la frecuencia de excitación, llegándose en casos en que ésto no sea posible a la utilización de dispositivos especiales que amortiguen las vibraciones forzadas. Para predecir la respuesta mecánica de un sistema de tuberías se tiene que en la realidad se presentan algunas condiciones que difieren considerablemente de lo que se supone la teoría, por ejemplo:

a) Los extremos de la tubería no están ni completamente fijos ni simplemente apoyados sino en una condición intermedia.

b) El diámetro de la tubería no es uniforme a lo largo de todo el desarrollo.

c) Por lo general la tubería es continua sobre varios apoyos.

d) Existen masas concentradas que en realidad son distribuidas porque su longitud es considerable.

Sin embargo a pesar de estas limitaciones es necesario tener conocimiento de las características de vibración de vigas de sección uniforme y definidas condiciones de apoyo en sus extremos para utilizar estos resultados como punto de partida.

En la Tabla 1 se muestra las frecuencias naturales para tuberías ideales con varias condiciones de Frontera.

En la Tabla 2 se muestra como corregir esos resultados en el caso de cargas concentradas adicionales.

CONSIDERACIONES SOBRE TECNICAS ANALITICAS Y EXPERIMENTALES

A este respecto mencionaremos que las técnicas analíticas por si solas, aunque representan una herramienta muy valiosa tienen serias limitaciones -

los instrumentos deben estar perfectamente calibrados, esta operación en condiciones de laboratorio es perfectamente lógica y normal, pero cuando los instrumentos tienen que viajar al campo durante su transporte pueden resultar afectados y quedar descalibrados y en campo es difícil calibrar. Por otra parte la temperatura que es el enemigo mayor de las componentes electrónicas no es controlable en el lugar donde se efectuan las mediciones o sea donde ocurren los problemas.

Suponiendo que las dificultades anteriores logren vencerse aún queda el problema de determinar que variebles medir y bajo que condiciones hacerlo, esta consideración es muy importante ya que el análisis de los datos tiene que hacerse en laboratorio y sino se obtuvo toda la información necesaria hay que regresar al campo a obtenerla con los consiguientes retrasos de tiempo, habiendo incluso ocasiones en que no es posible repetir las condiciones en que se efectúo la primera medición con lo que crece la complejidad de la interpretación de resultados.

Es en este punto en el que las técnicas analíticas complementan a las experimentales en el sentido de que si se tiene una idea del comportamiento dinámico de una componente o sistema, las determinaciones experimentales pueden enfocarse hacia la verificación del modelo analítico o bien los puntos de medición pueden determinarse a partir de los resultados analíticos y en general puede concentrarse más el esfuerzo de la obtención de datos hacia los puntos más relevantes

TABLA 1.- FRECUENÇIAS NATURALES DE VIGAS ELASTICAS DE ACERO

tipo de viga	Factor de frecuencia λ_1	λ_2
Cantilivcr	3.52	22.0
Simplemente apoyada	9.87	39.5
empotrada-apoyada	15.8	50.0
empotrada-apoyada	22.4	61.7
libre-libre	22.4	61.7

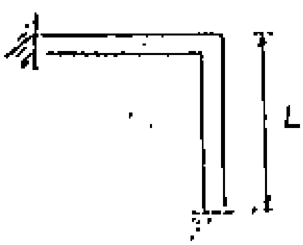
$f_i = 223\ \lambda_i (k/L^2)$

k: radio de giro, pulgs.

L: longitud, pies

$E = 30 \times 10^6$ lb/pulg^2 $\quad \gamma' = 0.283$ lb/ pulg^3

	Fuera del plano	En el plano
"L" de piernas iguales	3.74	15.4
"U" de piernas iguales	2.0	3.1



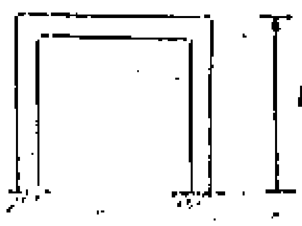
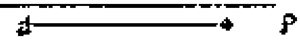
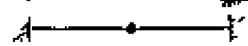


TABLA 2.- CORRECCIONES PARA CALCULO DE FRECUENCIAS NATURALES CUANDO EXISTEN CONCENTRACIONES

Tipo de viga	Factor "C"	Figura
Cantiliver	3.9	
Simplemente apoyada	2.0	
empotrada apoyada	2.3	
empotrada doble	2.7	

$$f_c = f \Big/ \sqrt{1 + c\,\frac{P}{W}}$$

p: carga concentrada, en lb.

W: peso de la viga, en lb.

f: frecuencia de la viga sin carga concentrada, en H_z

CONSIDERACIONES TEORICAS

La mayor parte de los programas de computadora para análisis dinámico de sistemas de tubería, están basados en el método del elemento finito. A continuación se presenta una breve descripción de la teoría en que están basadas este tipo de herramientas.

Las ecuaciones de equilibrio dinámico para el modelo de elemento finito, pueden expresarse en forma matricial como se indica en la ecuación 1 en donde $[M]$ es la matriz de masas, $[C]$ es la matriz de amortiguamiento - viscoso, $[K]$ es la matriz de rigidez, $\{u\}$ son los desplazamientos nodales y F (t) es el vector de fuerzas externas.

En muchos programas de elemento finito, se utilizan matrices de masas concentradas mientras que en otros se utiliza la denominada matriz de masa consistente. La matriz de masas concentradas es diagonal, de manera que se desprecia la inercia rotacional. La matriz de masa consistente se evalua por un procedimiento similar al utilizado en la formulación de la matriz de rigidez. Esta matriz es simétrica no diagonal y se toma en cuenta los grados de libertad de rotación. Las características de amortiguamiento de una estructura son generalmente más difíciles de determinar que su masa o rigidez.

En la mayor parte de los casos se supone que el amortiguamiento es del - tipo viscoso o sea dependiente de la velocidad, como se indica en la ecuación 1.

La matriz de rigidez puede tener distintas formas, dependiendo del tipo - de análisis que se valla a realizar. Para análisis elástico es una matriz simétrica y contiene gran cantidad de ceros.

ANALISIS LINEAL

En este caso sólo se consideran deformaciones pequeñas y las matrices $[M]$, $[C]$, $[K]$ son simétricas y poco pobladas. Estas propiedades se - utilizan en la implementación de los programas de computadora para reducir los requerimientos de almacenaje.

Como en estructuras con gran número de grados de libertad se requiere demasiado tiempo de computadora para resolver problemas dinámicos, se acostumbra utilizar métodos de condensación para reducir los grados de libertad dinámico del modelo.

Los dos métodos más usados son:

1.- Condensación estática
2.- Reducción de Guyan

En el primer método se eliminan los grados de libertad asociados a masa nula de la matriz de rigidez, y se supone que no hay amortiguamiento asociado. En el segundo que se discute más en detalle, se supone que algunos grados de libertad están " esclavizados a otros " la ecuación general de movimiento puede describirse en forma particionada como se muestra en la ecuación 2, en donde U_1 representa el vector de desplazamiento nodales que se desea retener y U_2 es el vector de desplazamiento correspondiente a ser eliminado. El superíndice T denota la traspuesta de la matriz o vector. El método de Guyan parte de la hipótesis de que la ecuación 2 se resuelve en conjunto o sujeta a la restricción indicada en la ecuación 3, que también se puede escribir como se indica en la ecuación 3' en donde el superíndice -1 denota la inversa de la matriz. Esto conduce al sistema de ecuaciones 4 que tiene menos grados de libertad que el original.

Los métodos de condensación generalmente trabajan adecuadamente, siempre que las masas más grandes se mantengan incluídas en el modelo reducido y los grados de libertad estén uniformemente distribuídos en toda la estructura. Para el análisis transitorio de sistemas estructurales, los métodos más utilizados son:

1.- Integración directa
2.- Super posición modal

INTEGRACION DIRECTA

Los métodos de integración directa están basados en la integración paso a paso de las ecuaciones acopladas de movimiento, representados por las ecuaciones 1 ó 4. Generalmente estos métodos utilizan fórmulas de diferencias para expresar los desplazamientos, las velocidades y las aceleraciones nodales. Los dos grupos de métodos que se utilizan para la integración en el tiempo de las ecuaciones dinámicas, son: integración implícita e integración explícita, aunque la mayor parte de los programas de elemento finito utilizan el primero. Existen gran cantidad de fórmulas para la integración de ecuaciones dinámicas, siendo los más utilizados el método β de Newmark, el método θ de Wilson y el método de Houbolt.

A continuación describiremos brevemente el método β de Newmark:

En este método los vectores de velocidad y desplazamiento en el tiempo t_{n+1} se expresan como se indican en las ecuaciones 5, en donde u_n, $\dot{u}_n$, y $\ddot{u}_n$ son los vectores de desplazamiento, velocidad y aceleración respectivamente al final del enésimo intervalo de tiempo, Δt es el intervalo de tiempo, β es un parámetro. El valor de ese parámetro puede variar entre 1/8 y 1/4 y su selección afecta la estabilidad del método. Combinando la ecuación 1 en los tiempos t_{n-1}, t_n y t_{n+1} con la ecuación 5 se puede obtener la ecuación 6.

Esta ecuación representa un conjunto de ecuaciones diferenciales simultáneas, que permiten obtener estados sucesivos en el tiempo a partir de un conjunto de condiciones iniciales. Una vez conocidos los desplazamientos mediante la ecuación 6, los vectores de velocidad y aceleración se obtienen mediante las ecuaciones 5. Este método es muy útil cuando se tiene comportamiento no lineal del material y/o deformaciones grandes, pero generalmente toma bastante tiempo de computadora.

SUPER POSICION MODAL

En este método los grados de libertad físicos del modelo estructural original, se reemplazan por sus coordenadas normales. Para lograr esto, se deben determinar primeramente las frecuencias naturales y las formas modales del modal no amortiguado, lo cual se logra extrayendo las raíces λ de la ecuación 7, mismas que representan los cuadrados de las frecuencias naturales. Una vez encontrados los valores característicos, la misma ecuación 7 permite obtener los vectores característicos que en este caso, representan las formas modales. Existen muchos métodos para efectuar estos cálculos, tales como: el de Givens, Householder, potencias inversas, etc..., varios de estos métodos permiten obtener todos los valores característicos del sistema y en problemas con muchos grados de libertad, se acostumbran utilizar en combinación con procedimientos de condensación. Otros métodos son adecuados para obtener sólo un número limitado de valores característicos. Otro procedimiento comúnmente utilizado para determinar frecuencias naturales y formas modales, es el método de matrices de transferencia o transición, que son más adecuados para analizar estructuras tipo cadena o de conectividad simple. La base de este método consiste en encontrar la relación entre el vector de estado en el nodo i con el vector de estado en un nodo contiguo j. La relación entre ambos vectores expresada en la ecuación 8, define a la matriz de transición T_i.

Utilizando en forma sucesiva expresiones del tipo de la ecuación 8 para todos los miembros o elementos del sistema de tubería y eliminando los vectores de estado intermedios mediante multiplicaciones matrizales, es posible expresar la relación entre el vector de estado en el primer nodo del sistema que se muestra en la ecuación 9. Aplicando las condiciones de frontera a esta ecuación, se llega a un polinomio cuya solución son los cuadrados de las frecuencias naturales del sistema. La aplicación repetida de la ecuación 9 para cada una de las estaciones una vez conocidas las frecuencias naturales, conduce a las formas y fuerzas modales.

Una vez conocidos valores y vectores de un sistema, las ecuaciones de movimiento se pueden desacoplar para lo cual; se utiliza la matriz de modos ecuación 10 en donde cada columna corresponde al vector característico i .

Como en algunos casos el número de grados de libertad retenidos en la ecuación 10, es menor que el número total de grados de libertad del sistema, es necesario introducir la transformación indicada en la ecuación 11 en donde el vector $\{q\}$ contiene los desplazamientos generalizados denominados coordenadas normales. Substituyendo la ecuación 11 en las ecuaciones 1 ó 4 se llega a las ecuaciones 12 y 13.

TRATAMIENTO DEL AMORTIGUAMIENTO

Debido a la ortogonalidad de los vectores característicos las matrices $[\bar{M}]$ y $[\bar{K}]$ son diagonales, lo que ha conducido también a que se suponga que $[\bar{C}]$ es también diagonal. Esta hipótesis es cierta si la matriz de amortiguamiento es una combinación lineal de las ecuaciones de masa y rigidez, como se indica en la ecuación 14. En este caso la ecuación 12 se puede escribir como un sistema desacoplado de m . ecuación de tipo indicado en la ecuación 15. La solución de esta ecuación conduce a las coordenadas normales, que mediante la ecuación 11 pueden conducir a la obtención de los desplazamientos $\{u\}$.

Cuando la respuesta dinámica puede representarse por un número ilimitado de modos, es obvio que el método de superposición modal es el más económico. Esto depende del contenido de frecuencia en el vector de excitación y en las características dinámicas de estructuras, debiendo notarse que la mayor parte de tiempo de computadora se ocupa en la solución de problemas de valores característicos.

ANALISIS SISMICO

En este caso la excitación dinámica consiste en aceleraciones transmitidas a la estructura através de sus puntos de apoyo, siendo usual expresar las aceleraciones absolutas como lo indica la ecuación 22 en donde $\{\ddot{u}_g\}$ es el vector de aceleraciones nodales del terreno y $\{\ddot{u}_r\}$, es el vector de aceleraciones nodales relativas al terreno.

Componiendo la ecuación 22 con la ecuación 1, se obtiene la ecuación 23. El vector del lado derecho de esta ecuación, representa las fuerzas de excitación inducidas por el movimiento del terreno. Esta formulación tiene la desventaja de que sólo se puede aplicar la misma aceleración del terreno, y todos los soportes de la estructura. En cada caso de tuberías de plantas nucleares, los códigos requieren que se apliquen distintas excitaciones del terreno en cada uno de los apoyos, o también en el caso de un sistema de tuberías que este unido o apoyado en uno o varios puntos de una o más estructuras, que también están sujetas a la misma aceleración del terreno en cuyo caso, los puntos de apoyo de la tubería estarán sujetos a diferentes aceleraciones. Para esos casos las ecuaciones de movimiento se expresan más convenientemente en términos de los desplazamientos absolutos como se muestra en la ecuación 24.

La mayor parte de los programas de computadora disponibles, están basados en la formulación indicada en la ecuación 23 en parte por que es la que se utiliza en el análisis de edificios, que es de donde han derivado muchos de los programas y por ser más sencillo de programar que la ecuación 24 . Además en general los movimientos del terreno se tienen en forma de aceleración, no de desplazamiento velocidad como se requiere en la ecuación 24.

En el análisis sísmico de sistemas de tuberías, se utilizan 2 métodos:

1).- Respuesta transitoria que consiste en obtener la historia de la respuesta de la estructura en el tiempo. Este tipo de análisis puede hacerse por superposición modal o por integración directa en el tiempo.

2).- Método del espectro de respuesta que utiliza los modos normales del modelo estructural, por lo tanto, restringido a comportamiento elástico lineal.

METODO DEL ESPECTRO DE RESPUESTA

En este método primeramente se determinan las frecuencias naturales y formas modales. Mediante las formas modales, se desacoplan las ecuaciones de movimiento y se obtienen ecuaciones desacopladas similares a las ecuaciones 15, y que pueden expresarse como se indica en la ecuación 25 en donde d es el vector de dirección nodal del sismo, cuyos componentes son números enteros que varían entre 0 y 1 y $\{\ddot{u}g\}$ es la aceleración del terreno producido por el sismo.

La solución de las ecuaciones 25 pueden escribirse en términos de la denominada integral Duhamel, como se indica en la ecuación 26. Esta ecuación indica que la respuesta del iésimo modo depende de la frecuencia natural no amortiguada, del porcentaje de amortiguamiento crítico y la aceleración del terreno.

El valor máximo de la integral de la ecuación 26 se le llama valor espectral de la velocidad relativa al terreno, mientras que los seudovalores del desplazamiento y aceleración relativos al suelo se definen como se indica en la ecuación 28.

Se denomina espectro de respuesta a una gráfica que muestra la respuesta máxima de desplazamiento, velocidad o aceleración, para una aceleración del terreno y un factor de amortiguamiento dados en función de la frecuencia natural de vibración. En la figura 1 se muestra un espectro de respuesta típico, obtenido de una aceleración horizontal del terreno de 1.0G (aceleración de la gravedad) se muestra en la figura 1.

De acuerdo a las ecuaciones 26 y 27 el valor máximo de la coordenada normal de desplazamiento del iésimo modo, esta dada por la ecuación 29.

Los valores máximos de los desplazamientos naturales físicos correspondientes, se obtienen mediante la ecuación 30.

ESTIMACION DE LA RESPUESTA MAXIMA

Para estimar la respuesta máxima de estructura una vez encontrados los valores máximos para cada modo, pueden calcularse mediante cualquiera de los tres métodos siguientes:

1).- Suma de valores absolutos de los valores máximos. Este valor es conservador ya que los máximos en general, no ocurren al mismo tiempo.

2).- Raíz cuadrática media, en donde la respuesta máxima se obtiene como lo indica la ecuación 32, en donde N es el número de grados de libertad del sistema. Los esfuerzos y otros valores de respuesta se obtienen también mediante expresiones de raíz cuadrática media (RMS).

3).- Método de la Naval Research Laboratories (NRL) en el cual la respuesta pico en el nodo r se define mediante la ecuación 33, en donde el primer término es el máximo de la contribución modal u_{ri} en el nodo r y u_{rj} no se incluye en la suma del segundo término. Este método da resultados intermedios entre los 2 métodos anteriores.

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = \{f(t)\} \tag{1}$$

$$\begin{bmatrix} M_{11} & M_{12} \\ M_{12}^T & M_{22} \end{bmatrix} \begin{Bmatrix} \ddot{u}_1 \\ \ddot{u}_2 \end{Bmatrix} + \begin{bmatrix} C_{11} & C_{12} \\ C_{12}^T & C_{22} \end{bmatrix} \begin{Bmatrix} \dot{u}_1 \\ \dot{u}_2 \end{Bmatrix} + \begin{bmatrix} K_{11} & K_{12} \\ K_{12}^T & K_{22} \end{bmatrix} \begin{Bmatrix} u_1 \\ u_2 \end{Bmatrix} = \begin{Bmatrix} f_1(t) \\ f_2(t) \end{Bmatrix} \tag{2}$$

$$[K_{22}]\{u_2\} = [K_{12}]^T\{u_1\} \tag{3}$$

$$\{u_2\} = -[K_{22}]^{-1}[K_{12}]^T\{u_1\} = [H_{12}]\{u_1\} \tag{3'}$$

$$[M_c]\{\ddot{u}_1\} + [C_c]\{\dot{u}_1\} + [K_c]\{u_1\} = \{f_c\} \tag{4}$$

$$[M_c] = [M_{11} + M_{12}H_{12} + H_{12}^T M_{12}^T + H_{12}^T M_{22} H_{12}]$$

$$[C_c] = [C_{11} + C_{12}H_{12} + H_{12}^T C_{12}^T + H_{12}^T C_{22} H_{12}]$$

$$[K_c] = [K_{11} + K_{12}H_{12}]$$

$$\{f_c\} = \{f_1(t)\} + [H_{12}]^T\{f_2(t)\}$$

Integracion Directa :

$$\left\{\dot{u}_{n+1}\right\} = \left\{\dot{u}\right\}_n + \frac{\Delta t}{2}\left\{\ddot{u}_n\right\} + \frac{\Delta t}{2}\left\{\ddot{u}_{n+1}\right\}$$

$$\left\{u_{n+1}\right\} = \left\{u_n\right\} + \Delta t\left\{\dot{u}_n\right\} + (1-\beta)\,\Delta t^2\left\{\ddot{u}_n\right\} + \beta\,\Delta t^2\left\{\ddot{u}_{n+1}\right\} \tag{5}$$

$$\left[\frac{1}{\Delta t^2}M + \frac{1}{2\Delta t}C + \beta K\right]\left\{u_{n+1}\right\} = \beta\left\{f_{n-1}\right\} + (1-2\beta)\left\{f_n\right\} + \beta\left\{f_{n+1}\right\}$$

$$+\left[\frac{2}{\Delta t^2}M - (1-2\beta)K\right]\left\{u_n\right\} - \left[\frac{M}{\Delta t^2} - \frac{1}{2\Delta t}C + \beta K\right]\left\{u_{n-1}\right\} \tag{6}$$

Superposición Modal:

$$\left[K-\lambda M\right]\left\{u\right\}=\left\{0\right\} \tag{7}$$

$$\left\{\begin{matrix} u_j \\ fe_j \end{matrix}\right\}=\left[T_i\right]\left\{\begin{matrix} u_i \\ fe_i \end{matrix}\right\} \tag{8}$$

$$\left\{\begin{matrix} u_m \\ fe_m \end{matrix}\right\}=\left[T_m\right]\cdots\left[T_2\right]\left[T_1\right]\left\{\begin{matrix} u_o \\ fe_o \end{matrix}\right\} \tag{9}$$

$$\left[\phi\right]=\left[\left\{\phi_1\right\}\left\{\phi_2\right\}\cdots\left\{\phi_m\right\}\right] \tag{10}$$

$$\left\{u\right\}=\left[\phi\right]\left\{q\right\} \tag{11}$$

$$\left[\bar{M}\right]\left\{\ddot{q}\right\}+\left[\bar{C}\right]\left\{\dot{q}\right\}+\left[\bar{K}\right]\left\{q\right\}=\left\{\bar{f}(t)\right\} \tag{12}$$

$$\begin{aligned}
\left[\bar{M}\right] &= \left[\phi\right]^T\left[M\right]\left[\phi\right] \\
\left[\bar{C}\right] &= \left[\phi\right]^T\left[C\right]\left[\phi\right] \\
\left[\bar{K}\right] &= \left[\phi\right]^T\left[K\right]\left[\phi\right] \\
\left\{\bar{f}(t)\right\} &= \left[\phi\right]^T\left\{f(t)\right\}
\end{aligned} \tag{13}$$

Tratamiento del Amortiguamiento

$$[C] = a_1[M] + a_2[K] \tag{14}$$

$$m_i\ddot{q}_i + c_i\dot{q}_i + k_i q_i = f_i(t), \quad i = 1, 2, \cdots, m \tag{15}$$

$$\ddot{q}_i + 2\xi_i\omega_i\dot{q}_i + \omega_i^2 q_i = \frac{1}{m_i} f_i(t)$$

en donde $\xi_i = \frac{c_i}{2m_i\omega_i}$ es la relación de amortiguamiento

y ω_i es la frecuencia natural

Análisis Sísmico:

$$\{\ddot{u}_a\} = \{\ddot{u}_r\} + \{\ddot{u}_g\} \tag{22}$$

$$[M]\{\ddot{u}_r\} + [C]\{\dot{u}_r\} + [K]\{u_r\} = -[M]\{\ddot{u}_g\} \tag{23}$$

$$[M]\{\ddot{u}_a\} + [C]\{\dot{u}_a\} + [K]\{u_a\} = [C]\{\dot{u}_g\} + [K]\{u_g\} \tag{24}$$

Método del Espectro de Respuesta:

$$\ddot{q}_i + 2\,\xi_i\,\omega_i\,\dot{q}_i + \omega_i^2\,q_i = \frac{-1}{m_i}\left\{\phi_i\right\}^T\left[M\right]\left\{d\right\}\ddot{u}_g \qquad (25)$$

$$q_i(t) = \frac{\left\{\phi_i\right\}^T\left[M\right]\left\{d\right\}}{\omega_i\, m_i}\int_0^t \ddot{u}_g(\tau)\exp\left[-\xi_i\,\omega_i(t-\tau)\right]\operatorname{sen}\omega_i(t-\tau)\,d\tau \qquad (26)$$

$$S_{v_i} = \left[\int_0^t \ddot{u}_g(\tau)\exp\left[-\xi_i\,\omega_i(t-\tau)\right]\operatorname{sen}\omega_i(t-\tau)\,d\tau\right]_{max.} \qquad (27)$$

$$S_{d_i} = \frac{1}{\omega_i}\,S_{v_i}$$

$$(28)$$

$$S_{a_i} = \omega_i\,S_{v_i}$$

$$q_i\Big|_{max.} = \frac{\left\{\phi_i\right\}\left[M\right]\left\{d\right\}}{\omega_i\, m_i}\,S \qquad (29)$$

$$\left\{u\right\}_i\Big|_{max} = \left\{\phi_i\right\}\,q_i\Big|_{max} \qquad (30)$$

Estimación de la respuesta máxima

Método 1)

$$\left\{ u \right\}\Big|_{max} = \sum_{i=1}^{m} \Big| \left\{ u \right\}_i \Big|_{max} \tag{31}$$

Método 2) RMS

$$u_r\Big|_{max} = \left[\sum_{i=1}^{m} (u_{ri})^2 \right]^{1/2} \qquad r = 1, \cdots, N \tag{32}$$

Método 3) NRL

$$u_r\Big|_{max} = \Big| u_{rl} \Big|_{max,max} + \left(\sum_{\substack{i=1 \\ i \neq j}}^{m} u_{ri}\Big|^2_{max} \right)^{1/2} \tag{33}$$

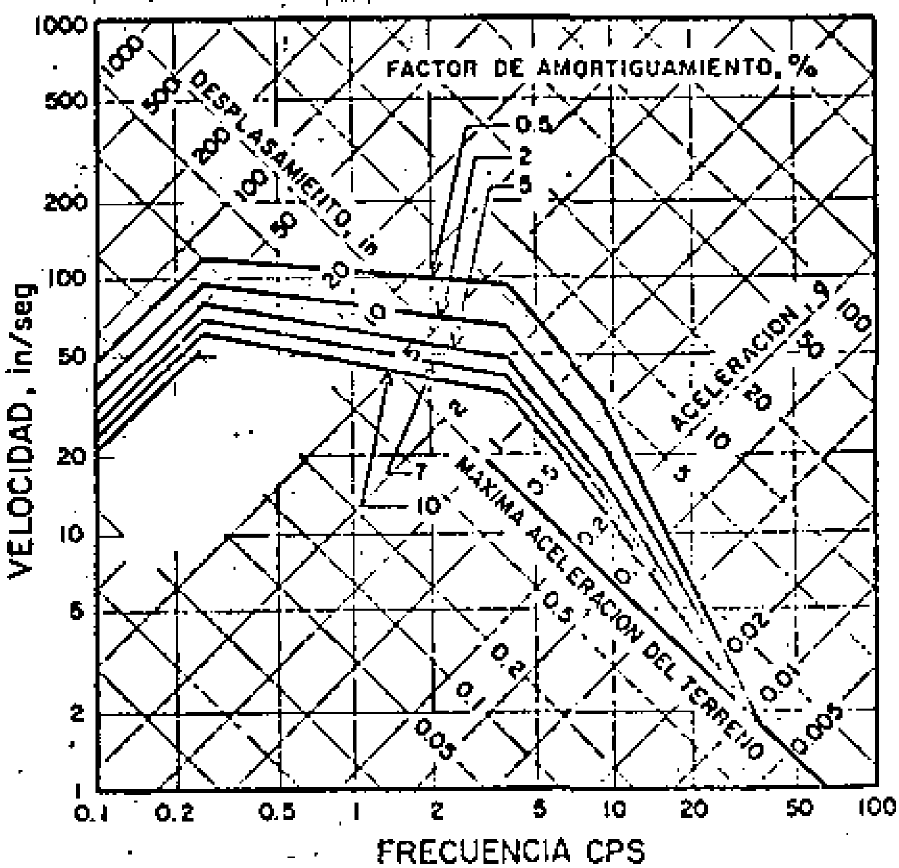


Fig- 1 Espectro de diseño para respuesta vertical escalado a 1 g. de aceleración horizontal del terreno (USA A.E.C)

Nombre del Programa	Tuberia Recta	Reduccion Concentrica	Tubo recto con cortante	Codos	Tes	Resortes		Elem. "gap"	Elemento de fricción	Masa		Cascarón
						Lineal	No-Lineal			Concentr.	Consist.	
ADLPIPE	Si	No	Si	Si	Si	Si	No	No	No	Si	No	No
ANSYS	Si	No	Si	Si	No	Si	Si	Si	Si	Si	Si	Si
MARC	Si	Si	Si	Si	No	Si	Si	Si	Si	No	Si	Si
NASTRAN	Si	Si	Si	No	No	Si	Si	Si	Si	Si	Si	Si
NUPIPE	Si	No	Si	Si	Si	Si	No	No	No	Si	No	No
PIPDYN	Si	No	Si	Si	Si	Si	No	No	No	Si	No	No
PIPERUP	Si	No	Si	Si	Si	Si	No	No	No	Si	No	No
PIPESD	Si	No	No	Si	Si	Si	No	No	No	Si	No	No
SAGS/DAGS	No	No	Si	Si	Si	Si	No	No	No	Si	No	No
SAP IV	Si	No	Si	Si	No	Si	No	No	No	Si	No	Si
STARDYNE	Si	No	Si	Si	Si	Si	Si	Si	No	Si	Si	No
WECAN	Si	No	Si	Si	No	Si	Si	Si	Si	Si	Si	Si

Analisis de tuberia por Elemento finito

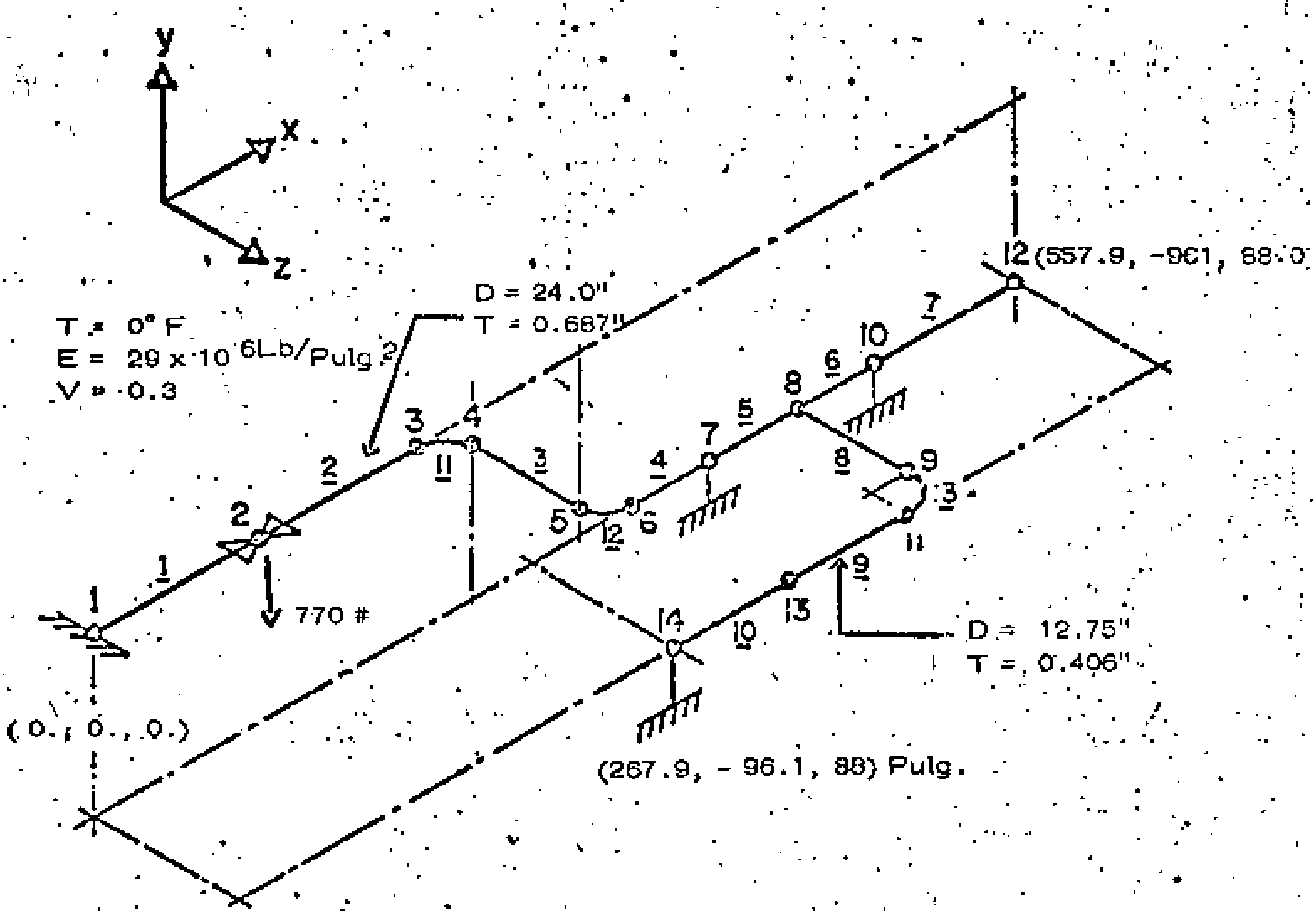
Nombre del programa	Condensación	Modal (Modos normales)					Integración directa en el tiempo				
		Frecuencias Naturales	Lineal	Elem No Lin Pseudo-Fuerza	Sismico	Espectro de Resp	Lineal	Elem No Lin	Plasticidad	Def grande	Sismico
ADLPIPE	Si	Si	Si	No	Si	Si	Si	No	No	No	No
ANSYS	Si	Si	Si	No	Si	Si	Si	Si	Si	Si	Si
MARC	Si	Si	No	No	No	No	Si	Si	Si	Si	No
NASTRAN	Si	Si	Si	Si	No	Si	Si	Si	No	No	No
PIPDYN	Si	Si	Si	No	Si	Si	Si	No	No	No	Si
PIPESD	Si	Si	No	No	No	Si	No	No	No	No	No
SAGS/DAGS	No	Si	Si	No	Si	Si	No	No	No	No	Si
SAP IV	No	Si	Si	No	Si	Si	Si	No	No	No	Si
STARDYNE	Si	Si	Si	Si	Si	Si	Si	Si	No	No	Si
WECAN	Si	Si	No	No	No	Si	Si	Si	Si	No	Si

Capacidades de varios programas de computadora
para análisis de tuberias

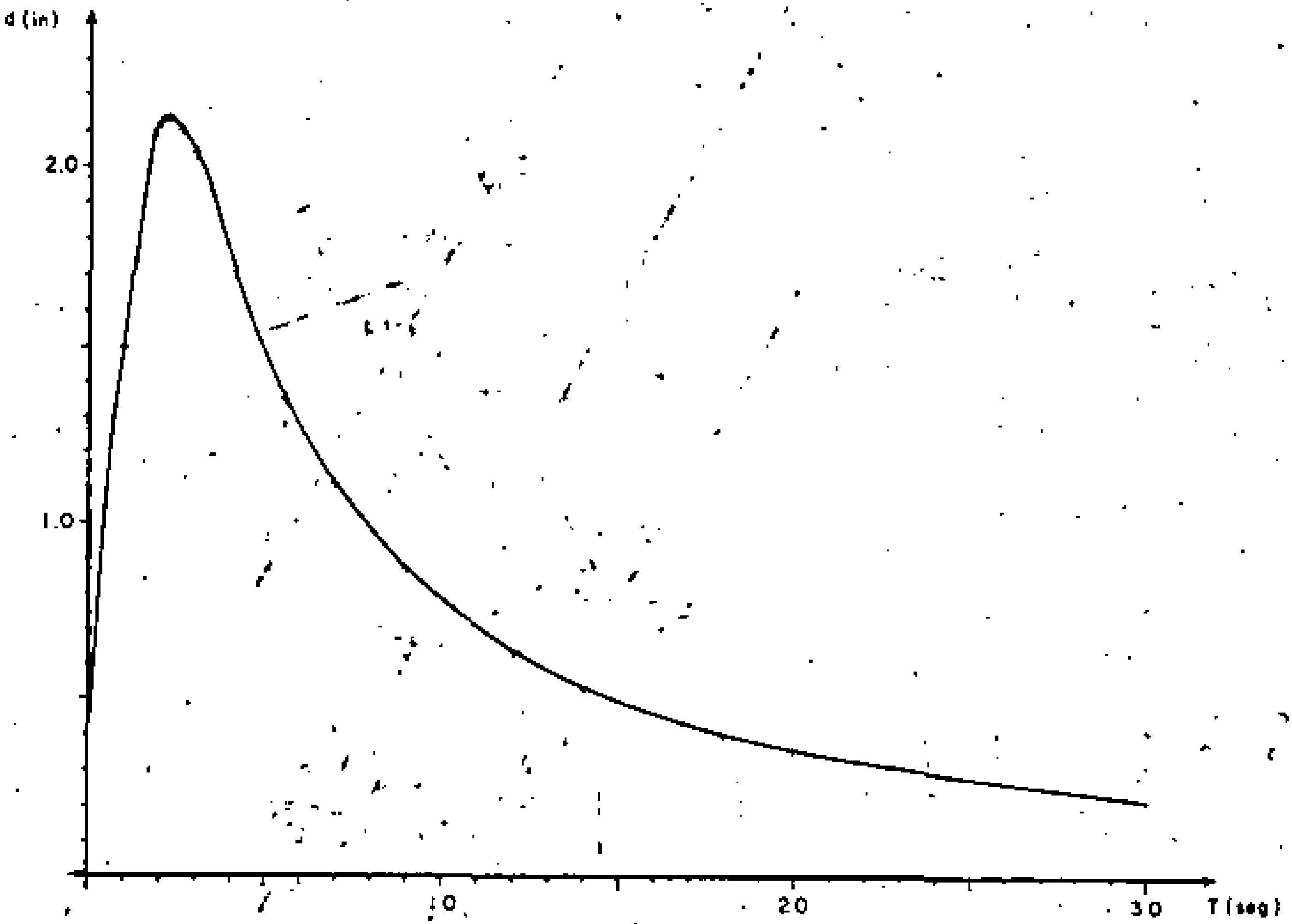
PROBLEM 12.2 PIPE NETWORK RESPONSE SPECTRUM ANALYSIS

Problem Definition

Ref: SAP IV Manual, problem 4.



MODO	FREC. CIRCULAR (RAD/SEG)	FRECUENCIA (CPS)	PERIODO (SEG)
1	9.133	1.454	0.6879
2	27.71	4.410	0.2268
3	50.24	7.997	0.1251
4	64.95	10.34	0.0967
5	74.47	11.85	0.0844



ESPECTRO DE DESPLAZAMIENTOS

REFERENCIAS

USA Standard code for Pressure Piping, Power Piping, ANSI B31.1-1977. Published by the American Society of Mechanical Engineers, 345, East 47th Street, New York, N.Y. 10017.

Pressure Vessel Technology, Part 1, Design and Analysis and Part 2, Materials and Fabrication, First International Conference on Pressure Vessel Technology, Delft, Holland, 1969. Published by the American Society of Mechanical, Eng. 345 East 47th Street, New York, N.Y. 10017.

Rodabaugh, E.C. and George H.H., "Effect of Internal Pressure on Flexibility and Stress-Intensification Factors of Curved Pipe or Welding Elbows", Trans. ASME, Vol. 79, p 939 (1957).

Brock, J.E., "Expansión and Flexibility", Chapter 4 of Piping Handbook, 5th edition (1969), Mc Graw-Hill Book Co., New York.

ASME Boiler and Pressure Vessel Code, Section III, Division 1, 1980.

C N T R O L   I N F O R M A T I O N

NUMBER OF NODAL POINTS = 14
NUMBER OF ELEMENT TYPES = 1
NUMBER OF LOAD CASES = 1
NUMBER OF FREQUENCIES = 5
ANALYSIS CODE (NDYN) = 3
  EQ.0, STATIC
  EQ.1, MODAL EXTRACTION
  EQ.2, FORCED RESPONSE
  EQ.3, RESPONSE SPECTRUM
  EQ.4, DIRECT INTEGRATION
  EQ.5, FREQUENCY RESPONSE
SOLUTION MODE (MODEX) = 0
  EQ.0, EXECUTION
  EQ.1, DATA CHECK
NUMBER OF SUBSPACE
ITERATION VECTORS (NAD) = 0
EQUATIONS PER BLOCK = 0
TAPE10 SAVE FLAG (N10SV) = 0
GRAVITATIONAL CONSTANT = 386.40
TOTAL BLANK COMMON (MTOT)=15500

UIRED BLANK COMMON FOR THIS STEP= 141

AL POINT INPUT DATA

DE NUMBER	BOUNDARY CONDITION CODES X	Y	Z	XX	YY	ZZ	NODAL POINT COORDINATES X	Y	Z		T
1	1	1	1	1	1	1	.000	.000	.000	0	.000
2	0	0	1	0	0	0	100.000	.000	.000	0	.000
3	0	0	0	0	0	0	200.000	.000	.000	0	.000
4	0	0	0	0	0	0	225.456	-10.544	.000	0	.000
5	0	0	0	0	0	0	300.456	-65.544	.000	0	.000
6	0	0	0	0	0	0	325.912	-96.058	.000	0	.000
7	0	1	0	0	0	0	375.912	-96.058	.000	0	.000
8	0	0	0	0	0	0	425.912	-96.058	.000	0	.000
9	0	0	0	0	0	0	425.912	-96.058	70.000	0	.000
10	0	1	0	0	0	0	475.912	-96.058	.000	0	.000
11	0	0	0	0	0	0	407.912	-96.058	88.000	0	.000
12	0	0	0	0	0	0	557.912	-96.058	.000	0	.000
13	0	0	0	0	0	0	337.912	-96.058	85.000	0	.000
14	0	1	0	0	0	0	267.912	-96.058	88.000	0	.000

32

P I P E   E L E M E N T   I N P U T   D A T A

C O N T R O L   I N F O R M A T I O N

```
REQUIRED BLANK COMMON FOR THIS STEP=     147
      NUMBER OF PIPE ELEMENTS           =      13

      NUMBER OF MATERIAL SETS           =       2

      MAXIMUM NUMBER OF MATERIAL
      TEMPERATURE INPUT POINTS          =       1

      NUMBER OF SECTION PROPERTY SETS   =       2

      NUMBER OF BRANCH POINT NODES      =       0

      MAXIMUM NUMBER OF TANGENTS
      COMMON TO A BRANCH POINT          =       4

      FLAG FOR NEGLECTING AXIAL
      DEFORMATIONS IN BEND ELEMENTS     =       0
      (EQ.1, NEGLECT)
```

M A T E R I A L   P R O P E R T Y   T A B L E S

```
MATERIAL NUMBER      = (    1)
NUMBER OF
TEMPERATURE POINTS   = (    1)
IDENTIFICATION       = ( 70 F CARBON STEEL                )
```

POINT NUMBER	TEMPERATURE	YOUNG*S MODULUS	POISSON*S RATIO	THERMAL EXPANSION
1	.00	29000000.0	.300	.000

```
MATERIAL NUMBER      = (    2)
NUMBER OF
```

SECTION PROPERTY TABLE

SECTION NUMBER	OUTSIDE DIAMETER	WALL THICKNESS	SHAPE FACTOR FOR SHEAR	WEIGHT/ UNIT LENGTH	MASS/ UNIT LENGTH	DESCRIPTION
1	24.000	.6870	.0000	.2875+02	.7440-01	24 INCH- SHD.40
2	12.750	.4060	.0000	.8500+01	.2200-01	12 INCH SCHD.40

ELEMENT LOAD CASE MULTIPLIERS

	CASE A	CASE B	CASE C	CASE D
X-DIRECTION GRAVITY	.000	.000	.000	.000
Y-DIRECTION GRAVITY	.000	.000	.000	.000
Z-DIRECTION GRAVITY	.000	.000	.000	.000
THERMAL DISTORTION	.000	.000	.000	.000
PRESSURE DISTORTION	.000	.000	.000	.000

PIPE ELEMENT INPUT DATA

ELEMENT NUMBER	ELEMENT TYPE	NODE -I	NODE -J	MATL. NUMBER	SECTION NUMBER		REFERENCE TEMPERATURE (BEND RADIUS)	INTERNAL PRESSURE (THIRD POINT)	DIRECTION COSINE A(YX) (X3-ORDINATE)	DIRECTION COSINE A(YY) (Y3-ORDINATE)	DIRECTION COSINE A(YZ) (Z3-ORDINATE)
1	TANG T	1	2	1	1		.00	.00	.0000	.0000	.000
2	TANG T	2	3	1	1		.00	.00	.0000	.0000	.000
3	TANG T	4	5	1	1		.00	.00	.0000	.0000	.000
4	TANG T	6	7	1	1		.00	.00	.0000	.0000	.000
5	TANG T	7	8	1	1		.00	.00	.0000	.0000	.000
6	TANG T	8	10	1	1		.00	.00	.0000	.0000	.000
7	TANG T	10	11	1	1		.00	.00	.0000	.0000	.000
8	TANG T	8	9	1	2		.00	.00	.0000	.0000	.000
9	TANG T	11	13	2	2		.00	.00	.0000	.0000	.000
10	TANG T	13	14	2	2		.00	.00	.0000	.0000	.000
11	BEND	3	4	1	1		.00	.00			
						(	35.000)	(TI) (	214.912)(	.000)(	.000
12	BEND	5	6	1	1		.00	.00			
						(	36.000)	(TI) (	211.000)(	-96.000)(	.000
13	BEND	9	11	2	2		.00	.00			
						(	15.000)	(TI) (	405.912)(	-96.000)(	89.000

REQUIRED BLANK COMMON FOR THIS STEP= 235

E Q U A T I O N   P A R A M E T E R S

```
TOTAL NUMBER OF EQUATIONS        =    75
BANDWIDTH                        =    16
NUMBER OF EQUATIONS IN A BLOCK   =    75
NUMBER OF BLOCKS                 =     1
```

E I G E N V A L U E   A N A L Y S I S

DETERMINANT SEARCH SOLUTION IS CARRIED OUT

CONTROL INFORMATION

```
     FLAG FOR ADDITIONAL PRINTING  =     1
       EQ.0, SUPPRESS
       EQ.1, PRINT

     STURM SEQUENCE CHECK FLAG (*) =     0
       EQ.0, PERFORM CHECK
       EQ.1, PASS

     MAXIMUM ITERATION CYCLES (*)  =    16

     CONVERGENCE TOLERANCE (*)     =        .1000-04

     CUT-OFF FREQUENCY (CPS)       =        .1000+00

     NUMBER OF STARTING ITERATION
     VECTORS TO BE READ FROM
     TAPE10 (*)                    =     0

     (*)  APPLICABLE TO SUBSPACE
          ITERATION SOLUTIONS ONLY
```

SOLUTION IS SOUGHT FOR FOLLOWING EIGENPROBLEM

```
NUMBER OF EQUATIONS                 =    75

HALF BANDWIDTH OF STIFFNESS MATRIX  =    18

NUMBER OF EQUATION BLOCKS           =     1

NUMBER OF EQUATIONS PER BLOCK       =    75
```

M O D A L   A N A L Y S I S

MODE NUMBER 1
FREQUENCY= 1.41361 HZ

EIGENVECTORS NORMALIZED TO A UNIT MASS MATRIX

DISPLACEMENTS/ROTATIONS OF UNRESTRAINED NODES

NODE NUMBER	X- TRANSLATION	Y- TRANSLATION	Z- TRANSLATION	X- ROTATION	Y- ROTATION	Z- ROTATION
2	1.56792-006	-5.84209-005	-6.66394-003	6.91158-005	1.66163-004	-7.81016-007
3	2.12961-006	-1.31194-004	-3.00271-007	9.67712-005	2.90485-004	-6.09779-007
4	9.14680-006	-1.15068-004	-4.22151-007	1.68150-004	5.11602-004	1.10592-006
5	1.12913-004	-2.46071-005	-1.02445-001	1.95635-004	5.97359-004	1.30777-006
6	1.23364-004	-6.27166-006	-1.09317-001	2.37273-004	7.06734-004	2.15162-007
7	1.24145-004	0.00000	-1.05117-001	2.33901-004	7.27357-004	2.41079-007
8	1.24925-004	6.39115-006	-1.11630-001	2.22542-004	7.33971-004	-6.02177-007
9	5.11449-002	-1.59242-002	-1.91601-001	2.22466-004	7.21146-004	-4.44379-005
10	1.24928-004	0.00000	-2.23565-001	2.22548-004	7.39279-004	-1.07971-007
11	4.36992-002	-1.62094-002	-1.79422-001	1.86115-004	6.55946-004	-1.09479-004
12	1.74929-004	-6.55454-006	-2.07207-001	2.22182-004	7.41574-004	-1.07957-007
13	6.36601-002	-9.86428-003	-1.22045-001	1.86215-004	6.46372-004	-1.22251-004
14	6.36866-002	0.00000	-3.27876-002	1.55158-004	6.44729-004	-1.40014-004

M O D A L   A N A L Y S I S

MODE NUMBER   2
FREQUENCY=   4.41978   HZ

EIGENVECTORS NORMALIZED TO A UNIT MASS MATRIX

DISPLACEMENTS/ROTATIONS OF UNRESTRAINED NODES

NODE NUMBER	X- TRANSLATION	Y- TRANSLATION	Z- TRANSLATION	X- ROTATION	Y- ROTATION	Z- ROTATION
2	3.44703-008	-1.14617-003	6.06726-003	-9.60700-005	-9.60712-005	-1.79524-005
3	6.88943-005	-2.91469-003	1.16305-002	-1.97740-004	-9.30025-005	-1.29953-005
4	2.25423-004	-4.76395-003	3.06820-002	-2.07483-004	-1.37909-005	2.72363-005
5	1.61841-003	-4.24051-004	3.49998-002	-2.00877-004	1.18522-004	3.59665-005
6	2.64733-003	-2.06525-005	3.09186-002	-1.79470-004	4.27134-004	1.87285-006
7	2.66394-003	0.00000	7.93522-003	-1.75673-004	5.01233-004	1.44676-007
8	2.86022-002	5.39587-006	-1.94014-002	-1.55770-004	5.99215-004	3.70017-007
9	6.64013-002	1.23465-002	-1.52999-002	-1.76264-004	1.70297-003	2.36419-005
10	2.98083-003	0.00000	-4.57986-002	-1.55770-004	6.17082-004	-1.34054-007
11	1.44057-001	3.43103-002	5.24674-002	-1.51656-004	5.19405-003	4.42162-005
12	2.33126-003	-1.15044-005	-1.02570-001	-1.75770-004	6.21064-004	-1.35172-007
13	1.44326-001	7.65406-003	4.41246-001	-1.51656-004	5.76596-003	1.04072-004
14	1.44339-001	0.00000	8.91177-001	-1.51656-004	5.90767-003	1.11540-004

MODAL PARTICIPATION FACTORS

MODE	X-DIRECTION	Y-DIRECTION	Z-DIRECTION
1	.2742+00	-.8435-01	-.5566+01
2	.6551+00	-.1862-01	.2763+01
3	-.1002+01	.2214+00	-.3501+01
4	.4274+01	-.3277+01	-.1356+01
5	.5617+00	-.2119+01	.3158+01

SPECTRUM TABLE (DISPLACEMENT SPECTRUM OF PIPDYN MANUAL

NUMBER OF POINTS = 16
SCALE FACTOR ( ) = .1000+01

INPUT POINT	PERIOD	SPECTRUM VALUE
1	.0000	.4842+00
2	.5000-01	.4633+00
3	.1000+00	.1495+01
4	.1500+00	.2037+01
5	.2000+00	.2125+01
6	.3000+00	.2107+01
7	.4000+00	.2109+01
8	.5000+00	.2078+01
9	.6000+00	.1328+01
10	.7000+00	.1104+01
11	.9000+00	.8049+00
12	.1000+01	.6213+00
13	.1400+01	.5249+00
14	.1800+01	.3976+00
15	.2000+01	.2512+00
16	.3000+01	.2142+00

REQUIRED BLANK COMMON FOR THIS STEP= 540

REQUIRED BLANK COMMON FOR THIS STEP= 540

RESPONSE SPECTRUM ANALYSIS

RESPONSE FOR MODE 1
FREQUENCY= 1.45391 HZ

DISPLACEMENTS/ROTATIONS OF UNRESTRAINED NODES

NODE NUMBER	X- TRANSLATION	Y- TRANSLATION	Z- TRANSLATION	X- ROTATION	Y- ROTATION	Z- ROTATION
2	6.56453-009	-2.11345-007	-3.71564-005	3.05704-007	6.95634-007	-3.27254-009
3	1.01071-006	-5.99418-007	-1.28286-004	4.11906-007	1.19936-006	-2.04674-009
4	4.07659-006	-5.26653-007	-1.02430-004	7.90956-007	2.19122-006	4.85109-009
5	5.70974-007	-1.03330-007	-4.73574-004	8.52625-007	2.49328-006	5.47264-009
6	5.16455-007	-2.62627-008	-4.95516-004	9.34478-007	2.97324-006	5.00765-010
7	5.15735-007	0.00000	-6.45389-004	9.79214-007	3.02637-006	5.90452-010
8	5.22992-007	2.67625-008	-8.03261-004	9.73551-007	3.07757-006	-2.17247-009
9	2.11116-004	-6.62458-005	-8.77297-004	9.31345-007	3.01905-006	-1.66035-007
10	5.23224-007	0.00000	-9.05626-004	9.73551-007	3.09412-006	-4.51806-010
11	2.66611-004	-7.62378-005	-7.01186-004	7.87704-007	2.74627-006	-4.56192-007
12	5.27231-007	-3.70643-006	-1.21101-003	9.73551-007	3.10457-006	-4.52175-010
13	2.66615-004	-4.03637-005	-5.63754-004	7.87704-007	2.70584-006	-5.57789-007
14	2.66622-004	0.00000	-3.71677-004	7.87704-007	2.69913-006	-5.86164-007

R E S P O N S E   S P E C T R U M   S T R E S S   C O M P O N E N T S   (KIND= 1)

FOR EACH ELEMENT, THE FOLLOWING INFORMATION IS PRINTED

1. NUMBER OF THE MODE WITH THE LARGEST STRESS.
2. VALUE OF THE STRESS COMPONENT IN THAT MODE.
3. IF REQUESTED, MODE BY MODE STRESSES.
4. RESULTANT OF THE MODAL SUMMATION. (SQUARE ROOT OF THE SUM OF THE SQUARES)

ELEMENT TYPE (3/D P I P E) / / / ELEMENT NUMBER (1)

	PX(I)	VY(I)	VZ(I)	TX(I)	MY(I)	MZ(I)	PX(J)	VY(J)	VZ(J)	
	4	4	4	5	4	4	4	4	4	
MAXIMUM	3.694+01	-3.455+01	-9.937+00	3.610+02	1.229+03	-3.761+03	3.694+01	-3.455+01	-9.937+00	3.
RESULTANT	3.704+01	3.463+01	1.398+01	4.730+02	1.936+03	3.769+03	3.704+01	3.463+01	1.398+01	4.

ELEMENT TYPE (3/D P I P E) / / / ELEMENT NUMBER (2)

	PX(I)	VY(I)	VZ(I)	TX(I)	MY(I)	MZ(I)	PX(J)	VY(J)	VZ(J)	
	4	4	3	5	1	4	4	4	3	
MAXIMUM	3.666+01	-1.905+01	-4.930+00	3.610+02	6.930+02	-3.355+02	3.666+01	-1.905+01	-4.930+00	3
RESULTANT	3.677+01	1.910+01	7.457+00	4.738+02	7.100+02	3.367+02	3.677+01	1.910+01	7.457+00	4

ELEMENT TYPE (3/D P I P E) / / / ELEMENT NUMBER (3)

	PX(I)	VY(I)	VZ(I)	TX(I)	MY(I)	MZ(I)	PX(J)	VY(J)	VZ(J)	
	4	4	5	3	1	4	4	4	5	
MAXIMUM	2.937+01	2.172+01	-1.976+00	3.147+02	3.574+02	1.525+03	2.937+01	2.172+01	-1.976+00	3.
RESULTANT	2.947+01	2.177+01	3.594+00	4.618+02	5.676+02	1.529+03	2.947+01	2.177+01	3.594+00	4.

42

ELEMENT TYPE (3/D P I P E) / / / ELEMENT NUMBER (4)

	PX(I)	VY(I)	VZ(I)	TX(I)	MY(I)	MZ(I)	PX(J)	VY(J)	VZ(J)	
	4	4	4	5	3	4	4	4	4	
MAXIMUM	2.679+01	-3.453+00	2.971+00	2.000+02	-4.379+02	-1.007+03	2.679+01	-3.453+00	2.971+00	2
RESULTANT	2.698+01	3.523+00	3.772+00	2.159+02	5.650+02	1.006+03	2.699+01	3.523+00	3.772+00	2.

ELEMENT TYPE (3/D P I P E) / / / ELEMENT NUMBER (5)

DIVISION DE EDUCACION CONTINUA
FACULTAD DE INGENIERIA U.N.A.M.

IX CURSO INTERNACIONAL DE INGENIERIA SISMICA

DISEÑO SISMICO DE ESTRUCTURAS ESPECIALES

P U E N T E S

Dr. Luis Esteva Maraboto

M. en C. Enrique del Valle Calderón

Julio, 1983

P U E N T E S

* Sesismic analysis of the elevated structure for the México city "metro"

** Evaluation of analytical procedures used in bridge seismic design practice

*** Earthquake-resistant design of bridges

**** Seismic response of multi-support structures

***** Influencia en la respuesta sísmica del puente Coatzacoalcos II, de las diferencias de fase en los movimientos de sus apoyos

****** Influencia en la respuesta sísmica del puente Coatzacoalcos II, de las diferencias de fase en los movimientos de sus apoyos. Segunda Parte

Agosto 1983

~~Julio, 1981~~

SEISMIC ANALYSIS OF THE ELEVATED STRUCTURE FOR THE MEXICO CITY "METRO"

Enrique del Valle (I)
Manuel Díaz-Canales (II)
Jorge Prince (III)
Alejandro Vázquez (IV)

Summary. Seismic analysis of the elevated structure for the Mexico City Metro is described. The structure was idealized as an inverted pendulum. Rotatory inertia and soil structure interaction effects were included in the dynamic analyses performed. A comparison with the results of the static analysis is made. Field tests to determine the actual dynamic properties in situ were carried out.

Introduction. An extension of the Metropolitan Transportation System (Metro) of Mexico City is under construction; it will have a new elevated line, 10 km long. Extensive studies were performed to determine the best type of structure, after which it was decided to use prestressed-concrete box-section beams, 8 m wide, cast in place and postensioned, with spans ranging from 25 to 40m supported on a single line of columns with variable cross section (fig. 1). The foundation consists of spread footings on friction piles.

Beam supports consist of neoprene and steel pads. Different thicknesses were used on each end in order to have a hinged-simple supported beam. Two pads on each side spaced 2.5m transversely to the beam take overturning effects. An extension of the end diaphragms enter a box left in the columns, to transmit all lateral loads to them. To avoid collapse of beams due to excessive movement during strong earthquakes tie-bars were used joining the ends of the two beams resting on each column.

Line loads are of two types: passenger trains with axle loads of 15.9 ton including impact, and a maintenance train, with axle loads of 25.0 ton. Different arrangements were used in order to obtain maximum effects when these loads were combined with earthquake.

Seismic analysis. The structure was analyzed using the Mexico City building code which specifies, for the high compressibility clay deposit where most of the line will be located, a seismic coefficient of 0.24 g, which should be increased 30 per cent for the case of special structures. To compute forces, this coefficient may be reduced according to ductility characteristics. For the Metro structure the reduction factor is 2; (ref.1).

The Code specifies that analyses may be static or dynamic. For the static analysis of inverted pendulum structures, defined as those having more than 50 per cent of the load concentrated at the top, with lateral forces resisted by a single element, rotatory inertia should be included using an expression given in the Code. An additional reduction of design forces is possible using a design spectrum and estimating the fundamental period of vibration. This reduction is generally possible in the case of very rigid structures on soft soil or flexible structures on stiff soil. Dynamic analysis may be step by step using four different accelerograms with intensity -

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II. Vicepresident, ICA Group
III. Subdirector, Institute of Engineering, National University of Mexico
IV. Head of Engineering. ISTME, ICA Group.

compatible with the Code, or a modal analysis using a design spectrum.

To obtain seismic effects the Code specifies that the structure should be analyzed in two orthogonal directions. For the case of inverted pendulum structures, seismic effects in one direction and 50 per cent of the seismic effects on the other direction are combined with gravity loads.

Static Analysis. According to the Code, seismic effects for inverted pendulum structures consist of a horizontal force and a moment applied at the top. The horizontal force is equal to the mass times the seismic coefficient reduced by ductility. An increment of 30% was applied due to the importance of the structure. The moment at the top due to rotatory inertia should be computed as

$$M_o = 1.5\ V_o r_o^2\ \theta_o/\delta_o$$

where V_o is the lateral force; r_o the radius of gyration of the mass with respect to a horizontal axis at the top of the structure, perpendicular to the direction of analysis; θ_o, the rotation at the upper end due to V_o and δ_o the horizontal displacement of this point due also to V_o.

As it was mentioned before, additional reductions might be obtained in the case of rigid structures on soft soil, therefore, the fundamental period of vibration was estimated using the following expression, which is a modification of that proposed in the Code to take into account rotational effects:

$$T = 6.3\ \left[(m\delta_1^2 + J\theta_1^2)/(V_o\delta_1 + M_o\theta_1)\right]^{1/2}$$

Here δ_1 and θ_1 are total displacements at the upper end due to the combined effect of V_o and M_o, m is the mass and J its polar moment of inertia.

Dynamic analysis. Three different models were considered for the dynamic analysis: cantilever column with mass concentrated at the top and perfectly fixed base, column with mass having rotatory inertia at the top and perfectly fixed base and column with mass having rotatory inertia at the top and soil-structure interaction at the base. Linear behavior was assumed using the models proposed in ref.2.

For the first model the moment at the upper end is zero and frequency is equal to the square root of m over k. For the second case, the frequencies are given by

$$w_{1,2}^2 = \frac{kJ + mk_r}{2\ KmJ} \pm \left(\frac{kJ + mk_r}{2KmJ} - \frac{k\ k_r - U^2}{K^2\ mJ}\right)^{1/2}.$$

where k is translational stiffness; k_r rotational stiffness; $K = 1-\delta\theta$; δ is the horizontal displacement at the top due to a moment k_r; Θ is the rotation at the top due to a horizontal force k; $U=\gamma k\ k_r$, γ is the rotation at the top due to a unit horizontal load or the lateral deformation due to a unit moment applied at the top.

Table 1 summarizes the above elastic properties of the column for both directions of analysis; table 2 shows values of m and J corresponding to the most adverse arrangement of live load.

Modal configurations for the second case are given by

$$x_{ij}/\epsilon_{ij} = {}^{k\delta}/K({}^{k}/K - mw_j^2)$$

where x and ϵ are total displacements and rotations.

The spectrum used corresponds to the soft soil of the city and is described by a = (0.06 + 0.225 T) g for T <0.8 sec; a = 0.24 g for 0.8 sec <T<3.3 sec; a = (0.792/T) g for T >3.3 sec. Where a, the spectral acceleration, may be reduced to compute seismic forces dividing by a ductility reduction factor Q = 2 for T>0.8 sec or by Q' = 1 +1.25 T for T<0.8 sec.

The fundamental period of the structure is smaller than 0.8 sec, therefore any increase in its value due to soil-structure interaction would increase the response and model 3 was necessary. As the structure is supported on friction piles the dynamic properties of the foundation are difficult to evaluate. As mentioned before the model used is described in ref. 2, it does not include the mass of foundation and adjacent soil. Stiffnesses in translation and rotation of the group of piles were computed using Hrennikof's method (ref. 3) Lateral stiffness computed is 21 000 ton/cm and rotational stiffness 3 200 000 ton-m/rad. The mechanical elements obtained by the three dynamic models are presented in table 3 for the least favorable load combination.

Comparison of results. It may be observed in table 3 that the moment computed by the static method is larger than that obtained with dynamic models 1 or 2, however, lateral force is larger in the dynamic model with soil-structure interaction and the moments at the base are larger than those computed statically.

Combination of effects in both directions leads to similar results in the static and dynamic analyses.

Research program and field tests. In order to evaluate the dynamic parameters of the structure a research project is underway at the writing of this paper. It includes free and forced vibration of beams and columns to measure effective modulus of elasticity, periods of vibration and soil-structure interaction effects, as well as theoretical studies to analyze more sophisticated models of soil-structure interaction and step by step analyses with typical accelerograms recorded on the soft-soil of Mexico City. Fig 2 shows a general view of the tests. Due to space limitations results of this research program may be described during the conference.

References.

1. Mexico City building code, 1976
2. Rascón, O.A. "Seismic effects on inverted pendulum structures" (in spanish). Rev. Soc. Mex. Ing. Sism., 1965
3. Bowles, J. Foundation Analysis and Design, McGraw Hill Book, Co., 1968.

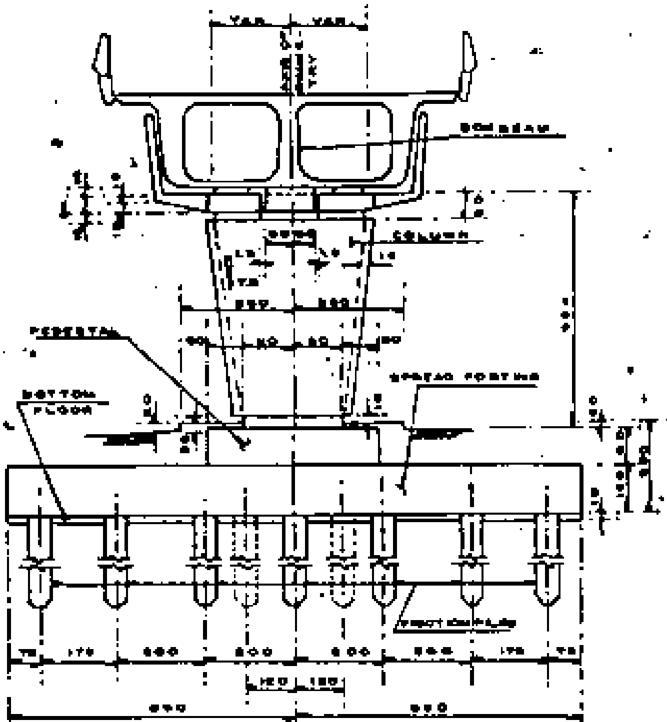

FIG. 1

TABLE 1

COLUMN PROPERTIES					
X DIRECTION			Z DIRECTION		
k	73214.37	ton./m	k	70684.7	ton./m
kr	848521.36	ton.m/rad	kr	692233.	ton.m/rad
β	3.158x10-6	1/ton.	β	3.515x10-6	1/ton.
θ	0.2311	rad./m	θ	0.24850	rad /m
j	3.63	m/rad.	j	3.14	m /rad.
K	0.160		K	0.220	
P	280760.47	ton.	P	221723.	ton.

TABLE 3

COMPARATIVE RESULTS						
ANALYZED MODEL		EARTHQUAKE DIRECTION	FORCES AND BENDING MOMENTS AT BASE OF COLUMN			
			V (ton.)	%	M (ton.-m)	%
STATIC ANALYSIS		X	128.27	100.0	1231.13	100.0
		Z	118.88	100.0	667.80	100.0
DYNAMIC ANALYSIS		X	117.91	93.4	641.44	52.1
		Z	118.42	99.4	630.08	94.4
		X	81.88	64.9	719.36	58.4
		Z	111.12	95.3	631.17	94.5
		X	138.59	109.7	926.16	75.2
		Z	143.71	123.3	791.83	118.6

TABLE 2

m AND J FOR ONE LOADING CONDITION							
EARTHQ. DIR.	MASS (ton.-seg²/m)				ROTATORY INERTIA (ton.-m-seg²)		
	GIRDER+ ADDLOAD	COLUMN	LIVE LOAD	TOTAL	GIRDER+ ADDLOAD	LIVE LOAD	TOTAL
X	76.46	7.01	22.11	105.58	746.41	566.83	1312.24
Z	76.45	7.01	20.04	103.51	61.92	17.91	79.83

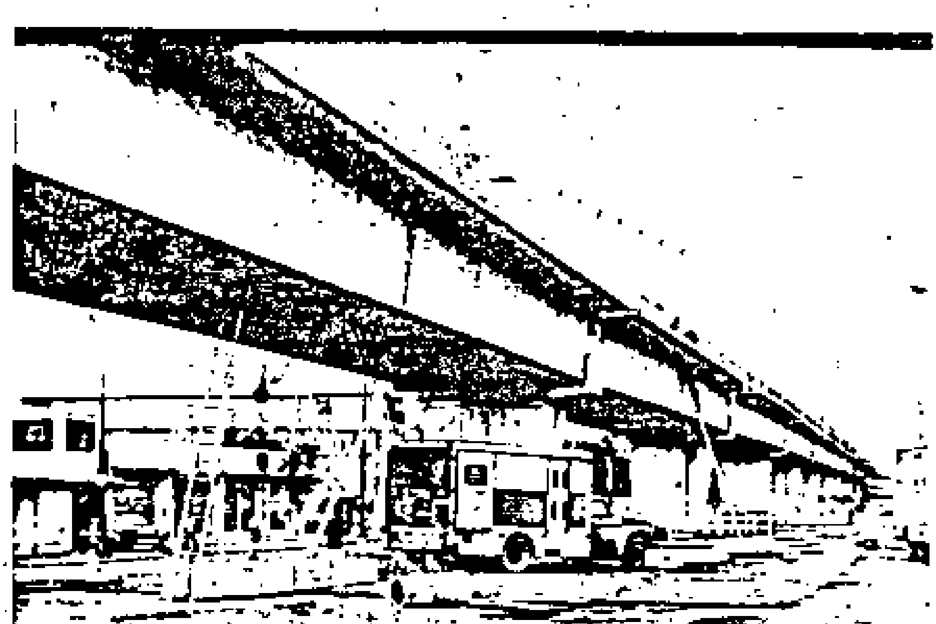

FIG. 2

EVALUATION OF ANALYTICAL PROCEDURES USED IN BRIDGE SEISMIC DESIGN PRACTICE

by

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Engineering Computer Corporation
R. V. Nutt, Senior Research Engineer
Engineering Computer Corporation
J. Penzien, Professor
University of California, Berkeley

INTRODUCTION

The accurate prediction of stresses and displacements induced in the various components of a structure during a strong motion earthquake is the key to improved earthquake resistant design. Predicting these stresses and displacements in bridge structures may be divided into the following two general tasks:

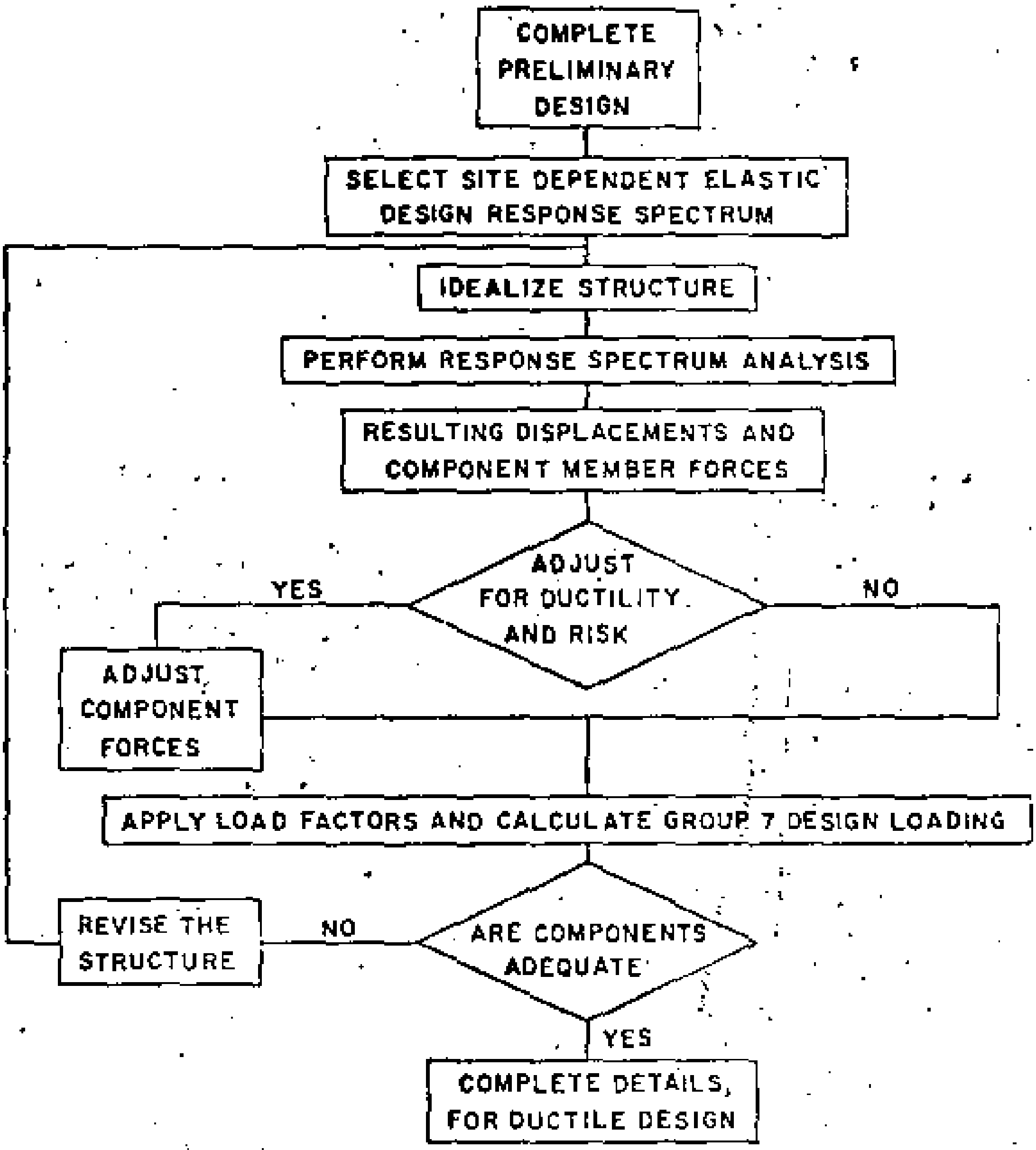
(1) Determination of the seismic load.
(2) Determination of the effect of this load on the structure.

These two tasks are typically reflected in current seismic design processes such as the one used at the Office of Structures, California Department of Transportation (CALTRANS). This process is depicted in Figure 1. The seismic load to which a structure will be subjected is determined by selecting the appropriate site dependent design response spectrum. The effect of this loading on the bridge structure is then determined by predicting the elastic response of the structure by any one of several methods, and reducing the elastically determined forces to account for the effects of structure yielding. Elastic displacements are generally considered to be equal to the actual displacements.

With the revolution in structural analysis brought on by the advent of modern digital computers, it may appear to the casual observer unfamiliar with structural dynamics, that the second task (i.e., predicting the effect of a given seismic loading) has evolved to a state which approaches an exact science. However, this is not the case. One of the primary reasons for this is the lack of field data on the actual magnitude of stresses and displacements occurring in bridges during a major earthquake.

In an effort to overcome, at least partially, this absence of data, a model structure was subjected to simulated earthquake loading on the shaking table at the University of California Richmond Field Station. Data gained from this experiment was correlated with results from a sophisticated research oriented computer program developed specifically to predict seismic response of bridge structures. This correlation study resulted in a substantial improvement in the algorithms used to calculate nonlinear response.

Many bridge designers do not have access to computer facilities and those that do must use programs that are less sophisticated than the one mentioned above. In practice, therefore, stresses and displacements are determined by more approximate means which employ several simplifying



CALTRANS
SEISMIC DESIGN PROCESS

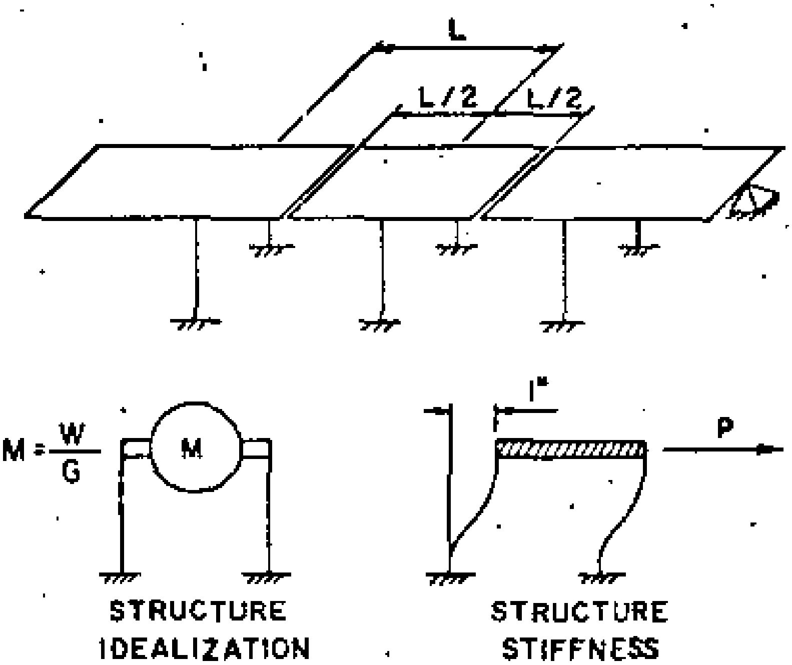
FIGURE 1

assumptions. With the present absence of field data, evaluation of these means can only be done by comparison with more sophisticated analytical approaches which are known to better model reality.

This paper deals with an evaluation of the currently used methods for predicting the response of bridge structures to a given seismic loading. An evaluation of both the equivalent static load and the response spectrum techniques for determining seismic effects on bridge structures is included. The experiences of the authors in their association with the University of California at Berkeley and the California Department of Transportation were drawn on to make this evaluation.

BACKGROUND

Prior to the San Fernando earthquake of 1971, bridges were generally designed for earthquake forces using an equivalent static force approach known as the Lollipop Method. In other words, the bridge bents were assumed to act independent of one another as single-degree-of-freedom oscillators with a lumped mass equivalent to the tributary deck mass as shown in Figure 2. Both structure period and load distribution were determined using this method.



"Lollipop" Idealization
Figure 2

Immediately following the earthquake, CALTRANS recognized the need to develop a more rational earthquake design procedure for bridges. Efforts were initiated to develop new earthquake design guidelines that would consider seismicity and the vibrational properties of both the bridge and the underlying soil. There were two basic approaches that evolved regarding the method that should be used to perform the seismic analysis for bridge design. Proponents of the first approach proposed that a simplified technique for applying equivalent, static force be devised that would allow the designer to use his present knowledge of the static behavior of structures to design the bridge. Those who favored the second approach, felt it was more desirable to train the bridge designer to perform more sophisticated analyses which more realistically considered the dynamic behavior of the structure.

The first approach required the development of an improved equivalent static force approach. It became evident to the CALTRANS engineer that the previously used Lollipop Method was not a realistic method of analysis. Efforts to find a simple but realistic method of applying an equivalent static force to a wide range of bridges resulted in the formulation of a uniform lateral load technique, known as the Uniform Load Method. This technique, which was the first attempt to revise the equivalent static force method, is still not totally satisfactory, however, in that it produces accurate results for only a limited number of bridge types.

At CALTRANS there were several factors that have made the second approach involving more sophisticated analysis the most desirable. Some of these factors are as follows:

(1) The unusual geometric alignments, support conditions, and restraints of many bridge structures on a modern highway system required more sophisticated three-dimensional mathematical idealizations to obtain realistic results.

(2) Sophisticated in-house computer capabilities were available with the required mathematical idealizations to perform a dynamic analysis.

(3) It was necessary to use the same computer program to perform a space frame analysis to effectively apply the Uniform Load Method as was required to perform a dynamic analysis. Thus with modest additional training, a more sophisticated analysis was possible at a relatively small additional effort and cost.

(4) There was a combination of: 1) willingness of management, 2) ability of bridge designers to learn new techniques, and 3) an availability of qualified personnel who were assigned to provide technical support on an ongoing basis.

This approach, which has proved successful at CALTRANS, resulted in the implementation of three-dimensional response spectrum modal analysis to determine design seismic forces for bridges on a routine basis.

The AASHTO Specification [1] for Bridges (1977) reflects the two approaches by specifying that the effect of seismic forces on bridges shall be evaluated by considering the dynamic response characteristics of the total bridge using one of the following methods:

(1) Equivalent static force
(2) Response spectrum dynamic analysis

For "special cases," the specifications recommended the use of dynamic analysis techniques. Special cases are considered to be structures with one or more of the following characteristics:

(1) Located adjacent to active fault(s)
(2) Located in area with unusual geologic conditions
(3) Unusual geometry, cost, importance, etc.
(4) Structure period greater than 3 seconds

These specifications were written following the San Fernando earthquake of 1971. They are to a very large degree the reaction of CALTRANS bridge design and research engineers to the failures that occurred during that earthquake.

The San Fernando earthquake also stimulated a renewed enthusiasm for additional theoretical and experimental studies into the seismic behavior of bridges. One of these studies, conducted at the University of California at Berkeley, was designed to investigate the effectiveness of existing bridge design methodology in providing adequate structural resistance to seismic disturbances. This project extended over approximately six years and included the following six phases:

(1) A review of the world's literature relating to seismic effects on highway bridges [2]

(2) An analytical investigation of the dynamic response of long, multiple span highway overcrossings [3]

(3) An analytical investigation of the dynamic response of short, single and multiple span highway overcrossings [4,5]

(4) Detailed model experiments on a shaking table to provide dynamic response data which could be used to verify theoretical response predictions [6]

(5) Correlation of experimental and theoretical response, and modification of analytical procedures as necessary [7]

(6) Preparation of recommendations for changes in seismic design specifications and methodology [8,9]

This project made substantial contributions to the advancement of the state of knowledge regarding the dynamic response analysis of bridge structures subjected to seismic loadings. As part of Phase 6 of this project, case studies were performed to evaluate the accuracy of results obtained from currently available computer analysis techniques. Of primary concern was the response spectrum technique that has gained wide use in bridge design. The results of these case studies provided the basis for the evaluation of response spectrum analysis presented in this paper.

10

EQUIVALENT STATIC FORCE METHODS

Introduction

The development of a realistic simplified equivalent static load approach for the dynamic analysis of bridges that would suffice for the final design of simple bridges and could even be used for preliminary design on the more complex bridges, is desirable for the following reasons:

(1) Simple extensions of what is currently used and would be easy to implement

(2) Does not require a computer

(3) Quick and easy to apply

The determination of seismic response by the equivalent static force method basically involves three steps:

(1) Calculating the period of the first mode of vibration in the direction under consideration.

(2) Obtaining the corresponding response coefficient "C".

(3) Distributing the resulting equivalent static earthquake force to the substructure elements.

Lollipop Method

In the past, the determination of the period and distribution of the earthquake force was accomplished by simply applying the formulas in the code. The idealization for the Lollipop Method implied the following simplifying assumptions about the dynamic behavior of a bridge:

(1) Each bent vibrates in its own natural period, independent of the other bents.

(2) The transverse bending and torsional stiffness of the superstructure do not contribute to the stiffness of the system.

There are several obvious over-simplified assumptions in this approach. Even for bridges of simple geometry, the assumptions were somewhat in error. The inaccuracies that occurred in the calculation of structural period resulted in unrealistic values for the equivalent static earthquake force. In addition, the distribution of this force was in error. The main advantage of this technique was that it was simple and easy to apply.

Uniform Load Method

To overcome the deficiencies in the Lollipop Method, an empirical approach, called the Uniform Load Method, was devised with the following objectives:

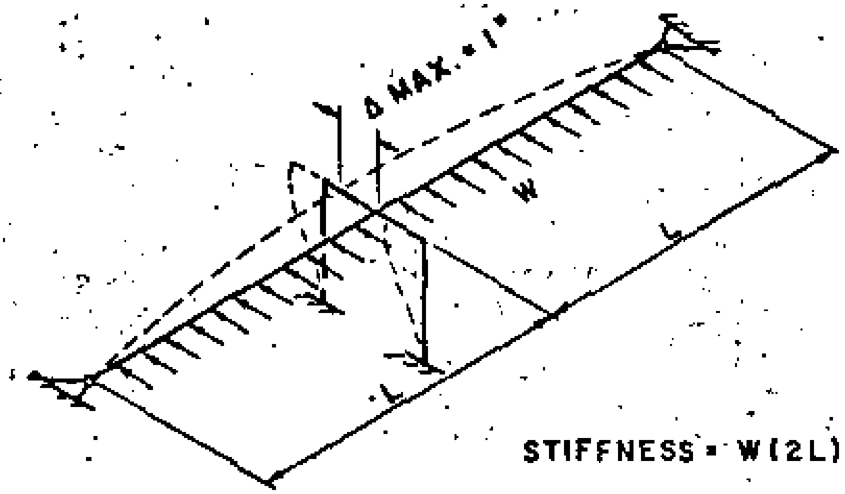
(1) Maintain continuity of the superstructure in determining the natural period of the system.

(2) Distribute the earthquake force to all of the participating elements of the bridge.

(3) Allow for ease of application using seismic design coefficients and static analysis techniques.

The steps in the Uniform Load Method approach can be summarized as follows:

(1) Apply a uniform horizontal load (usually taken as unity) to the structure in the direction of vibration as shown in Figure 3.



Uniform Load Idealization
Figure 3

(2) Perform a static analysis on the structure to determine the resulting displacements and member forces due to the applied uniform load.

(3) Adjust the maximum displacement to 1 inch. Using this adjustment factor, adjust the uniform load to correspond to a maximum displacement of 1 inch.

(4) Multiply the adjusted uniform load by the length of the structure. This is the value for stiffness which, along with the total dead load of the structure, can be used to compute the fundamental transverse period of the structure.

(5) Having obtained the period, determine the response coefficient "C" from the response curves.

(6) Determine the total earthquake force acting on the structure by combining the response coefficient with the framing factor and the total dead load.

(7) Convert the total earthquake force into an equivalent uniform load.

(8) To determine forces in the members due to this uniform earthquake loading, prorate the forces in the members from the original uniform loading applied to the structure.

The desirability of using a simple approach employing a seismic coefficient in a static analysis, rather than a complex dynamic analysis, has provided the impetus for implementing the Uniform Load Method. Recent experience has shown that this empirical approach gives accurate results for certain types of simple bridges, but it can require more effort than a response spectrum dynamic analysis. This is because the Uniform Load Method requires a space frame analysis for all but very simple structures to properly analyze the transverse stiffness of the columns interacting with the superstructure.

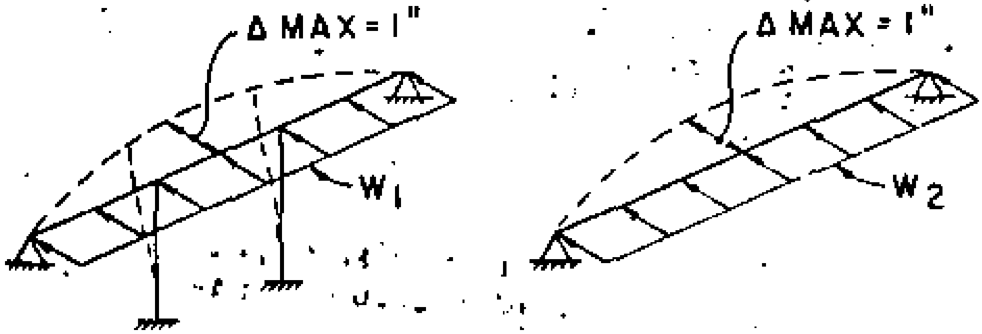
Several case studies [10] were performed to evaluate the accuracy and limitations of the Uniform Load Method as compared to a response spectrum dynamic analysis. For comparison, the Lollipop Method was also included in these case studies. In selecting bridges for these case studies, different structural and geometric characteristics were considered in order to evaluate the effect of the following parameters:

(1) Number of spans
(2) Ratio of span lengths
(3) Number of columns per bent
(4) Curvature
(5) Skew
(6) Structure width
(7) Column length and fixity

An attempt was made to categorize the types of structures which could be accurately analyzed by the Uniform Load Method. It was found that the single most important criterion for categorizing the structure was the relative stiffness between the superstructure and substructure. In order to quantify this criterion, a stiffness index was established.

The Stiffness Index relates the relative contribution of the columns to the transverse stiffness of the entire structure. As illustrated in Figure 4, the Index is found by taking the ratio of the transverse stiffness of the entire structure, including the columns, to the stiffness of the superstructure alone, acting as a simple beam.

Based on the cases considered, it was observed that the Uniform Load Method can yield accurate results for structures with certain characteristics. Continuous structures on a straight, non-skewed alignment could generally



$$\text{STIFFNESS INDEX} = \frac{W_1}{W_2}$$

Stiffness Index Definition
Figure 4

be analyzed using this approach provided the stiffness index was 2 or less. However, for structures with a stiffness index greater than 2, only those with balanced span lengths and equal column stiffnesses could be accurately analyzed. This method was not satisfactory for structures with skewed supports, intermediate hinges, or curved alignments.

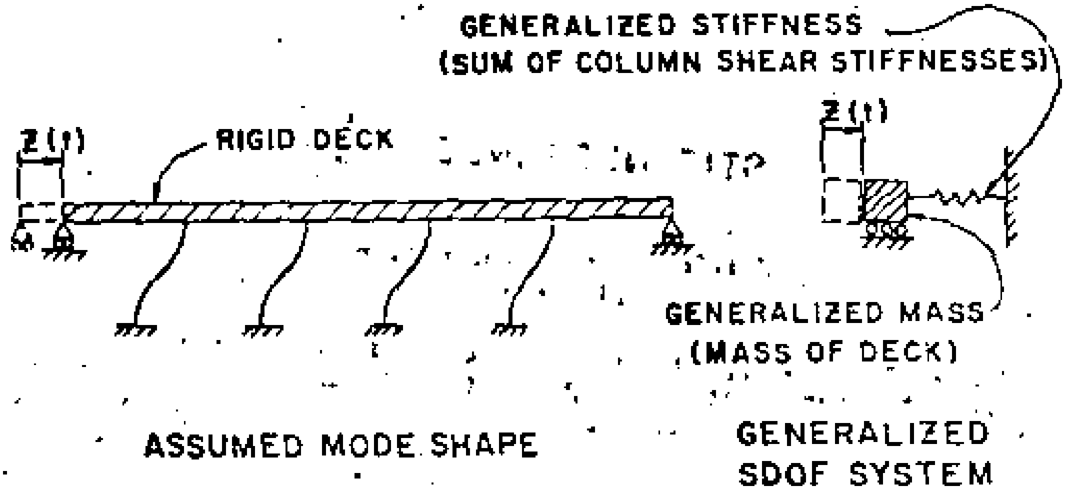
Since there are several limitations to the Uniform Load Method and since it generally requires a space frame analysis, there is a need to develop a simple but effective means for applying the equivalent static force approach to bridge structures.

In the development of an equivalent lateral force analysis procedure, it is necessary to determine the period of a structure and the distribution of the resulting lateral force. A reliable method for calculating the period must include the effective stiffness of the deck, restraining devices and soil springs, and the discontinuity of expansion joints, in addition to the individual column stiffnesses. In short, the true dynamic behavior of the bridge should be considered. The period should, if estimated, be an underestimated value to provide a conservative estimate of the equivalent lateral force. It is unlikely all bridge types will lend themselves to simplified techniques, but a large percentage of common types of bridges should be covered. Both longitudinal and transverse modes should be considered. Above all, the method should not require the use of a computer.

Generalized Coordinate Method

Another equivalent static force approach, that shows promise, can also be used to determine the period and earthquake response of certain types of bridges by applying energy principles to a generalized single-degree-of-freedom system. This method is based on the premise that the shape of the vibrating structure can be assumed and expressed mathematically in terms of a single generalized coordinate. The longitudinal and transverse modes of vibration can be separated into two classes of generalized single-degree-of-freedom systems.

For the longitudinal mode of vibration the structural displacement is characterized by the behavior of a rigid deck, limiting all the columns to equal longitudinal displacements as shown in Figure 5. This is the classical approach which has been used in the past to determine the longitudinal earthquake force for design.



Generalized Coordinate Approach
Longitudinal Mode
Figure 5

The transverse mode of vibration is more complex in that the transverse displacement of the columns are not all equal but rather are functions of their position along the superstructure as shown in Figures 6 and 7. In addition to this, the continuous superstructure will undergo bending and will thus make a contribution to the potential energy of the system.

The reliability of this method depends on the ability to predict and define the structure's mode shape. The effective application of this technique also requires that one mode dominate in each direction. Fortunately, many of the simpler bridges being designed today satisfy both of these requirements.

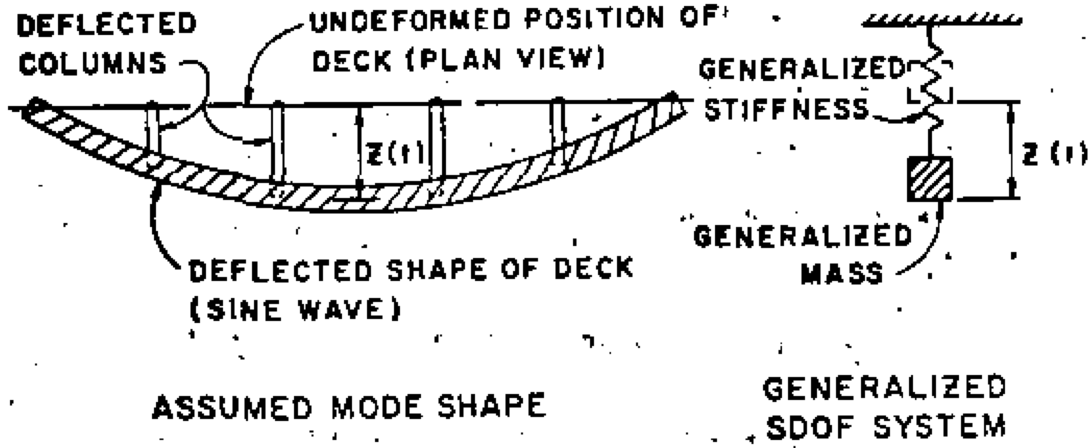
The method may be applied to girder deck bridge with no more than one intermediate hinge and having the following characteristics:

(1) Tangent or nearly tangent alignment

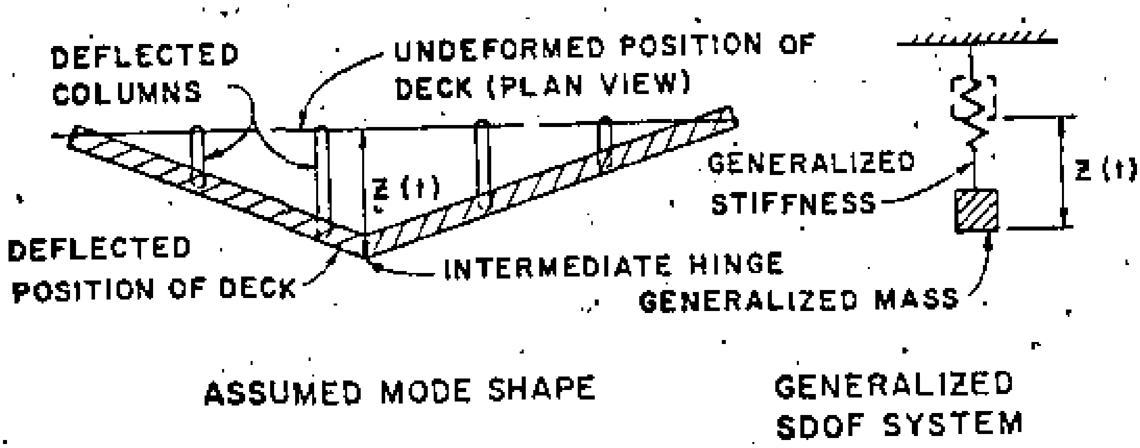
(2) Deck length to width ratio less than 15

(3) Skew angles of the abutments and supports less than twenty degrees

(4) Approximately uniform span lengths and column stiffness



Generalized Coordinate Approach
Transverse Mode (Continuous Deck)
Figure 6



Generalized Coordinate Approach
Transverse Mode (Intermediate Hinge)
Figure 7

The basic approach of the method is outlined in the following steps:

(1) Assume the predominate mode of vibration and define a generalized coordinate at the location of maximum displacement in the direction under consideration.

(2) Calculate virtual work done by external forces and internal member forces as the structure vibrates through a unit virtual displacement at the assumed generalized coordinate.

(3) Equate work to zero and solve for the structure period of the predominate mode in terms of the "Generalized Mass" and the "Generalized Stiffness".

(4) Determine the seismic coefficient from the appropriate response spectrum chart.

(5) Determine the earthquake excitation factor and scale the seismic coefficient.

(6) Determine the maximum generalized displacement.

(7) Determine the individual column forces using the generalized displacement calculated.

(8) Calculate member forces, apply ductility factors and design the member.

It should be noted that the first three steps given above are used only in the development of the formulas. The designer need not repeat these steps for each design since they are implied in the use of the formulas.

This approach was tested on several bridges which had previously been analyzed by the response spectrum technique. In most cases where this approach could be applied, the results compared well with those from the response spectrum analysis. In almost all cases, the comparison was better than was obtained using either the Uniform Load Method or the Lollipop Method.

Although the generalized coordinate approach to the equivalent static force method is not widely used, it appears to be a definite improvement over the other two methods.

THE RESPONSE SPECTRUM TECHNIQUE

Introduction

The response spectrum dynamic analysis procedure is indeed an improvement over the equivalent static force method. There are limits to its applicability, however.

The first shortcoming of the response spectrum approach is that the time domain has been removed. Since maximum modal responses do not occur simultaneously, it is necessary to use a statistical combination of modal responses such as root mean square in order to obtain realistic design loads. The actual combination of modal response depends on several factors related to the type of structure and the nature of the actual ground motion. Therefore, the use of a statistical approach to replace the effects of the removed time domain may not yield realistic results in certain cases.

Another deficiency in the response spectrum is that the duration of shaking is not accounted for by the spectrum. The major effect of duration will be on stiffness degradation and strength loss once the member begins yielding.

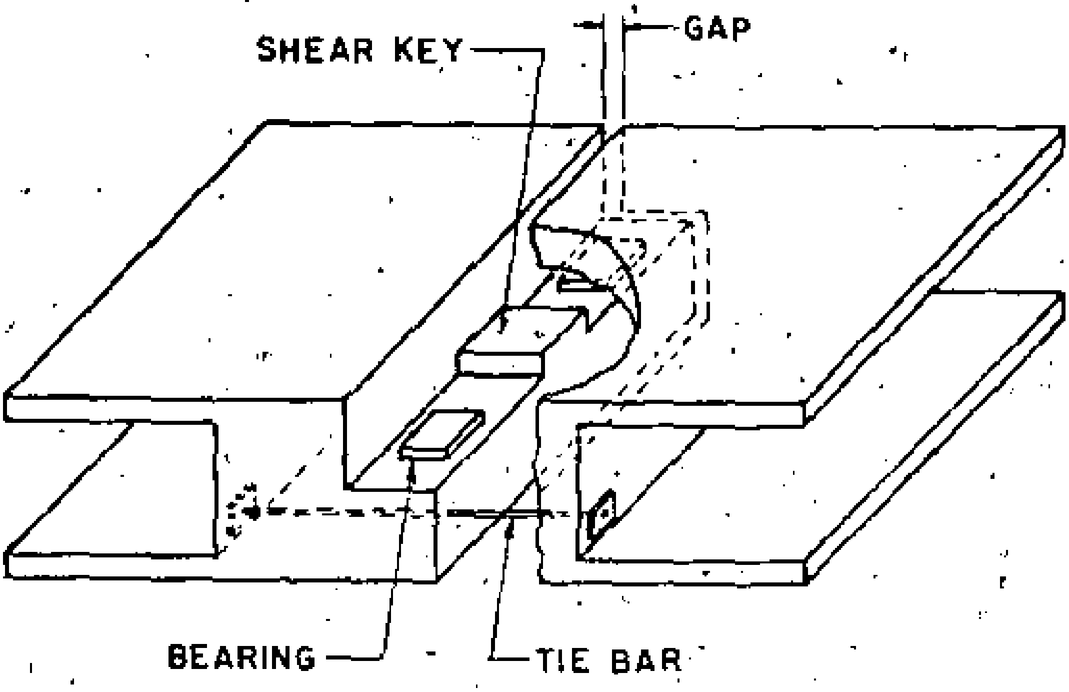
Since postelastic behavior is not specifically accounted for in the overall response analysis, a ductility factor or reduction factor is applied to reduce the forces obtained from a linear response spectrum analysis. This factor is applied either directly to the response spectrum or to the forces obtained from an unreduced spectrum. Because little is known about ductile behavior of bridges, the ductility factors used to determine the magnitude of reduction in bridge design have been extrapolated from research on building structures. Furthermore, the linear analysis does not account accurately for nonlinear behavior at expansion joint hinges, nor does it provide a means for assessing the redistribution of stress as yielding occurs in the ductile members. The analytical capabilities which evolved through the various phase of the University of California research project made it possible to evaluate the nonlinear behavior in the columns and expansion joint hinges. Recognizing both the limitations inherent in using elastic analysis techniques and the availability of improved analytical capabilities developed and refined during this research effort, case studies were conducted on three bridges to evaluate the analytical approaches currently used for seismic design of highway bridges.

The purpose of these case studies were to compare the results of a time history analysis that considers nonlinear behavior with results from both a linear time history and response spectrum analysis. Based on this comparison, the effectiveness of the current response spectrum approach as shown in Figure 1 can be evaluated.

<u>Properties of the Bridges</u>

Three bridges which were designed by the California Department of Transportation were selected for this study. All three structures consist of curved concrete box girder decks cast monolithically with single column bents. Because of the length of the bridges, each structure has one or more intermediate expansion joints to accommodate temperature movement.

This type of structure is common in California and is typically used in freeway interchanges. During the San Fernando earthquake of 1971, some of the most spectacular failures involved this type of bridge [2,11]. One of the primary cause of failure appeared to be the separation of expansion joint hinges. As a result, all structures of this type designed since the earthquake, including the three used in this study, have been fitted with restrainers designed to prevent separation. These restrainers must be gapped to allow freedom of movement for temperature, etc. A typical expansion joint hinge of this type is shown in Figure 8.



Typical Bridge Expansion Joint
Figure 8

In order to obtain a better understanding of the behavior of this type of bridge, each of the structures selected had a different fundamental period of vibration. A summary of some of the important properties of these bridges is shown in Table 1. These bridges are shown in Figure 9, 10, and 11.

Bridge No.	Spans Length (ft)	Spans No.	Curve Radius (ft)	Column Lengths (ft) Min.	Column Lengths (ft) Max.	Hinges No.	Hinges Span Location	Periods of the First 20 Modes (Sec) Max.	Periods of the First 20 Modes (Sec) Min.
1	694	6	600	24.3	26.3	1	3	.40	.07
2	1138	8	1075	25.1	49.4	1	5	1.11	.15
3	1410	9	1050	60.7	85.6	2	3,7	1.94	.21

Basic Characteristics of Bridges Selected
for Case Studies
Table 1

Methods of Analysis

The following three types of analyses were performed on each of the three bridges selected.

SUPERSTRUCTURE PROPERTIES

L = 694.0 FT

A = 83.7 FT^2

I_x = 814.1 FT^4

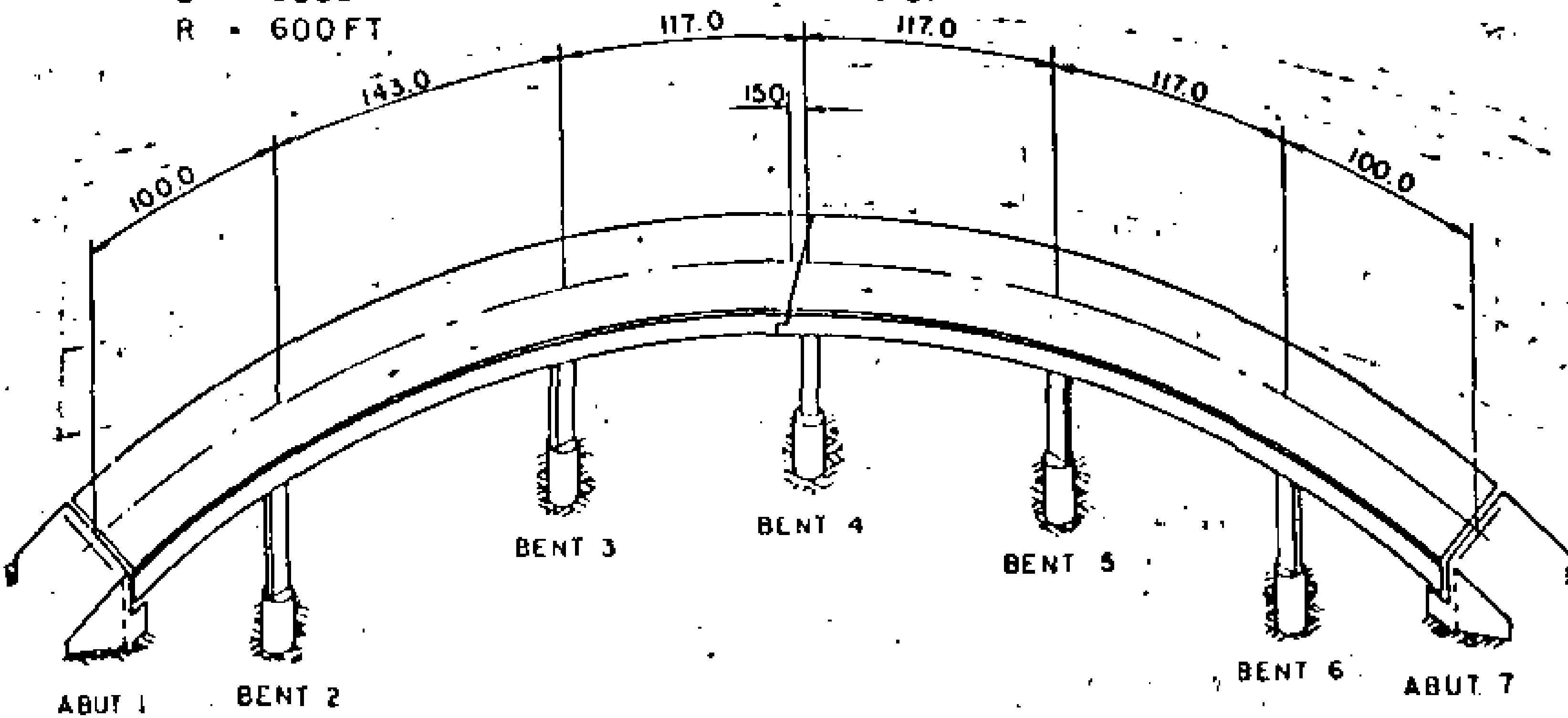
I_y = 353.7 FT^4

I_z = 12868.8 FT^4

DL = 12.56 $^{K/FT}$

E = 3000 ^{KSI}

R = 600 FT



BRIDGE 1

ROUTE 80 ON-RAMP OVERCROSSING

SUPERSTRUCTURE A

$L = 521.8^{FT}$

$A = 48.0^{FT^2}$

$I_x = 850.0^{FT^2}$

$I_y = 3130.5^{FT^4}$

$I_z = 381.4^{FT^4}$

$DL = 7.2^{K/FT}$

$E = 3000^{KSI}$

SUPERSTRUCTURE B

$L = 621.7^{FT}$

$A = 54.5^{FT^2}$

$I_x = 850.0^{FT^2}$

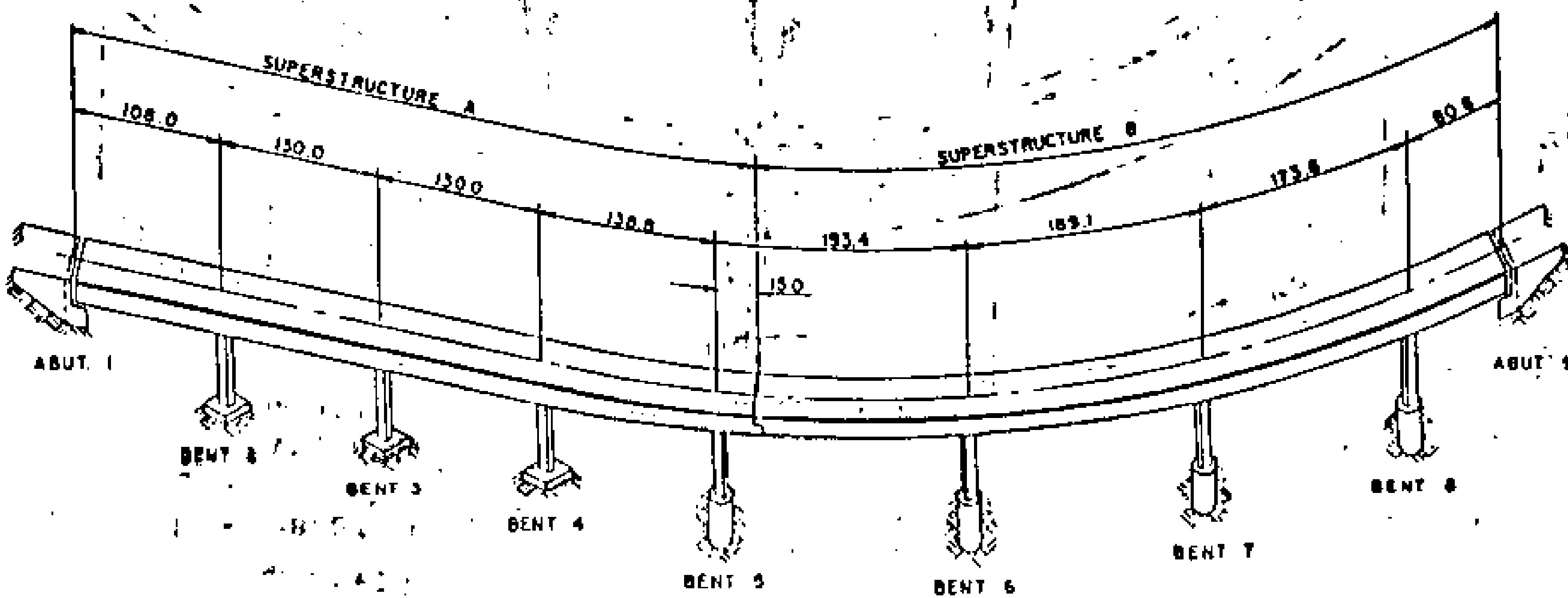
$I_y = 3528.3^{FT^4}$

$I_z = 406.5^{FT^4}$

$DL = 8.175^{K/FT}$

$E = 3000^{KSI}$

$R = 1083^{FT}$



BRIDGE 2

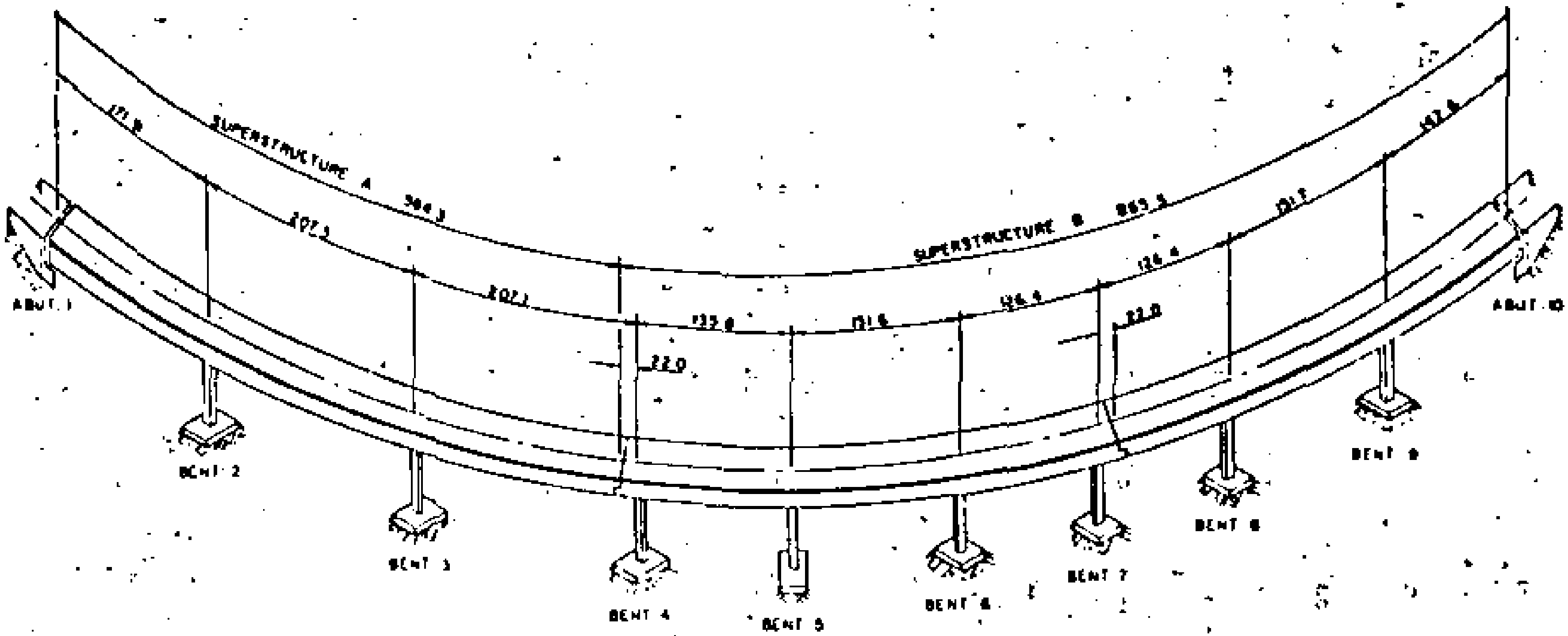
NORTHEAST CONNECTOR OVERCROSSING

SUPERSTRUCTURE A

$A = 81.3\ FT^2$
$I_x = 1000\ FT^4$
$I_y = 9713.5\ FT^4$
$I_z = 664.6\ FT^4$
$DL = 12.195^{K/FT}$
$E = 3000^{KSI}$
$R = 1062\ FT$

SUPERSTRUCTURE B

$A = 75.5\ FT^2$
$I_x = 980\ FT^4$
$I_y = 9152.9\ FT^4$
$I_z = 640.0\ FT^4$
$DL = 11.325^{K/FT}$
$E = 3000^{KSI}$
$R_{\mathbb{C}} = 1062\ FT$



BRIDGE 3
SOUTHWEST CONNECTOR OVERCROSSING

(1) A response spectrum modal analysis, which is the approach that was used at CALTRANS, and appeared to be the most desirable for general use in bridge design.

(2) A linear time history modal analysis, which includes consideration of the time domain but not the effects of nonlinear behavior.

(3) A nonlinear dynamic analysis, which employed a step-by-step integration technique and included the effects of both expansion joint and column nonlinearity.

The linear analysis capabilities of STRUDL (STRUctural Design Language) were used to perform the response spectrum and linear time history analyses [12]. STRUDL is a well-known general purpose computer program for static and dynamic analysis of linear structural systems. The MCAUTO proprietory version was used [13].

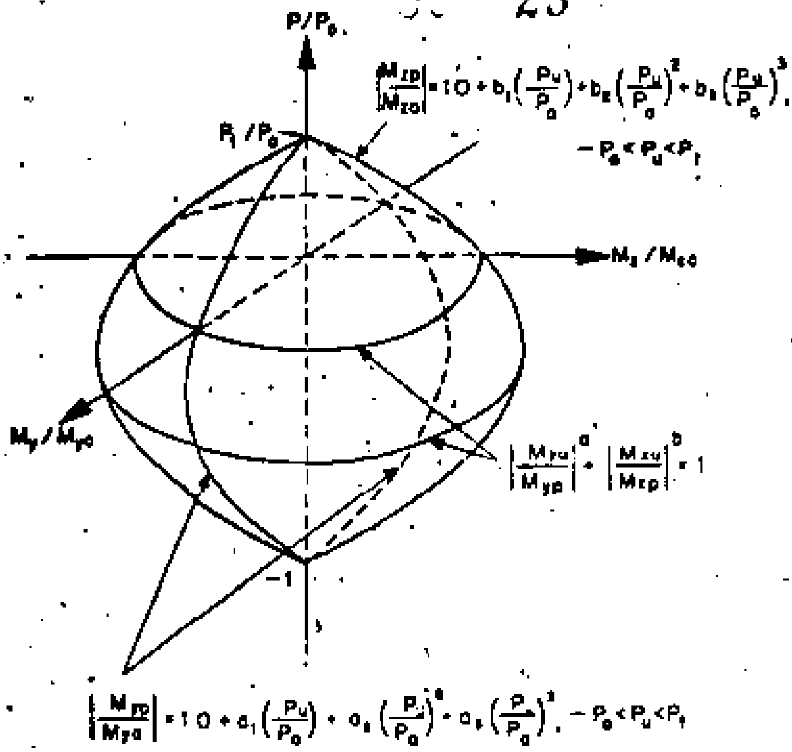
The nonlinear analysis was performed by the NEABS (Nonlinear Earthquake Analysis of Bridge Systems) program [3,7]. This computer program uses a step-by-step integration procedure which assumes piecewise linear behavior over each increment of time. The linear acceleration method was used for this study. Loading was input as rigid support accelerations. The program element library has the conventional linear elements plus the following nonlinear element types:

(1) Elasto-plastic straight beam elements
(2) Bi-linear boundary spring elements
(3) Nonlinear expansion joint elements

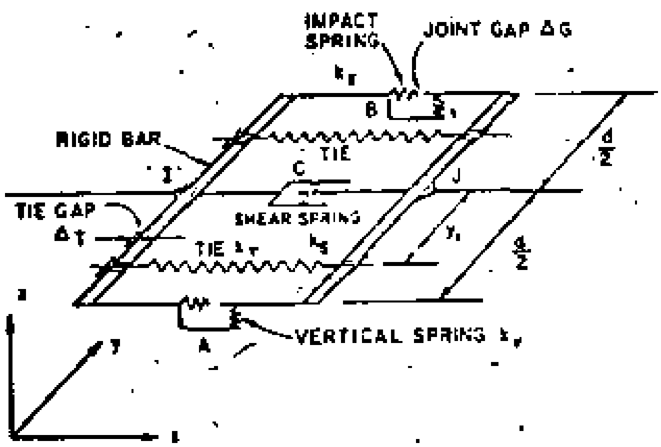
The two nonlinear parameters considered for this study were the yielding of the single column bents, and the nonlinearity of the expansion joint hinges.

The yielding of columns was limited to axial and flexural yielding along an interaction yield surface. The yield surface for a typical bridge column is shown in Figure 12. The ultimate capacity of the column in shear was considered to be infinite.

The nonlinear behavior of the expansion joint hinges were modeled using the expansion joint element shown in Figure 13. In this expansion joint hinge idealization, the restrainers were assumed inactive until movement at the joint was sufficient to take up the gaps which are normally placed in the restrainer anchorages to allow for normal movements of the joint. When the restrainers were active, they behaved in an ideally elasto-plastic manner. Relative movement at the hinge was limited by stiff impact springs which were activated upon closure of a seat gap. This represented banging of the two adjacent superstructure sections. The vertical and shear stiffnesses of the bearing pads were also included in the expansion joint element. Relative movement of the pads at the pad-concrete interface when the Coulomb friction force is overcome was also considered.



Yield Surface Description
Figure 12



Expansion Joint Idealization
Figure 13

Rigid support motion was used for all of the bridges. The SI 8+ time history ground motion developed by Seed and Idress [14] for a simulated 8+ Richter magnitude earthquake was used. The response spectrum for this motion, shown in Figure 14, was generated for 5 percent damping. This ground motion was applied to the bridges in the two orthogonal directions. The longitudinal and transverse motions were directed parallel and perpendicular to a line between the abutments.

With three types of analysis for each of the three bridges studied and ground motion in two directions, the total number of cases examined amounted to 18.

The bridge decks and columns were modeled with space frame members. Masses in the deck were lumped at the quarter points. Column masses were lumped at the third points. For simplicity, the base of each column was assumed fixed at the footing. The abutments were assumed to be free to move in the longitudinal direction. A typical structure idealization showing the location of lumped masses is shown for each bridge in Figures 15, 16 and 17.

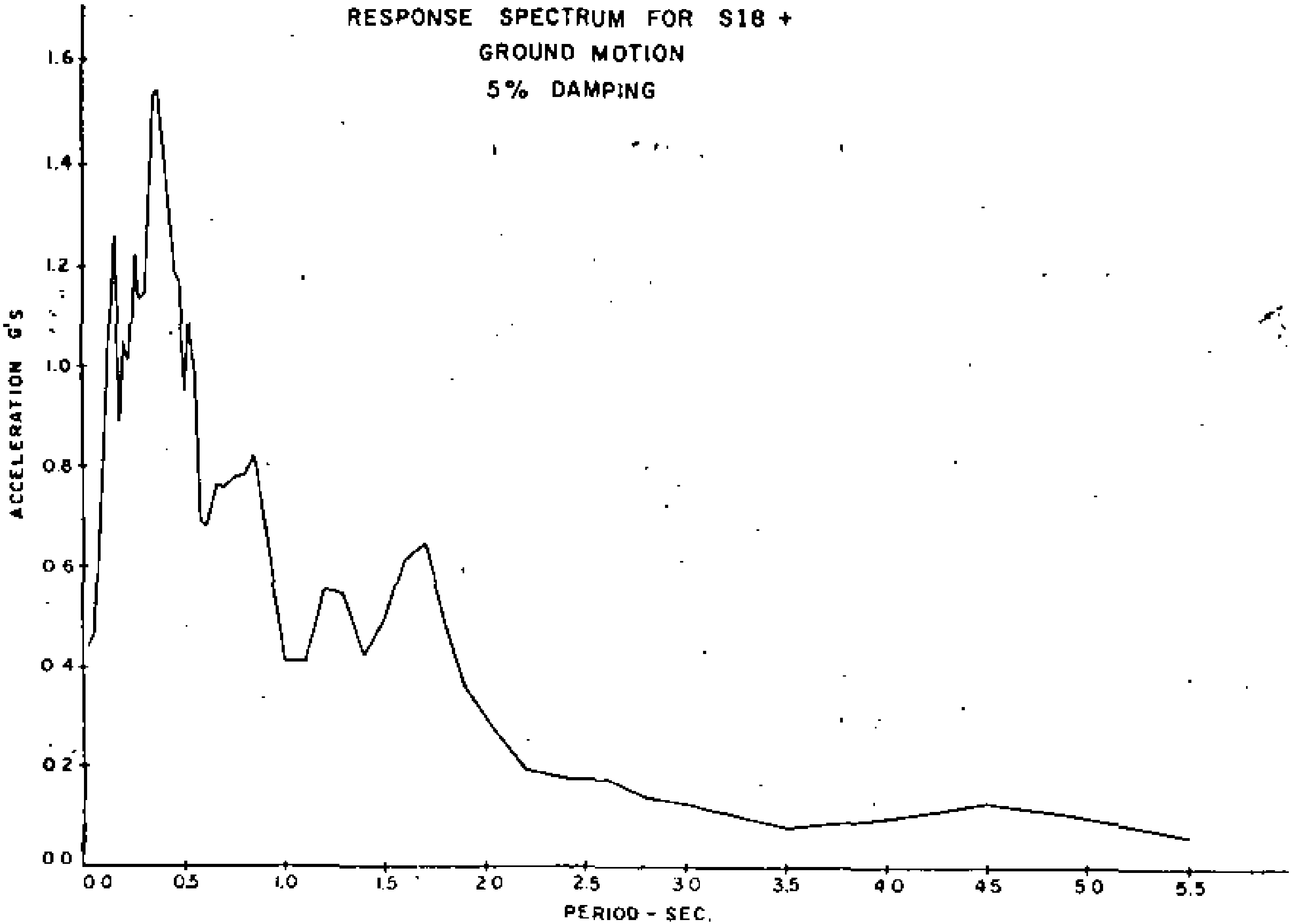
The hinge idealization for the elastic analyses was modeled by releasing main girder member axial forces, and superimposing transversely eccentric space frame members between both sections of the superstructure to account for the restrainers. This idealization assumes no gap and both tension and compression at the restrainers.

The expansion joint element used in the nonlinear analysis includes several parameters which more realistically describes the boundary conditions at the hinge. Design values shown on the plan drawings for tie and seat gaps were used. In actuality, these values will vary depending on such factors as temperature and shrinkage. Cable restrainer stiffnesses were calculated assuming an effective Young's modulus of 13,800 kips per square inch. The yield force in a typical 3/4 inch restrainer was taken as 30.6 kips. The shear stiffness of elastomeric bearing pads was calculated based on an assumed shear modulus of 135 psi. The coefficient of sliding friction for elastomeric pads on concrete was assumed to be 0.4. For lubricated sliding steel plates, the shear stiffness was assumed to be very high and the friction very low. For the purposes of modeling impacting of the superstructure, the impact spring was assumed to have the axial stiffness of the shortest adjacent section of superstructure.

Nonlinear column elements were used at locations where column yielding might be expected. Nonlinear columns were modeled on NEABS by mathematically describing the yield surface as shown in Figure 12.

Results

Modal participation factors indicated that all three structures had a tendency to respond in more than one mode. Also, because of the curved alignments, each of the bridges had some modes which included high participation in more than one global direction. This makes it likely that similar internal resisting forces will result due to seismic excitation in either global direction.


RESPONSE SPECTRUM FOR S18 +
GROUND MOTION
5% DAMPING
ACCELERATION G'S
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0.0
0.5
1.0
1.5
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3.0
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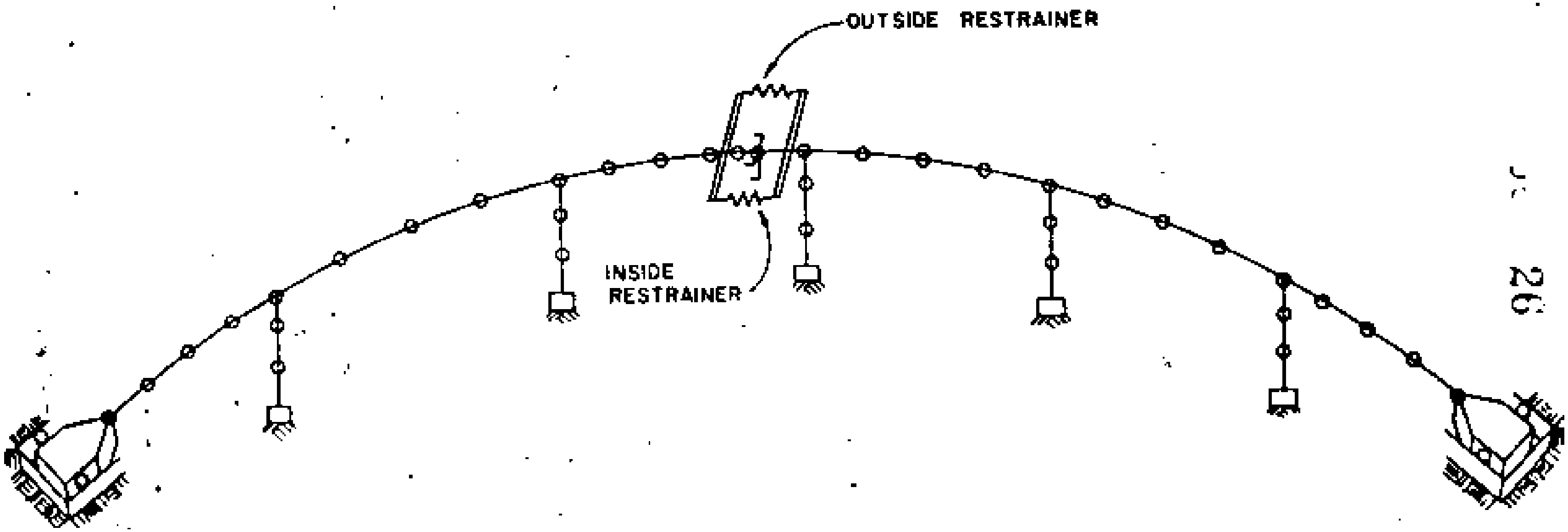
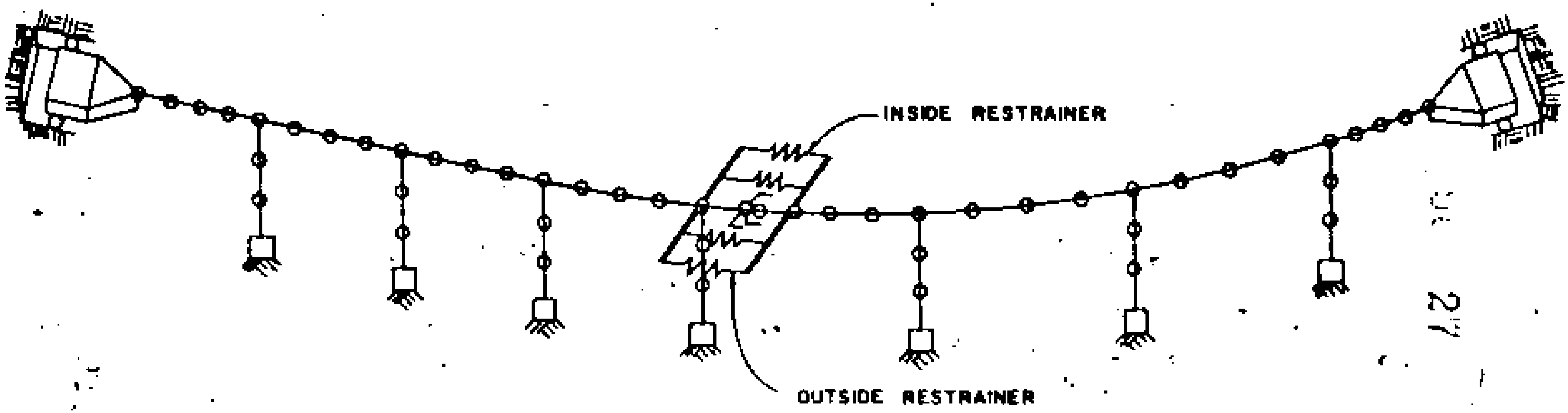
PERIOD - SEC.



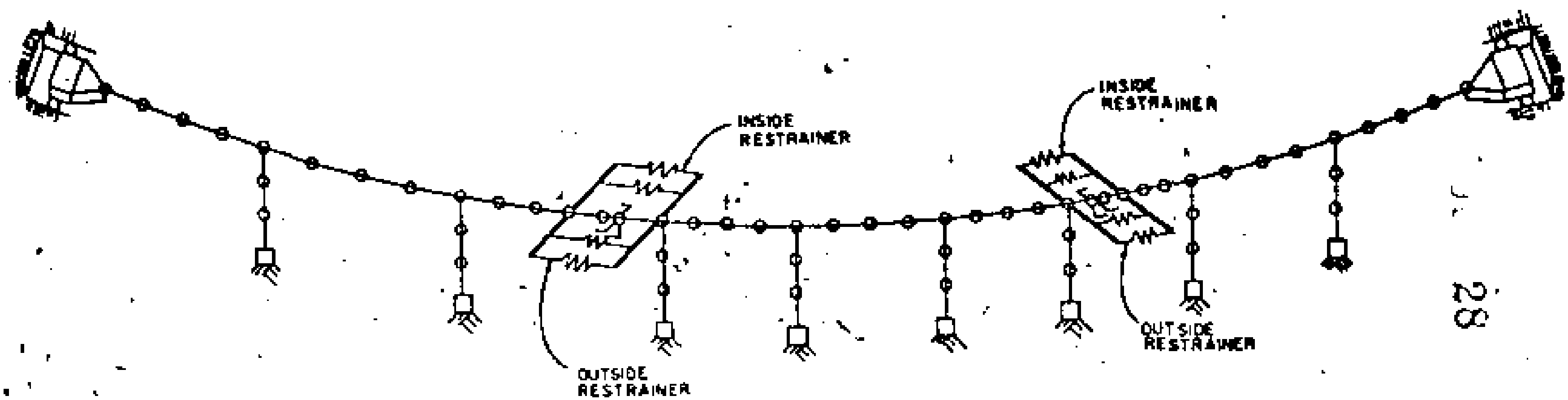
BRIDGE 1

ROUTE 80 ON-RAMP



BRIDGE 2

NORTHEAST CONNECTOR OVERCROSSING



BRIDGE 3

SOUTHWEST CONNECTOR OVERCROSSING

Current bridge design practice is to consider seismic excitation in each of the global directions separately. However, because of the possibility of simultaneous excitation in more than one global direction, and the sensitivity of certain internal force components to excitation from different directions, it would appear that earthquake resistant design would be improved by considering some simultaneous contribution from seismic loading in each of the global directions.

In the case of Bridge 1, the modal periods of the first few modes were very close, and occurred near the peak on the response spectrum for the ground motion used. This resulted in the in-phase modal contributions in the direction of ground motion. In the horizontal direction perpendicular to the ground motion, however, the tendency was for the modes to respond almost exactly out of phase. This was accounted for in both the linear and nonlinear time history analysis. The response spectrum analysis, however, which was based on a root-mean-square combination of modal response, yielded results that did not agree well with the time histories. This was more pronounced as indicated by forces resulting in the direction perpendicular to the ground motion.

Because of the high response of several modes in each of the bridges studied, it was found that a combination of modes that included the peak response plus the RMS of the remaining responses yielded results more in agreement with the linear time history in most cases.

The nonlinear time history analysis results indicated that significant column yielding could be expected in Bridges 1 and 2 while Bridge 3 would have experienced very little yielding. Since these bridges were designed to resist different intensity loadings, this was not considered to be significant.

Bridge 1, because of its lower fundamental period, was subjected to a considerable number of stress reversals that resulted in substantial yielding of the columns. Intuitively, from observing the time history of yielding for these columns, it would appear that a great deal of column degradation would have occurred. Yet the ductility demands, which were based on the maximum nonlinear column deformations, were well below the values considered to be available based on monotonic loading experiments. This points up an interesting deficiency in the current method of designing bridge columns. Based on the above observation, it would appear that short period structures would have a reduced available ductility in the columns due to the increased column degradation that would occur during the larger number of excursions into the nonlinear range. Not only is this not considered in applying a ductility reduction factor to column forces derived from an elastic analysis, but it is common practice to further reduce the forces in short period structures by a risk factor of 2. It would appear that this is just opposite to what should be done.

The nonlinear results for Bridge 2 yielded the highest single maximum column ductility demand of all three structures. The ductility demands in the remaining columns were not as high. It was interesting to note that the elastic moments from this earthquake were approximately double the yield moments. Therefore, had the normal ductility reduction factor been used to design the column for this seismic loading, the ductility demands would have been even higher. The reason for the high ductility demands in this single column, was the nonuniformity of column stiffness and yield moments which resulted in nonuniform yielding. The current practice of approximating non-linear behavior by applying a constant ductility reduction factor to an elastic analysis cannot predict this type of behavior.

The effect of large deadload moments was demonstrated in the nonlinear results for Bridge 2. Column yielding was more pronounced in the direction of high deadload moments. This resulted in a biased response that resulted in a tendency to relieve the deadload moments due to yielding. Since this would effect the distribution of normal service load moments and shears following an earthquake, this should be considered during design.

In all the transverse loading cases where column yielding occurred, the nonlinear analysis yielded seismic shear forces at the abutments that were greater than the linear time history analysis results. This is because the columns were incapable of carrying all the shear forces determined in the elastic analysis, and the excess was transferred through the deck to the abutments. This same phenominon was observed at the hinge in Bridge 2. This particular hinge was located near a stiff column that behaved similar to an abutment during an earthquake. In general, however, hinge shear key forces were slightly less in the nonlinear analysis.

The maximum deck displacements from the nonlinear analysis were almost always less than those from the elastic time history analysis. The exceptions to this were when localized maximum yielding occurred early in the earthquake, and when the deadload moments caused biased yielding as mentioned earlier. Classical methods of predicting nonlinear displacements based on equating strain energy from an elastic analysis to the sum of strain energy and energy dissipated in a yielded structure did not apply for these bridges.

It was obvious that because of reduced deck displacements and the normal gaps that are placed at the hinges to allow for free movement, that hinge restrainers were not stressed in the single hinge bridges. Stresses were developed in the restrainers in the two hinge bridge. The banging action that occurred between the adjacent sections of superstructure caused these forces to vary considerably from the elastic analysis, however. Currently, there appears to be no way of accurately predicting restrainer forces from an elastic analysis. The methods currently used seem to, at least for these bridges, yield conservative results.

CONCLUSIONS AND RECOMMENDATIONS

Based on the evaluation of the current methods for determining dynamic response to seismic loading, the following general recommendations can be made relative to the improvement of seismic design methodology for bridges:

(1) The Uniform Load Method for applying the equivalent static force approach to seismic design of bridges is not totally satisfactory. An improved method using energy principles should be further developed and implemented into the bridge design process.

(2) The response spectra currently used in the AASHTO specifications should be revised so as not to include the reduction for ductility. Ductility reductions should be made on an individual component basis.

(3) Seismic design provisions should consider the simultaneous application of earthquake motion in the three component directions since there is in many types of bridges coupling between the component directions within each mode of vibration.

(4) The PRMS (i.e., peak plus RMS of the remaining) combination of modal contributions resulting from a response spectrum analysis is an improvement for certain bridges analyzed by the response spectrum technique and may potentially be used for bridges having two modes of vibration with approximately equal periods.

(5) Seismic design provisions should establish some threshold of yielding for moderate earthquakes expected to occur several times during the expected life of the bridge. The need for this aspect of seismic design becomes more prevalent when consideration is given to the unequal distribution of ductility demands in a structure having non-uniform column stiffnesses.

(6) The number and levels of inelastic excursions which take place in reinforced concrete columns during a maximum credible earthquake should be such that stiffness and strength degradations are minimal. This control is accomplished by proper design and detailing of reinforcement.

(7) The seismic design should provide for an increase of approximately 1.5 to 2 in the forces at the abutments derived from an elastic analysis if yielding in the columns is anticipated.

(8) Design provision for combining girder moment due to dead and live-loads should include the effects of deadload moment redistribution due to possible relief of deadload moments at the location of a plastic hinge in the column during an earthquake.

(9) The use of intermediate hinges should be avoided if possible in bridges located in areas of high seismicity.

(10) Nonlinear computer capabilities should be made more user oriented for the practicing engineer and should be disseminated to the engineering profession so that they can be used to:

(a) Make parameter studies of the seismic nonlinear behavior of bridges

(b) Develop more realistic seismic design code provisions

(c) Apply nonlinear analysis as a design tool for complex bridges

The questions raised during the course of this evaluation indicate the need for future studies to perfect analytical capabilities for predicting seismic response. Some of the areas that need particular attention are as follows:

(1) Stiffness and Strength Degradation - The possibility of occurrence and the effects of stiffness and strength degradations in reinforced concrete columns on nonlinear dynamic response should be considered.

(2) Energy Absorption - The important role of inelastic energy absorption in the columns and expansion joint restrainers should be studied further.. Special attention should be given to developing a clearer understanding of the concept of ductility and how it relates to bridge design so that elastic analysis techniques may be used with a greater degree of confidence by the bridge engineer.

(3) Restrainer Units - Non-uniform yielding and ductility demands in columns result in larger forces at the restrainer units for bridges with more than one intermediate hinge. These effects should be studied further to investigate the current minimum specification in the code and to determine if elastic analysis techniques currently used can predict these restrainer forces.

(4) Response Spectrum Analysis - Special studies to improve the results gained from a response spectrum analysis are needed. The determination of the most effective means of combining modal results for a particular bridge is especially needed.

(5) Equivalent Static Force - Additional studies should be made to better define the degree of applicability of the generalized coordinate approach to the simplified equivalent static force method for the seismic analysis of bridges.

A computer capability such as NEABS represents a powerful research tool. It may be effectively used for studying special problems related to bridge design and analysis, and for analyzing bridge response due to past and future earthquakes. Because of its potential for advancing the state of knowledge, these computer capabilities should be made more user oriented to provide researchers and engineers with effective means for analytically studying bridge seismic behavior.

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EARTHQUAKE-RESISTANT DESIGN OF BRIDGES

CONTENTS

BRIDGE AND STRUCTURAL COMMITTEE,
JAPAN SOCIETY OF CIVIL ENGINEERS

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1. INTRODUCTION

This is an almost completely revised version of "Earthquake Engineering of Bridges" contained in the fourth edition of this book published in 1973. The purpose here is to briefly summarize the current earthquake provisions for the design of bridge structures in Japan and to show some of the typical examples of the earthquake-resistant design of Japanese bridges.

There have been a number of revisions made for earthquake provisions for the design of bridges, which can be clearly seen in the past four editions of this book (1960, 1964, 1968 and 1973). As the concept of the aseismic design of bridges became more clear in recent years, there has been an effort to establish a unified earthquake-resistant code. The establishment of the "Specifications for the Earthquake-Resistant Design of Highway Bridges" in 1971 is one of the results of such efforts. The content of this edition is intended to be more concisely and systematically presented as compared with those of the earlier editions which had to inevitably include different clauses from different codes belonging to a number of organizations. Also in this edition, considerable part of "examples" of earthquake resistant design was replaced by more recent ones.

It may be worth noting here that a good earthquake-resistant design of a bridge is not that of super-structure or sub-structure alone. The most economical and rational aseismic design as a whole bridge system is only achieved when due consideration is paid for topographical, geological and soil conditions of the site.

2. PROVISIONS FOR EARTHQUAKE-RESISTANT DESIGN OF BRIDGES

2.1 SYSTEM OF CURRENT SPECIFICATIONS.

There used to be a number of different earthquake provisions specified by various concerned organizations such as the Japanese Highway Public Corporation and the Tokyo Expressway Public Corporation. However, since 1971 when the "Specifications for the Earthquake-Resistant Design of Highway Bridges" was issued by the Japan Road Association, this superseded the provisions previously found in different codes as far as the aseismic design of highway bridges is concerned. This specifications was established in order to give a common basis for the design methodology and, therefore, places emphasis on the method of evaluating seismic forces, the basic principles to be exercized for treating soil conditions at the site, and the general provisions to be observed for structural details in the earthquake-resistant design of highway bridges. It does not include clauses explicitly describing the design procedures for substructures of highway bridges, which are currently designed according to the "Specifications for the Design of Sub-Structures of Highway Bridges" also issued by the Japan Road Association.

Parallel with the specifications for highway bridges mentioned above, the Japanese National Railways has its own system of design specifications for the structures under the jurisdiction of JNR. There also are cases in which special specifications are created for the

earthquake-resistant design of structures in important, large-scaled projects. The "Specifications for Earthquake-Resistant Design of Honshu-Shikoku Bridges" is a typical example, which was established primarily for the aseismic design of long-span suspension bridges having large foundations under the sea.

Since the general philosophies behind different earthquake provisions are similar, the specifications for highway bridges will be referred to in some of the following sections (2.2, 2.3, 2.4, 2.8 and 2.9), and those for the JNR bridges in Sections 2.5, 2.6, 2.10, 2.11, 2.12, and 2.13.

The specifications which will be frequently mentioned in the following sections are listed below with their abbreviations.

SEHB "Specifications for the Earthquake-Resistant Design of Highway Bridges (1971)", Japan Road Association.

SSHB "Specifications for the Design of Sub-Structures of Highway Bridges", Japan Road Association.

- Survey and Design in General (1966)
- Design of Pile Foundations (1964)
- Design of Spread Footing Foundations (1968)
- Design of Piers and Abutments (1968)
- Design of Caisson Foundations (1970)

SRS "Specifications for the Design of Railway Structures", Japanese National Railways.

- Foundation Structures and Structures Resisting Earth Pressure (1974)

2.2. SEISMIC COEFFICIENT METHOD.

For the design of highway bridges, SEHB specifies the seismic coefficient method if a structure has the fundamental natural period shorter than 0.5 seconds. The horizontal design seismic coefficient k_h in this case is obtained by the following formula,

$$k_h = \nu_1 \nu_2 \nu_3 k_0 \quad \cdots\cdots \quad (1)$$

where k_h: Horizontal design seismic coefficient,
k_0: Standard horizontal design seismic coefficient (= 0.2),
ν_1: Seismic zone factor,
ν_2: Ground condition factor, and
ν_3: Importance factor.

The factors ν_1, ν_2 and ν_3 are given in Fig. 1 and Tables 1, 2 and 3. The coefficient k_h is

Table 1. Seismic Zone Factor ν_1 (refer to Fig. 1).

Zone	Value of ν_1
A	1.00
B	0.85
C	0.70

Table 2. Ground Condition Factor ν_2

Group	Definitions*1	Value of ν_2
1	(1) Ground of the Tertiary era or older (defined as bedrock hereafter) (2) Diluvial layer*2 with depth less than 10 meters above bedrock	0.9
2	(1) Diluvial layer*2 with depth greater than 10 meters above bedrock (2) Alluvial layer*3 with depth less than 10 meters above bedrock	1.0
3	Alluvial layer*3 with depth less than 25 meters, which has soft layer*4 with depth less than 5 meters	1.1
4	Other than the above	1.2

Notes: *1 Since these definitions are not very comprehensive, the classification of ground condition shall be made with adequate consideration of the bridge site.
Depth of layer indicated here shall be measured from the actual ground surface.
*2 Dense alluvial layers such as dense sandy layer, gravel layer, or cobble layer are included under this category.
*3 Alluvial layer includes a new sedimentary layer made by landslide.
*4 Layers whose bearing capacities should be ignored as will be described in Section 2.4.

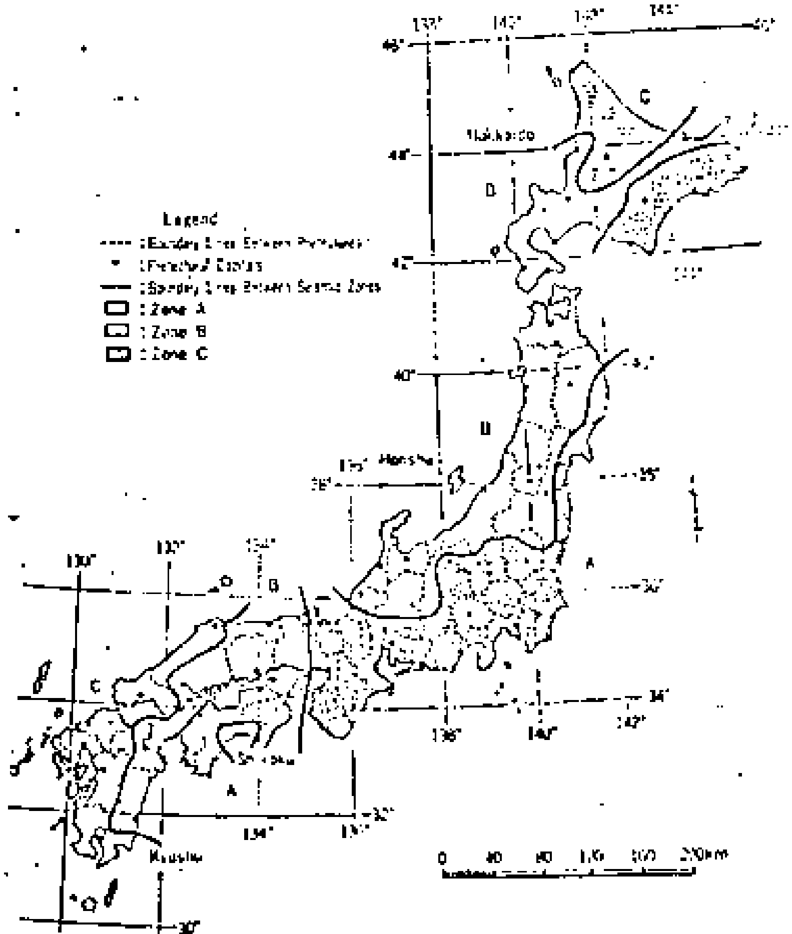


Fig. 1. Seismic Zoning Map.

Table 3. Importance Factor ν_3

Group	Definitions	Value of ν_3
1	Bridges on expressways (limited access highways), general national highways and principal prefectural highways Important bridges on general prefectural highways and municipal highways	1.0*
2	Other than the above	0.8

Note:* The value of ν_3 may be increased up to 1.25 for special cases in Group 1.

rounded to two decimals with the minimum value of 0.10. The maximum horizontal design seismic coefficient is 0.24 for most cases but may be increased to 0.30 as seen from the footnote of Table 3. The vertical seismic force is not considered in general except for the design of bearing at the junction of super- and sub-structures. In the latter case, the vertical design seismic coefficient of $k_v = 0.1$ is used.

For the structures under the jurisdiction of JNR, SRS specifies the basic horizontal seismic coefficient of 0.15 or 0.2 according to the zone classification, which is modified by considering the ground condition of the site and the importance of the structure. The minimum and the maximum value of the horizontal seismic coefficient are 0.11 and 0.27, respectively, and the vertical seismic coefficient of 0.10 should be taken into account.

2.3 MODIFIED SEISMIC COEFFICIENT METHOD CONSIDERING DYNAMIC STRUCTURAL RESPONSE.

When a bridge is supported on tall and flexible piers and its fundamental natural period is relatively long, the conventional seismic coefficient method usually becomes inadequate. For a highway bridge with the fundamental natural period of the super- and sub structure system longer than 0.5 seconds, SEHB specifies that the seismic coefficient be modified by considering dynamic structural response:

$$k_{hm} = \beta k_h \qquad \cdots\cdots (2)$$

where k_{hm} is the design horizontal seismic coefficient, k_h is the quantity as described in

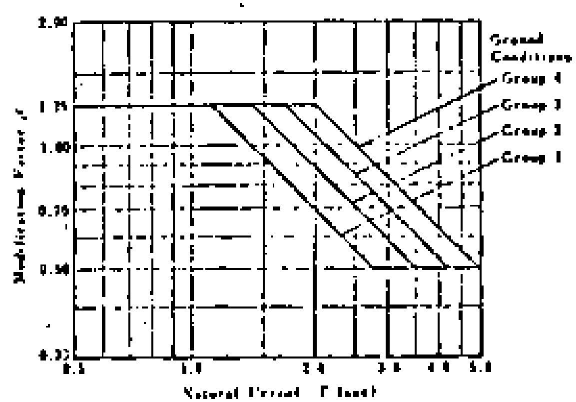


Fig. 2. Modification Factor β (refer to Table 4).

Section 2.2, and the modification factor β which depends on the fundamental natural period and the ground condition of the site is to be obtained from Fig. 2 and Table 4. The value of k_{hm} is also rounded to two decimals and the minimum value is 0.05. It should be noted that the natural period here is that of the whole bridge system consisting of super- and sub-structures and that the modified seismic coefficient k_{hm} should not be used for the design of the super-structures of suspension bridges, arch-type or cantilever-type bridges in which flexible super-structures result in long natural periods.

Table 4. Value of Modification Factor β.

Group of Ground Conditions Given in Table 3	Value of β for Fundamental Period T (sec.)		
1	$\beta=1.25$ for $0.5\leq T\leq 1.1$	$\beta=1.40/T$ for $1.1\leq T\leq 2.8$	$\beta=0.50$ for $T\geq 2.8$
2	$\beta=1.25$ for $0.5\leq T\leq 1.4$	$\beta=1.75/T$ for $1.4\leq T\leq 3.5$	$\beta=0.50$ for $T\geq 3.5$
3	$\beta=1.25$ for $0.5\leq T\leq 1.7$	$\beta=2.10/T$ for $1.7\leq T\leq 4.2$	$\beta=0.50$ for $T\geq 4.2$
4	$\beta=1.25$ for $0.5\leq T\leq 2.0$	$\beta=2.50/T$ for $2.0\leq T\leq 5.0$	$\beta=0.50$ for $T\geq 5.0$

Table 5. Formulas for Fundamental Periods of Bridges Supported by Spread Foundations or Pile Foundations

Type of Structural System		Direction	Formulas for Fundamental Periods — Pier Material: Reinforced Concrete	Formulas for Fundamental Periods — Pier Material: Steel
1	Most of super-structures are of continuous type with fixed supports on most of piers. If devices for limiting excessive displacement are installed, movable supports may be considered as fixed. Both abutments are rigid and at least one end of the bridge has fixed support on either of them, as shown in Fig. 3.	Transverse	$T=2\pi\sqrt{\frac{0.2W_P+W_U}{3EIg}h^3}$ (A-1)	$T=2\pi\sqrt{\frac{0.3W_P+W_U}{4.5EIg}h^3}$ (A-2)*
		Longitudinal	$T=\frac{2}{\pi}\sqrt{\frac{W_P}{EIg}}h^2$ (A-3)	
2	Other than the above. For example, a bridge consisting of girders with both ends simply supported.	Longitudinal or Transverse	$T=2\pi\sqrt{\frac{0.3W_P+W_U}{3EIg}h^3}$ (A-4)	

Notes: T : Fundamental period in second of the system consisting of a sub-structure and the section of super-structures supported by it.
W_P: Weight of the pier in t.
W_U: Weight of the section of super-structures in t supported by the sub-structure under consideration.
E : Young's modulus of the pier material in t/m^2.
I : Moment of inertia of the pier in m^4 in the direction considered (average value when cross-section varies with height).
h : Height of the pier in m, and
g : Acceleration of gravity (= 9.8 m/sec^2).
* Eq. (A-2) shall be used when the ratio of the length between the supports, L, on both abutments to the width, b, between outside girders is less than approximately 10 (see Fig. 3).

sidering each part of the bridge consisting of a pier and the super-structures supported by it. Therefore, if the bridge is supported by piers with different heights, the modified design seismic coefficient differs for each pier. The fundamental natural period of the bridge supported by spread or pile foundations is obtained by the formulas given in Table 5 and that of the bridge supported by caisson foundation by the formulas in Table 6. Since the formulas in Table 5 are obtained by assuming that 1) the base of the footing is below the surface of the hard supporting ground (the design ground surface described in Section 2.4) and 2) the elastic deformation of the pier above the footing has the primary effect on the overall deformation of the sub-structure, they should not be used if the base of the footing is above the design ground surface.

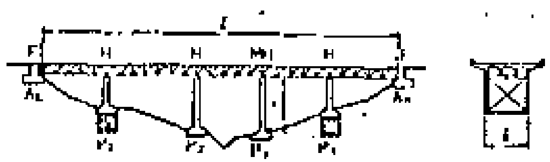
Table 6. Formulas for Fundamental Periods of Bridges Supported by Caisson Foundations

Type of Structural System		Direction	Formulas for Fundamental Periods
1	Type 1 in Table 5	Transverse	One of Eqs. (A-1) [illegible] (A-2) and (A-5)* that gives the largest value of β
		Longitudinal	Eq. (A-3)
2	Type 2 in Table 5	Transverse	One of Eqs. (A-4) and (A-5)* that gives the largest value of β
		Longitudinal	

* Eq. (A-5) is as follows:

$$T=2\pi\sqrt{\frac{\left(h+\frac{2}{3}l\right)^2 W_u+\left(\frac{1}{3}h^2+\frac{2}{3}hl+\frac{4}{9}l^2\right)W'_p+\frac{l^2}{9}W_c}{g\left\{K_H\frac{bl^3}{36}+K_V\frac{Al^2}{4}+K_S I_B\right\}}} \quad \text{(A-5)}$$

where
T : Fundamental period in second of the system consisting of a sub-structure and the section of the super-structures supported by it.
W'_p : Weight of the pier in t.
W_u : Weight of the part of super-structures in t supported by the sub-structure under consideration.
W_c : Weight of the caisson foundation in t.
h : Height of pier in m.
A : Cross-sectional area in m^2 at the base of the caisson foundation.
I_B : Moment of inertia in m^4 at the base of the caisson foundation in the direction considered.
K_H : Horizontal coefficient of subgrade reaction in t/m^3 at the level of the base of the caisson foundation.
K_V : Vertical coefficient of subgrade reaction in t/m^3 at the base of the caisson foundation, and
K_S : Horizontal coefficient of subgrade reaction in t/m^3 for shear deformation at the base of the caisson foundation.



F: Fixed Support
M: Movable Support
H: Hinged Support

Fig. 3. An Example of Type 1 Structural System in Table 5.

2.4. DESIGN GROUND SURFACE.

SSBB specifies the soil layers whose bearing capacities are neglected in earthquake resistant design. The design ground surface is defined as the level of the lower boundary of the neglected layer if it extends continuously below the actual ground surface. The neglected layer is assumed to have zero cohesion and zero angle of internal friction but its surcharge effect should be taken into account for the examination of the bearing capacity of the soil below the footing. It is not necessary to apply seismic forces to the structural parts, soils and water below the design ground surface nor to consider the seismic earth pressure to the sub-structure from the neglected layer.

The soil layers to be neglected in earthquake-resistant design are:

1) Sandy soil layers vulnerable to liquefaction—Saturated sandy soil layers within 10 m of the actual ground surface with a standard penetration test N-value less than 10, a coefficient of uniformity less than 6, and a D_{10}-value of the grain size accumulation curve between 0.04 mm and 0.5 mm. If the D_{10}-value is between 0.004 mm and 0.04 mm or between 0.5 mm and 1.2 mm, special consideration should be paid because of moderate liquefaction potential.

2) Extremely soft cohesive soil layers—Cohesive or silty soil layers within 3 m of the actual ground surface with a compressive strength as determined by unconfined compression test or field test less than 0.2 kg/cm².

2.5. SEISMIC EARTH PRESSURE.

For the design of structures to resist earth pressure, active earth pressure, passive earth pressure or earth pressure at rest is considered according to the condition of the relative displacement between structure and soil. Design formulas are based on the Coulomb's earth pressure theory, from which the Mononobe-Okabe's formulas for the calculation of seismic earth pressure are derived. Though there are some minor differences between the formulas specified in SSBB and SRS, they are basically the same. Table 7 summarizes the formulas used for the calculation of earth pressure specified in SRS, which is used for the design of structures under the jurisdiction of the Japanese National Railways.

2.6. HYDRODYNAMIC PRESSURE DURING EARTHQUAKES.

According to SRS, hydrodynamic pressure during an earthquake acting to a column-like structure is calculated by either of the following formulas:

(i) When $B/H \leq 2$ and $B/L \leq 3$,

$$p_y = \alpha k_h \gamma_w A \frac{B}{L}\left(1 - \frac{B}{4H}\right)\sqrt[4]{\frac{y}{H}} \qquad \cdots\cdots(3)$$

(ii) When $2 \leq B/H \leq 4$ and $B/L \leq 3$,

$$p_y = \alpha k_h \gamma_w A \frac{B}{L}\left(0.7 - \frac{B}{10H}\right)\sqrt[4]{\frac{y}{H}} \qquad \cdots\cdots(4)$$

where

p_y = Seismic hydrodynamic pressure per unit length at depth y from the water surface.

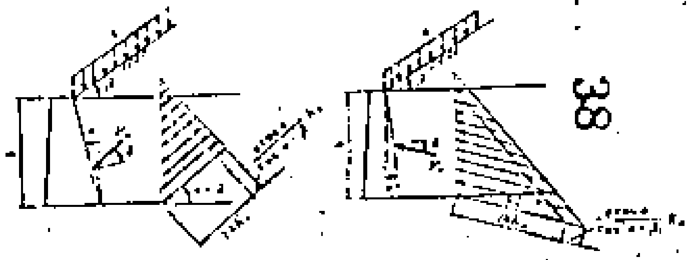
Table 7. Summary of Formulas for Calculation

	Active Earth Pressure	
	Non Cohesive Soils	Cohesive Soils
At Ordinary Times	$p_a = \gamma h K_a + \frac{q \cos\alpha}{\cos(\alpha-\beta)} K_a$	$p_a = \gamma(h - Z_0)K_a + \frac{q\cos\alpha}{\cos(\alpha-\beta)}K_a$ $Z_0 = \frac{2c}{\gamma}\tan\left(45° + \frac{\phi}{2}\right)$
	$K_a = \frac{\cos^2(\phi-\alpha)}{\cos^2\alpha\cos(\alpha+\delta)\left\{1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\cos(\alpha+\delta)\cos(\alpha-\beta)}}\right\}^2}$ Not applicable if $\alpha+\delta \geqq 90°$. Assume $\phi-\beta=0$ if $\phi-\beta<0$.	
During Earthquakes	Above groundwater table: $p_{ae} = (1-k_v)\left\{\gamma h K_{ae} + \frac{q\cos\alpha}{\cos(\alpha-\beta)}K_{ae}\right\}$	Replace h in p_{ae} for non-cohesive soils by $(h - Z_0)$, where Z_0 is as defined above.
	Below groundwater table: Replace γ by γ' in p_{ae} and θ by θ' in K_{ae}.	
	$K_{ae} = \frac{\cos^2(\phi-\alpha-\theta)}{\cos\theta\cos^2\alpha\cos(\alpha+\delta+\theta)\left\{1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta-\theta)}{\cos(\alpha+\delta+\theta)\cos(\alpha-\beta)}}\right\}^2}$ Not applicable if $\alpha+\delta+\theta \geqq 90°$. Assume $\phi-\beta-\theta=0$ if $\phi-\beta-\theta<0$.	

Notes:

1) Effect of hydrostatic pressure should be considered in the cases of active earth pressure and earth pressure acting to immovable walls.

2) α, β, q and h are shown in Fig. A.



Active Earth Pressure. Passive Earth Pressure

Fig. A. Explanation of Some of the Symbols.

1) p_a = Active earth pressure.
p_p = Passive earth pressure.
p_0 = Earth pressure at rest.
p_{ae} = Active earth pressure during earthquakes.

α = Correction factor for different types of movements of a structure (usually = 1).

A = Horizontal gross cross sectional area of a structure including hollows.

L = Length of a structure parallel to the direction of ground shaking.

B = Width of a structure perpendicular to the direction of ground shaking.

...of Earth Pressure K_p in SRS.

Passive Earth Pressure		Earth Pressure Acting to Immovable Wall
Non-Cohesive Soils	Cohesive Soils	
$p_p = \gamma h K_p + \frac{q\cos\alpha}{\cos(\alpha-\beta)} K_p$	$p_p = \gamma(h+Z_0')K_p + \frac{q\cos\alpha}{\cos(\alpha-\beta)} K_p$ $Z_0' = \frac{2c}{\gamma}\tan\left(45°-\frac{\phi}{2}\right)$	Vertical walls: $p_0 = (\gamma h + q)K_0$ Inclined walls: $p = \sqrt{p_h^2 + p_v^2}$ $p_h = \gamma h \sin\alpha$ $p_v = \gamma h K_0 \cos\alpha$
$K_p = \frac{\cos^2(\phi+\alpha)}{\cos^2\alpha\cos(\alpha-\delta)\left[1-\sqrt{\frac{\sin(\phi+\delta)\sin(\phi+\beta)}{\cos(\alpha-\delta)\cos(\alpha-\beta)}}\right]^2}$ Not applicable if $\alpha+\phi \geq 90°$ or $\phi+\beta<0$.		
Above groundwater table: $p_{pE} = (1-k_v) \times \left\{\gamma h K_{pE} + \frac{q\cos\alpha}{\cos(\alpha-\beta)} K_{pE}\right\}$ Below groundwater table: Replace γ by γ' in p_{pE} and θ by θ' in K_{pE}.	Replace h in p_{pE} for non-cohesive soils by $h+Z_0'$, where Z_0' is as defined above	Vertical wall: $p_{0E} = \gamma h K_0(1+k_h)$ $+ q k_h$ Inclined wall: $p_{0E}' = \sqrt{p_h^2 + p_{0E}^2}$
$K_{pE} = \frac{\cos^2(\phi+\alpha-\theta)}{\cos\theta\cos^2\alpha\cos(\alpha-\delta-\theta)\left[1-\sqrt{\frac{\sin(\phi+\delta)\sin(\phi+\beta-\theta)}{\cos(\alpha-\delta+\theta)\cos(\alpha-\beta)}}\right]^2}$ Not applicable if $\phi+\beta-\theta<0$.		

p_{pE} = Passive earth pressure during earthquakes.
p_{0E} = Earth pressure acting to immovable walls during earthquakes.
K_a = Active earth pressure coefficient.
K_p = Passive earth pressure coefficient.
K_0 = Coefficient of earth pressure at rest (0.5 for sand and medium to stiff clay with $N \geq 4$, 0.6 for soft clay with $2 \leq N < 4$, and 0.7 for extremely soft clay with $N < 2$).
K_{aE} = Active earth pressure coefficient during earthquakes.
K_{pE} = Passive earth pressure coefficient during earthquakes.
k_h = Horizontal design seismic coefficient.
k_v = Vertical design seismic coefficient.
γ = Unit weight of soil.
γ' = Unit weight of soil in water.
γ_w = Unit weight of water.
ϕ = Angle of internal friction of soil.
δ = Angle of friction between structure and soil (generally taken as $\phi/3$ but should be considered as zero during earthquakes).
c = Cohesion of soil.
Z_0 and Z_0' = Self standing height of cohesive soil in the case of active and passive earth pressure condition, respectively.
$\theta = \tan^{-1}\frac{k_h}{1-k_v}$
$\theta' = \tan^{-1}\left(\frac{k_h}{1-k_v}\frac{\gamma}{\gamma'}\right)$

k_h = Horizontal design seismic coefficient,
γ_w = Unit weight of water,
H = Total depth of water, and
y = Depth from the water surface.

...owever, since this hydrodynamic pressure is small for the piers of an ordinary river crossing bridge, its effect may generally be ignored.

SEHB gives the following formulas for the total hydrodynami... e acting to the structure:

(i) Wall-type structure as shown in Fig. 4(a).

$$P = \frac{7}{12} k_h \gamma_w B H^2 \qquad \cdots\cdots(5)$$

$$H_g = \frac{1}{2} H \qquad \cdots\cdots(6)$$

(ii) Column-type structure as shown in Fig. 4(b).

$$P = \frac{3}{4} k_h \gamma_w B^2 H\left(1-\frac{B}{4H}\right) \quad \text{for } \frac{B}{H} \leq 2 \qquad \cdots\cdots(7)$$

$$P = \frac{3}{8} k_h \gamma_w B^2 H \quad \text{for } \frac{B}{H} > 2 \qquad \cdots\cdots(8)$$

$$H_g = \frac{1}{2} H \qquad \cdots\cdots(9)$$

Eq. (5) is derived from the wellknown Westergaard's formula, and the point of applica tion of the hydrodynamic force is taken as a half of the depth of water for the purpose of simplicity and the increased safety margin. Eq. (7) is obtained from Eq. (3) by putting $a=1$ and $A=BH$, and by integrating over the distance y from zero to H. Eq. (8 simply states that the hydrodynamic force for $B/H>2$ is assumed to be equal to tha for $B/H=2$ since this force decreases with the increase in B/H as indicated by Eq. (4. The point of application of the force is again assumed to be $0.5H$ from the botton because of the same reasons described above.

For the seismic stability analysis of a wall-type structure as shown in Fig. 4(a), hydro static and hydrodynamic pressures and the inertia force of the structure are concurrentl taken into account in the direction of the hydrostatic pressure, and only the inert force and earth pressure are considered in the opposite direction. In the latter case, th effect of hydrostatic and hydrodynamic pressures is not considered for the purpose increased safety margin. The hydrodynamic pressure in one direction alone is consider in the design of a column-like structure surrounded by water since hydrostatic pre sure in this case is self-equilibrating.

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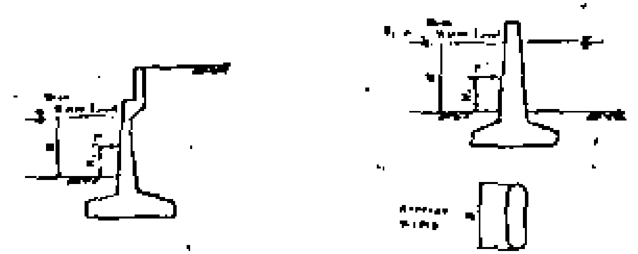


(a) Wall-type Structure. (b) Column-type Structure.

Fig. 4. Symbols Used in Eqs. (5)—(9).

2.7. ALLOWABLE STRESSES.

When earthquake forces are combined with primary loads (or dead load), allowable stresses are increased because the occurrence of such a combination is less frequent. Table 8 summarizes the factors used for increasing allowable stresses of steel and reinforced concrete members in bridge structures. Table 9 gives allowable tensile stresses of concrete and factors used for increasing allowable compressive stresses of prestressed concrete members. For the examination of the earthquake-resistance of prestressed concrete bridges against failure, a factor of safety of not less than 1.3 should be observed for the most unfavorable combination of dead load and earthquake forces.

Table 8. Factors for Increasing Allowable Stresses for Combination of Earthquake Effect and Primary Loads.

	Specifications	Combination	Factor for Increasing Allowable Stress
Steel	SRS	P* + EQ + T	1.70
	SHB	D + EQ + T	1.70
	SSHB	P** + EQ	1.50
Reinforced Concrete	SRS, SSHB and RCHB	D + EQ	1.50
		D + T + SH + EQ	1.65

Notes: SHB = Specifications for the Design of Highway Bridges.
RCHB = Specifications for the Design of Reinforced Concrete Highway Bridges.
P* = D + Live Load + Impact + Centrifugal Force.
P** = D + Live Load + Impact + Earth Pressure + Hydrostatic Pressure + Uplift.
EQ = Earthquake Force.
T = Effect of Temperature Variation.
D = Dead Load.
SH = Effect of Drying Shrinkage.

Table 9. Allowable Stresses of Prestressed Concrete Members for Earthquake Effect.

	Allowable Tensile Stresses in Tensile Zone of a Member (kg/cm²)			Factor for Increasing Allow. Comp. Stresses in Comp. Zone of a Member	Allowable Tensile Stress of Prestressed Bars of Wires	Allowable Tensile Stress of Steel Bars
	Strength of Concrete (kg/cm²)					
	300	400	500			
SRS	25	30	35	65%	—	1.5× ordinary allow. stress
TEPC**	45	50	55	60%	0.9× Yield Stress	Yield Stress
PCHB***	—*	—*	—*	65%	As above	As above

* Tensile stresses must be totally resisted by steel bars and prestressing bars or wires ignoring the tensile strength of concrete.
** Aseismic Design Standard of Tokyo Expressway Public Corporation.
*** Specifications for the Design of Prestressed Concrete Highway Bridges.

2.8. GENERAL PROVISIONS FOR DESIGN OF STRUCTURAL DETAILS.

The earthquake-resistant design should provide sufficient stability against seismic disturbances for a bridge as a whole and for its all component parts. It is essential for a bridge to maintain its structural integrity even in the event of a strong seismic motion. For this purpose, the importance of the design of structural details cannot be over-emphasized.

SEHB states that special attention should be paid to the following respects:

1) When abutments are constructed on soft ground, the possibility of failure of the surrounding soil should be examined.

2) When the substructures of a bridge are built on different types of ground, or of different structural types, or of different dimensions, the structural details should be designed by taking into account that the substructures may respond differently during an earthquake.

3) Joints between super- and sub-structures, connections between piers and foundations, or between footings and piles in pile foundations are often vulnerable to seismic disturbances. Design of such structural details should be made by considering accuracy of evaluation of ground conditions, existence of construction joints, accuracy of construction, etc.

There are three possible methods applicable to the design of bridge supports with special reference to preventing spans from falling off their supports during earthquakes. They are 1) installation of adequate stoppers at expansion bearings to restrict excessive movement, 2) widening bridge seats, and 3) connecting adjacent girders or girder with pier or abutment. According to SEHB, an expansion bearing should be provided with an adequate stopper, and in addition either of 2) or 3) mentioned above should be incorporated. A stopper may be an integrated part of a bearing or may be installed independently near the bearing. The horizontal design seismic coefficient used for the design of such stoppers is 1.5 times greater than that obtained by Eq. (1) or (2). SEHB specifies that the length S (in cm) between the end of the bearing and the edge of the sub-structure (as shown in Fig. 5(a)) be not less than the following values:

$$20 + 0.5\,l \quad \text{for } l \leq 100\text{ m}$$
$$30 + 0.4\,l \quad \text{for } l > 100\text{ m}$$

in which l is the span length in meters. These values were determined from past experience and are considered sufficient to prevent collapse of girders and to prevent spalling of concrete in the peripheral region of the substructure's crest during strong earthquake disturbances. From the similar considerations, the minimum length between the ends of girders at a suspended joint (as shown in Fig. 5 (b)) is specified as 60 cm. For particularly important bridges constructed on soft ground (Group 4 ground in Table 2), it is desirable to have the length S greater than 35 cm and the length at a suspended joint greater than 70 cm.

Connections between super- and sub-structures should be so designed as to reliably transmit the seismic inertia forces from the former to the latter. For this purpose, SEHB specifies the following methods:

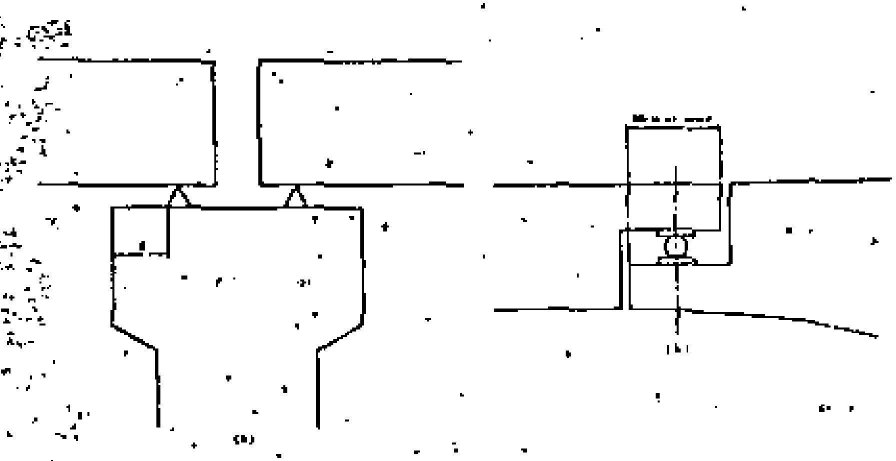


Fig. 5. Wider Bridge Seats on Pier Top or at Suspended Joint.

1) For cast-in-place reinforced concrete bridges, seismic forces should be transmitted by the bearing of the ribs on the underside of the shoe. The concrete near the shoe should be so constructed as to resist seismic forces as an integrated part of the pier or the abutment. The transmission of seismic forces from the upper bearing assembly to the girder is made by the anchors fixed on the upper bearing assembly.

In case that the ribs of the shoe become unable to effectively transmit forces from the pier or the abutment, it is desirable to design the anchor bolts between the shoe and the sub-structure to transmit the total seismic force alone.

This method is recommended not only for cast-in-place reinforced concrete bridges but, if possible, for prefabricated concrete bridges and steel bridges.

2) When no bearing resistance of concrete is expected, anchor bolts alone should be able to transmit the total seismic force. In this case, either of the following methods is to be used:

(a) A steel plate is firmly tightened to the anchor bolts which are buried in the sub-structure during its construction, and the shoe is welded to the steel plate after the erection of girders.

(b) A hole is prepared during the construction of sub-structure, the shoe with anchor bolts is placed on it and the hole is filled with cement mortar. Or anchor bolts alone are first buried in cement mortar and the shoe is fixed to the bolts.

3) The diameter of anchor bolts should be not less than 25 mm, and the length within concrete not less than 10 times the diameter.

2.9. INERTIA FORCES OF SUPER-STRUCTURES ACTING TO SUB-STRUCTURES.

One of the most important loadings for the earthquake-resistant design of sub-structures is the inertia force of super-structures acting to the sub-structures through bearings. In SEHB, these inertia forces are obtained as follows (refer to Fig. 6 for the symbols used):

(i) Horizontal seismic force H_{AL} acting to left abutment;

Smallest of $R_{AL}f_{AL}$ and $1/2\ k_hW_A$.

(ii) Horizontal seismic force $(H_{PA}+H_{PB})$ acting to pier;

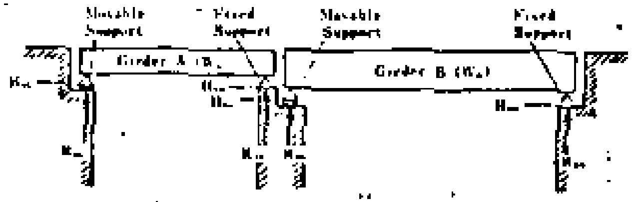
Largest of k_hW_A and $(1/2\ k_hW_A+R_{PB}f_{PB})$

where $R_{PB}f_{PB}\leq 1/2\ k_hW_B$.

(iii) Horizontal seismic force H_{BR} acting to right abutment;

k_hW_B

In the design of sub-structures, the inertia forces exerted from super-structures are assumed to act at the level of the base of bearings in the longitudinal direction, and at the level of the center of gravity of the super-structures in the transverse direction.



k_h = Horizontal design seismic coefficient by Eq. (1) or (2).
W_A = Dead load of super-structure A.
W_B = Dead load of super-structure B.
R_{AL} = Reaction at left abutment due to W_A
R_{PA} = Reaction at pier due to W_A
R_{PB} = Reaction at pier due to W_B
R_{BR} = Reaction at right abutment due to W_B
f_{AL} = Coefficient of static friction at movable bearing on left abutment.
f_{PB} = Coefficient of static friction at movable bearing on pier.

Fig. 6. Seismic Forces Acting to Sub-Structures from Super-Structures.

2.10. SAFETY FACTORS FOR FOUNDATION DESIGN.

As was mentioned in Section 2.4, the bearing capacities of the soils above the design ground surface are neglected in the earthquake-resistant design of foundation structures. This concept is also adopted in SRS, on which the contents of Sections 2.11, 2.12 and 2.13 are based.

For the evaluation of the vertical bearing capacity of the soil beneath the base of a spread footing foundation, it is useful to introduce the concept of the increased load, which is defined as

$$\begin{aligned}(\text{Increased Load}) &= (\text{Vertical Load by Foundation}) \\ &\quad -(\text{Weight of Pre-existing Soil}) \\ &= N-\gamma D_f A' \qquad \ldots\ldots(10)\end{aligned}$$

where γ is the unit weight of soil, D_f is the depth to the base of the footing, and A' is the effective bearing area of the footing. Then, by denoting the ultimate bearing capacity by Q and the factor of safety by F_s, the following condition must be satisfied for the foundation to be stable;

$$N-\gamma D_f A'\leq \frac{1}{F_s}Q \qquad \ldots\ldots(11)$$

The factors of safety specified in SRS for foundation design are summarized in Table 10.

Table 10. Factors of Safety Specified in SRS for Stability of Foundation Structures.

Loading Condition	Factor of Safety
Primary Loads	3
Primary Loads + Temporary Loads	2
Earthquake Load	1.5 (1.2*)

* When live loads (axle loads) are considered.

11. DESIGN OF SPREAD FOOTING FOUNDATIONS.

The stability of a spread footing foundation is examined for the vertical and horizontal bearing capacities of the surrounding soils and the overturning of the foundation.

The ultimate vertical bearing capacity for the increased load is calculated by

$$Q=A'\{I_c\alpha cN_c+I_r\beta\gamma_1B_eN_r+I_q\gamma_2D_f(N_q-1)\} \qquad \ldots\ldots(12)$$

and the allowable bearing capacity is obtained by

$$R_a=\frac{1}{F_s}Q+\gamma_2D_fA' \qquad \ldots\ldots(13)$$

in which the following notations are used (also refer to Fig. 7):

$A'=B'L'$ = Effective area of footing.

$B'=B-2e_x$ = Effective width of footing.

$L'=L-2e_y$ = Effective length of footing.

e_x = Eccentricity of resultant in x-direction.

e_y = Eccentricity of resultant in y-direction.

B_e = Minimum of B' and L'.

I_c, I_r, I_q = Modification factors for inclined loads.

$I_c=I_q=(1-\delta/90)^2$

$I_r=(1-\delta/\phi)^2$ where $0\leq 1-\delta/\phi$.

$\delta=\tan^{-1}(H/N)$

H = Horizontal force acting to footing.

N = Effective vertical force at the base of footing.

ϕ = Internal friction of the soil below footing.

α, β = Shape factors of the base of footing (refer to Table 11).

c = Cohesion of the soil below footing.

γ_1 = Effective unit weight of the soil below footing.

γ_2 = Effective unit weight of the soil above the base of footing.

N_c, N_r, N_q = Bearing capacity factors of the soil below footing (refer to Table 12).

F_s = Factor of safety as given in Table 10.

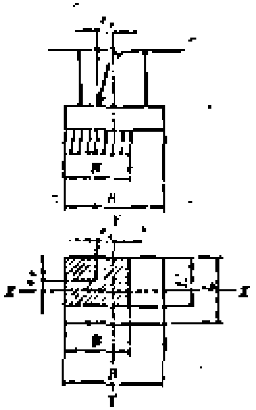


Fig. 7. Effective Width and Area of Rectangular Footing.

The ultimate horizontal bearing capacity at the base of footing is evaluated by

$$R_f=N\tan\delta+A'c' \qquad \ldots\ldots(14)$$

and that due to earth pressures at the front face of footing is obtained by

$$R_p=\alpha L(\Sigma p_p\Delta h)-L(\Sigma p_a\Delta h) \qquad \ldots\ldots(15)$$

Table 11. Shape Factors of Base of Footing.

Shape Factor	Shape of the Base of Footing			
	Long and Narrow Strip	Square	Rectangle	Circle
α	1.0	1.3	$1+0.3\dfrac{\text{Min of } B' \,\&\, L'}{\text{Max of } B' \,\&\, L'}$	1.3
β	0.5	0.4	$0.5-0.1\dfrac{\text{Min of } B' \,\&\, L'}{\text{Max of } B' \,\&\, L'}$	0.3

Table 12. Bearing Capacity Factors

ϕ (degree)	N_c	N_r	N_q
0	5.3	0	1.0
5	5.3	0	1.4
10	5.3	0	1.9
15	6.5	1.2	2.7
20	7.9	2.0	3.9
25	9.9	3.3	5.6
28	11.4	4.4	7.1
32	20.9	10.6	14.1
36	42.2	30.5	31.6
Equal or Greater than 40	95.7	114.0	81.3

or, for retaining walls, by

$$R'_p=\alpha L(\Sigma p_p\Delta h) \qquad \ldots\ldots(16)$$

By using the quantities calculated above, the allowable horizontal bearing capacities for the general case and for the design of retaining walls are evaluated by Eq. (17) and (18) shown below, respectively:

$$R_{ah}=\frac{1}{F_s}(R_f+R_p) \qquad \ldots\ldots(17)$$

$$R_{ah}'=\frac{1}{F_s}R_f+R'_p \qquad \ldots\ldots(18)$$

In Eqs. (14) through (18), the following notations are used except for those already defined (also refer to Fig. 8):

δ = Angle of friction between the base of footing and the soil below.

Supporting Layer	When Footing Is Constructed In-Situ.	When Prefabricated Footing Is Placed.
Soil or Soft Rock	$\delta=\phi$	$\delta=2/3\phi$
Hard Rock	$\delta=45°$	$\delta=30°$

c' = Cohesion between the base of footing and the soil below.

α = Shape factor of the front face of footing as shown in Fig. 9 as a function of D_f/B; $\alpha=1$ for retaining walls.

$p_p=\gamma_1 hK_p+2c\sqrt{K_p}$

$p_a=\gamma_2 hK_a-2c\sqrt{K_a}$

K_p = Passive earth pressure coefficient.

K_a = Active earth pressure coefficient.

Δh = Thickness of the layer whose horizontal resistance is taken into account.

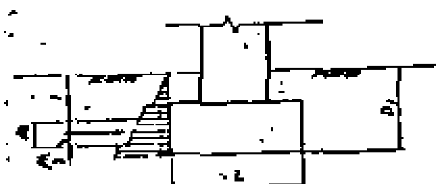


Fig. 8. Ultimate Horizontal Bearing Capacity at the Front Face of Footing.

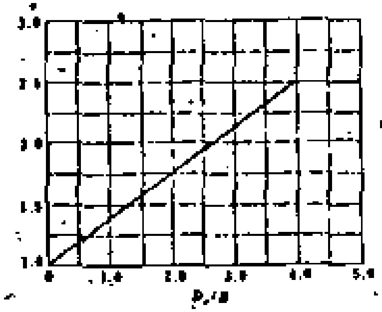


Fig. 9. Shape Factor (α) of the Front Face of Footing for the Evaluation of Eq. (15).

The stability of a spread footing against overturning is examined by the amount of eccentricity of the point of application of the load resultant acting to the base of the foundation. According to SRS, the eccentricity should not exceed the values shown below (also refer to Fig. 10):

(i) For primary loads,

e_o (but, if consolidation settlement is expected, $e_o/2$).

(ii) For primary plus temporary loads,

$e_o+\Delta x/4$.

(iii) For earthquake load,

$e_o+\Delta x/2$.

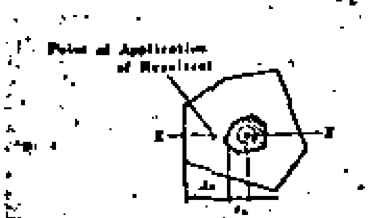
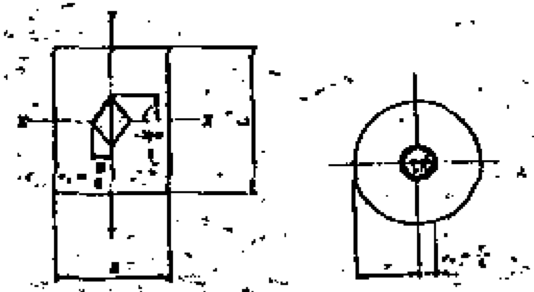


Fig. 10. Explanation of e_o and Δx.



(a) Rectangular (b) Circular

Fig. 11. Examples of e_o

In the above expressions and in Fig. 10,

X-X = The line connecting the center of resistance of foundation base and the point of application of the resultant.

e_o = The eccentricity that makes the minimum vertical subgrade reaction zero.

Δx = The distance in X-X direction between the perimeter of the footing base and the point with e_o.

Fig. 11 and Table 13 illustrate two specific cases in which the shape of the footing base is rectangular or circular.

Table 13. Examples of Maximum Allowable Eccentricities, e_o

Loading Condition	Maximum Allowable Eccentricities	
	Rectangular Footing*	Circular Footing
Primary Loads	$B/6$	$r/4$
Primary Loads + Temporary Loads	$B/4$	$3r/10$
Earthquake Load	$B/3$	$5r/8$

* Eccentricity only in the X-X direction.

2.12. DESIGN OF PILE FOUNDATIONS.

In the design of pile foundations, considerations should be paid for 1) the vertical carrying capacity of the pile, 2) the overturning of the pile-supported structure, and 3) the displacement of the structure under the action of design loadings. Displacement here includes settlement, rotation and horizontal displacement.

The compressive load on a pile should not exceed the allowable vertical carrying capacity of the pile as obtained from the following formula:

$$R_{a}=\frac{1}{F_s}\{Q_p+Q_f-(R_n)\}-W_p+(W_s)+\gamma D'_f A \quad \cdots\cdots (19)$$

in which

R_a = Allowable carrying capacity at the top of the pile,

F_s = Factor of safety given in Table 10,

Q_p = Ultimate resistance of the base of the pile,

Q_f = Ultimate skin friction over the embedded shaft length of the pile,

R_n = Negative skin friction (to be considered when necessary),

W_p = Dead weight of the pile,

W_s = Weight of the soil displaced by the pile (not considered when the ultimate carrying capacity is determined from loading tests),

γ = Average unit weight of the soil,

D'_f = Depth from the ground surface to the base of the footing, and

A = Cross-sectional area of the pile (Effective area in the case of pile group).

The tensile load on a pile should not exceed the allowable pullout resistance calculated by

$$R_t = \frac{1}{F_t} Q_t + W_p \qquad \ldots\ldots(20)$$

which

R_t = Allowable pullout resistance at top of the pile, and

Q_t = Ultimate pullout resistance of the pile.

The stability against overturning during an earthquake is examined as follows;

(i) The point of application of the resultant should be located inside of the center of the exterior pile in the pile group.

(ii) When the tensile load on some piles is found to exceed the allowable pullout resistance, the maximum compressive load recalculated by ignoring these tensile piles should be less than the allowable vertical carrying capacity.

(iii) When only the vertical carrying capacity is examined during design because of apparently favorable conditions, any pile in the pile group should not be subjected to tensile load.

Allowable displacements are not specified explicitly, but they are determined by considering the serviceability of the structure under consideration.

13. DESIGN OF CAISSON FOUNDATIONS.

Design of caisson foundations is performed by taking into account the vertical bearing capacities at the top and at the base of the caisson, the horizontal bearing capacity, the overturning moment and the displacements caused by the design loadings.

The allowable vertical bearing capacity for the increased load (cf. Section 2.10) at the base of a caisson is obtained by dividing the ultimate vertical bearing capacity calculated by an expression almost identical to Eq. (12) by an appropriate factor of safety given in Table 10. The vertical carrying capacity at the top of the caisson includes the effect of skin friction, and the allowable capacity is calculated by using a formula similar to Eq. (19). For the purpose of increased margin of safety, the effect of skin friction is usually neglected when examining the stability during earthquakes. The ultimate horizontal bearing capacity is obtained by using equations similar to Eqs. (14) and (15), and the allowable capacity is evaluated by Eq. (17). The allowable resisting moment is also obtained by dividing the ultimate resisting moment by an appropriate factor of safety. The methods adopted in SRS to evaluate various loads acting to the caisson and the ultimate resisting moment are too complicated to be included here.

2.14. SPECIFICATIONS FOR EARTHQUAKE-RESISTANT DESIGN OF HONSHU-SHIKOKU BRIDGES.

The Honshu-Shikoku Project now under way in Japan is a large-sized project to connect Honshu and Shikoku Islands by three independent routes of bridges over the Seto-Inland Sea. Though all of the project has not been finalized yet, there will be eleven suspension bridges and two cantilever-truss bridges with spans greater than 400 m to be constructed among other shorter-span bridges. Since the Japanese National Railways started to study the feasibility of constructing bridges connecting these two islands in 1955, investigati[illegible] have been continued by the Ministry of Construction, the Japan

Highway Public Corporation and the Railway Construction Public Corporation. In 1970 the Honshu-Shikoku Bridge Authority was established by law as the overseeing body for the project.

The first report on the technical investigation was published by the Committee for Construction of Honshu-Shikoku Bridges established in the Japan Society of Civil Engineers in response to the request from both the Ministry of Construction and JNR. It contained design specifications on loads, materials and safety factors, and it also emphasized the necessity for further research into the earthquake-resistant problems. In order to further examine the results presented in the first report, a committee was organized in JSCE in 1970 and the Draft of the Specifications for Earthquake-Resistant Design of Honshu-Shikoku Bridges was issued in 1972. This draft was revised in 1974 and was finally adopted as the Specifications in 1976, which is currently used for the earthquake-resistant analysis of the proposed Honshu-Shikoku bridges.

This specifications is used for the design for bridges with span lengths greater than 200 m, and bridges with shorter span lengths are designed according to SEHB (Specifications for the Earthquake-Resistant Design of Highway Bridges (1971)).

The design horizontal acceleration is taken as 180 gal at the subsoil level where the foundation is supported. This value was obtained by assuming an M=8 class earthquake at about 150 km southeast of the construction sites.* This design earthquake is expected to occur once or twice for every 100 years. The effect of smaller but near earthquakes on bridges was judged minor in comparison with that of the design earthquake.

Two design calculation methods are specified, as follows: 1) the preliminary earthquake-resistant design is performed by the modified seismic coefficient method, and the result is examined by the dynamic response analysis, and 2) when the preliminary design is obtained by the overriding factors other than seismic consideration, it is examined by means of the response spectrum technique. In the latter case, use of the dynamic response

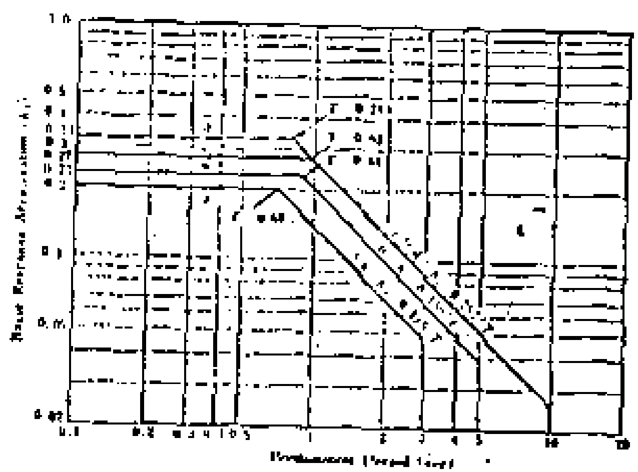


Fig. 12. Basic Response Acceleration Used for Modified Seismic Coefficient Method in Earthquake-Resistant Design of Honshu-Shikoku Bridges.

analysis by the numerical integration method is required for the result obtained by the spectrum technique.

In the modified seismic coefficient method, a structure is idealized as a one-degree-of freedom system, in which the predominant period is defined as that period which produces the most unfavorable stress or deformation in the members of the main structural system under consideration. The basic response acceleration in terms of g (= acceleration of gravity) is obtained from Fig. 12. One of the curves (a)—(d) is selected from Fig. 12 according to the type of structure, namely

- Curve (a) : For foundation structures having large cross-sectional areas with sufficient embedded length, in which dissipation damping is expected to be more pronounced than structural damping.
- Curve (b) : For foundation structures which do not correspond to either Curve (a) or (c).
- Curve (c) : For foundation (or sub-) structures, mostly of steel members, with relatively long protruded length above the ground surface, in which most damping is due to structural damping.
- Curve (d) : For main towers, suspended structures and main cables of suspension bridges.

For the spectrum response technique, a sufficient number of modes should be taken into account so that maximum responses of various parts of the structure can be determined with the required accuracy. The total response acceleration spectra are represented by Curve (a) in Fig. 12 for a damping factor of 10%, Curve (b) for a damping factor of 5%, and Curve (d) for a damping factor or 2%.

In the dynamic response analysis, time histories of various responses of a structure, generally idealized as a multi-degree-of-freedom system, are calculated from the equations of motion by using recorded earthquake motions as input. The earthquake motions to be used for analysis are 1) representative ground motions recorded near the construction site, and 2) the 1940 El Centro records. For both cases, the maximum acceleration is normalized to 180 gal. The latter was chosen because there are a number of results available from past analysis obtained by using the El Centro records.

3. EXAMPLES OF EARTHQUAKE RESISTANT DESIGN OF BRIDGES

3.1. INTRODUCTION.

There are various ways to design sub-structures of a bridge to resist earthquakes. Some examples are shown here in which special devices have been developed.

3.2. EXAMPLES OF DESIGNING SUB-STRUCTURE AND SUPER-STRUCTURE.

3.2.1. An Abutment Carrying All the Horizontal Seismic Force in Longitudinal Direction.

When it is difficult to make the intermediate piers so rigid as to carry all the horizontal seismic force, special abutments which carry the horizontal force are needed.

For the continuous girders of the Washinosugawa Railway Bridge[1] shown in Fig. 1, flexible intermediate piers were used because of the deep gorge. A special fixed abutment was constructed at the right-hand side to support the horizontal force due to an earthquake.

The symbols M, F and H indicate movable shoe, fixed shoe and hinge, respectively. Simple girders placed between the continuous girders and the fixed abutment are connected to them by horizontal connecting rods shown in the figure in order to transmit the horizontal force caused by an earthquake to the fixed abutment.

In the case of the Nakagawa Railway Bridge also, the horizontal seismic force in longitudinal direction is expected to be carried by the abutment situated at the right-hand

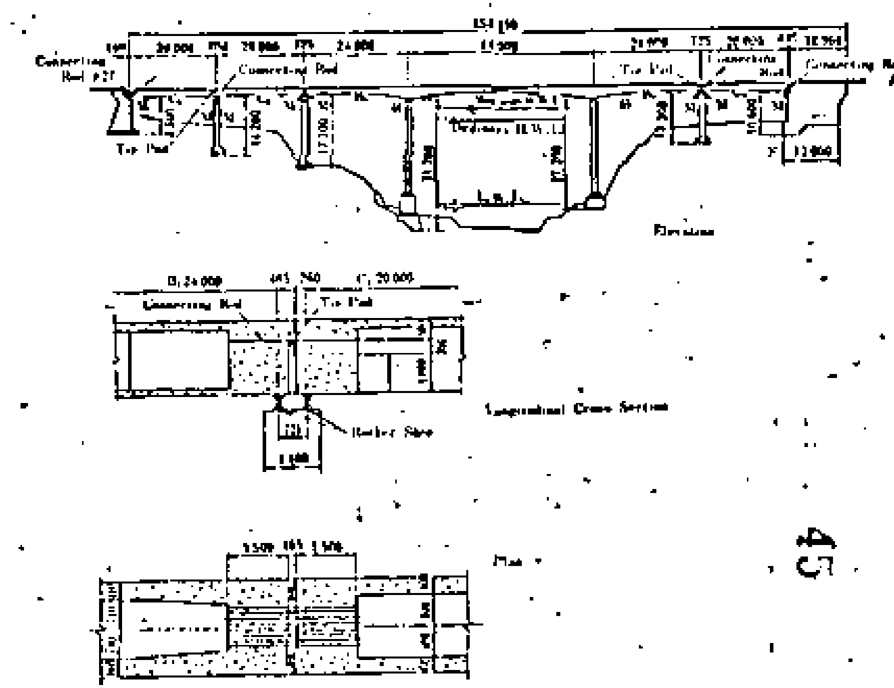


Fig. 1. Wasinosugawa Railway Bridge.

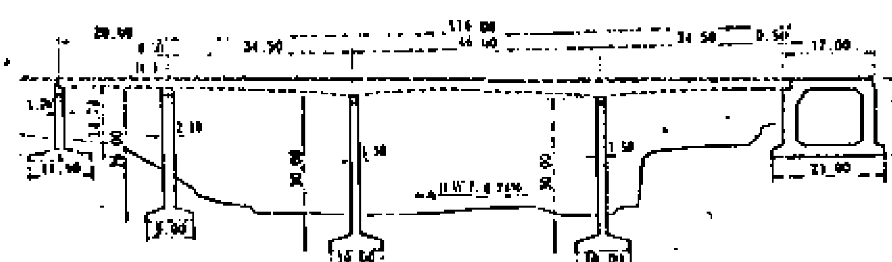


Fig. 2. Nakagawa Railway Bridge.

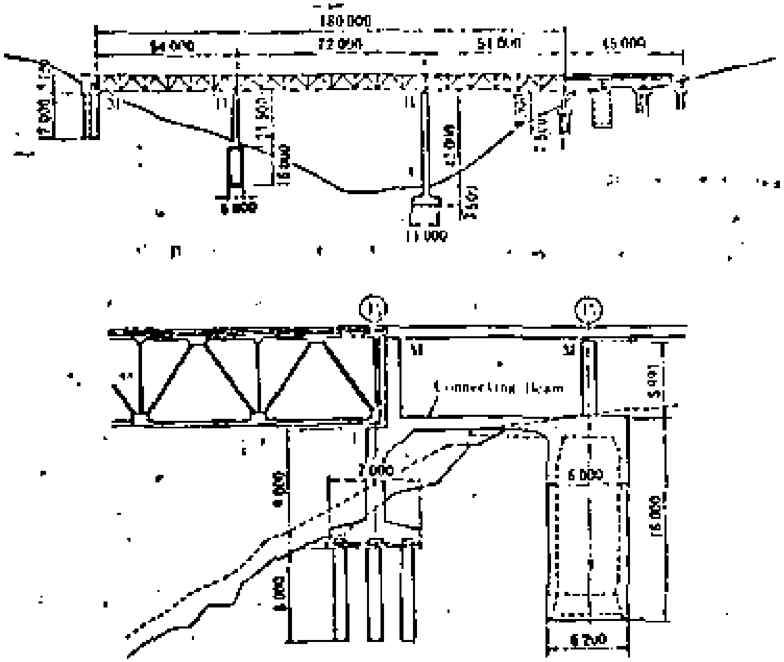


Fig. 3. Sakaigawa Bridge.

side. Mesnager Hinges with lead plates of 50 mm thickness are applied to the intermediate piers of the central 3-span continuous girder.

In the case of the Sakaigawa Bridge[3] on the Chūō Expressway shown in Fig. 3, Piers 2 and 3 are connected by an underground tie beam as illustrated in the figure. They are designed to carry the longitudinal seismic force from the continuous truss. The tall intermediate piers are more affected by the transverse seismic force of the truss than by the longitudinal one. Analysis of the dynamic response of the bridge to earthquakes was carried out to investigate the influence of earthquake motions on the structural system as a whole.

3.2.2. Intermediate Piers Carrying All the Horizontal Seismic Force.

On the contrary, there are cases where the intermediate piers are employed which are so rigid to carry all the horizontal seismic force. Since the seismic displacement at the ends is assumed large in this case, special devices against the fall-down of the girder are required.

The Yoneyama Bridge[3] shown in Fig. 4 is a national highway bridge. It crosses over a deep valley with 3-span continuous curved steel girders. Steel box columns are used for the piers. Longitudinal seismic forces from the superstructure are transmitted by fixed shoes to piers 1 and 2 and carried by them. Evaluation of seismic effect was carried out by the conventional analysis and the design was examined by dynamic analysis. Movable shoes on pier 3 have stoppers to restrict an excessive displacement and the fall-down of the girders.

The girder of the Azumagawa Railway Bridge[4] is rigidly connected to the central pier so that the bending moment of the substructure due to seismic force is cut down about 20 per cent. The design was examined by dynamic analysis.

3.2.3 Bridges on Soft Ground.

The Odakase Railway Bridge[5] shown in Fig. 6 is an elevated reinforced concrete bridge of multi-span continuous rigid-frame structure. The bridge was designed to carry the horizontal forces at abutments A, B and C built directly on the hard layer at the places where the soft layer was thin. Most of the transverse horizontal force is carried by the abutments at the both ends through the bending rigidity of the upper and lower slabs spanning 90 m between abutments A and B, and 80 m between abutments B and C.

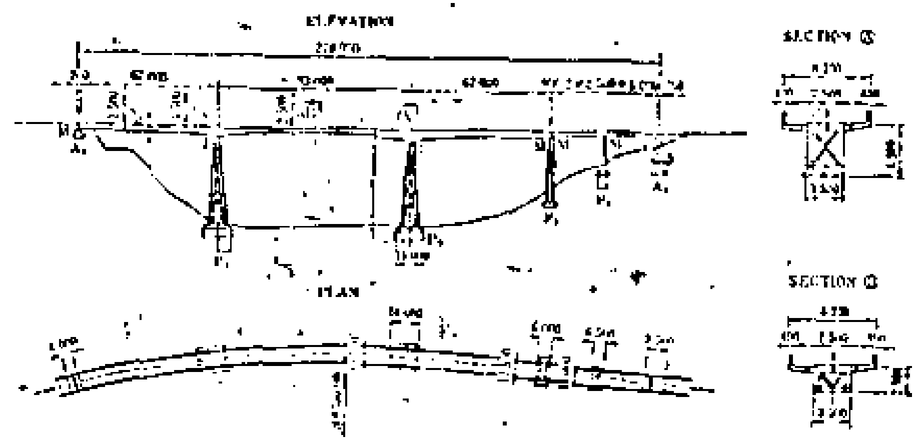


Fig. 4. Yoneyama Bridge.

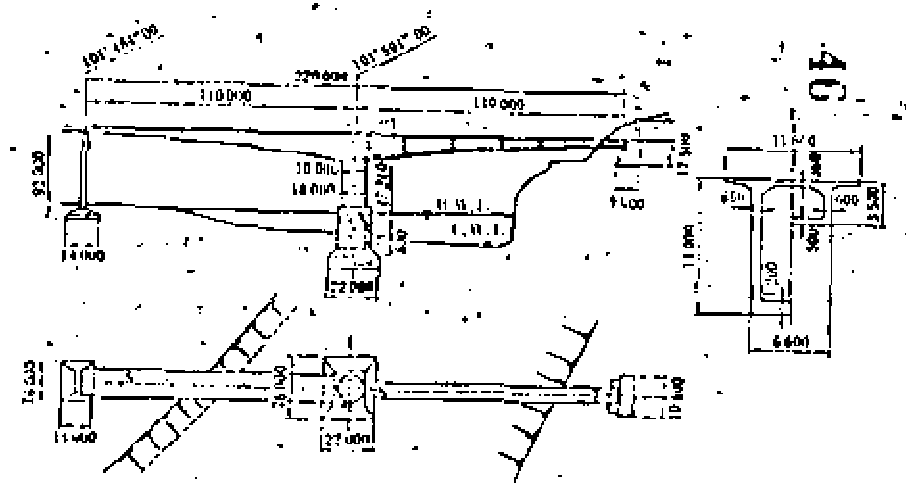


Fig. 5. Azumagawa Railway Bridge.

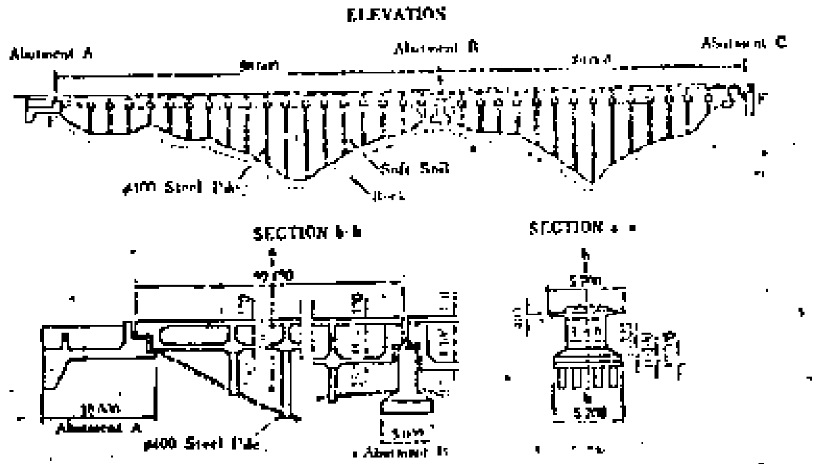


Fig. 6. Odahase Railway Bridge.

A similar idea is employed to the Ohgawara Railway Bridge on the New Tōhoku Line which is shown in Fig. 7. The piers are designed to follow the seismic motion of the ground of a large amplitude, where their shoes, carrying all the vertical forces, slide freely by the horizontal ones.

There are cases where the heads of the piles driven through soft soil layers to the bearing stratum and the girders are rigidly connected to make up a rigid frame structure to withstand the horizontal seismic force in the longitudinal and the transverse direction.

In the case of the Suzukinuma Railway Bridge[6] shown in Fig. 8 and in Photo. 1 & 2, the heads of steel pipe piles, 1 m in diameter, were rigidly connected to the concrete girders to form rigid frame structures. In the design calculations, the soft stratum was considered as liquid.

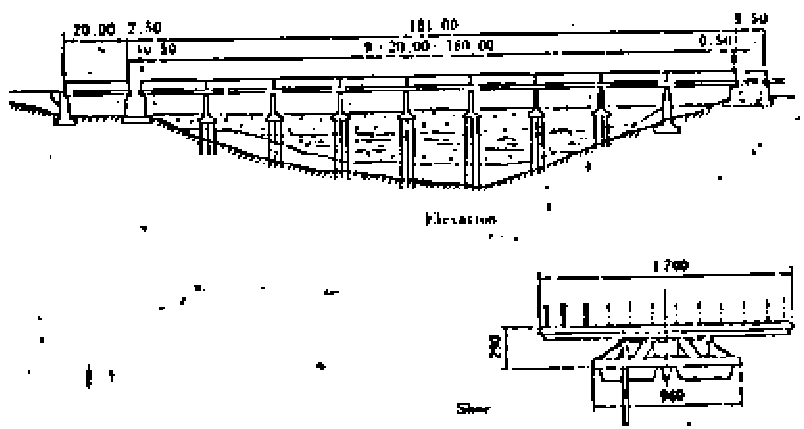


Fig. 7. Ohgawara Railway Bridge.

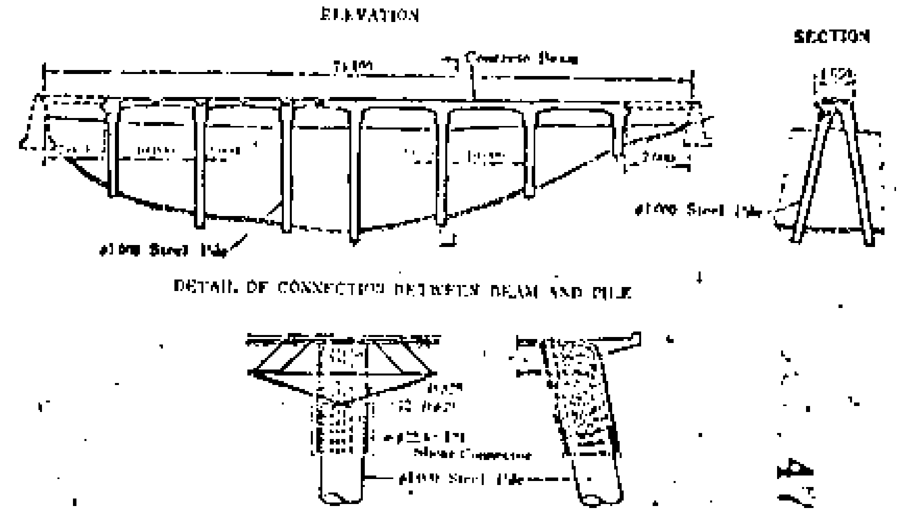


Fig. 8. Suzukinuma Railway Bridge.

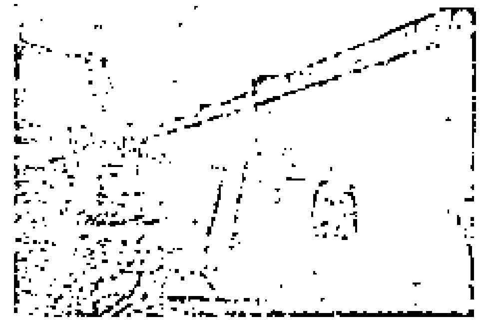


Photo. 1. Overall View

Photo. 2. Pile's Head

The intermediate piers of the Ushizugawa Railway Bridge on the Nagasaki Main Line shown in Fig. 9 are not designed to take the horizontal forces in the longitudinal direction but the abutments supported on reinforced concrete piles carry them. The connecting devices (see the details of joints A, B, and C) are designed to transmit the horizontal forces from the steel girders to the abutments.

In the case of the Hirai Bridge[7] shown in Fig. 10, a caisson is placed on the heads of the piles reaching to the bearing layer which are terminated in an intermediate layer of the soft ground. The horizontal force is mainly resisted by the passive earthpressure on the caisson. The vertical force is carried by the piles. The figure also shows the details of the connection between the piles and the caisson. To ascertain the combined action of the piles and the caisson, dynamic and static loading tests were conducted for a completed structure. Observations are being made by using seismometers, earthpressure gauges and accelerometers to investigate the vibration characteristics during an earthquake.

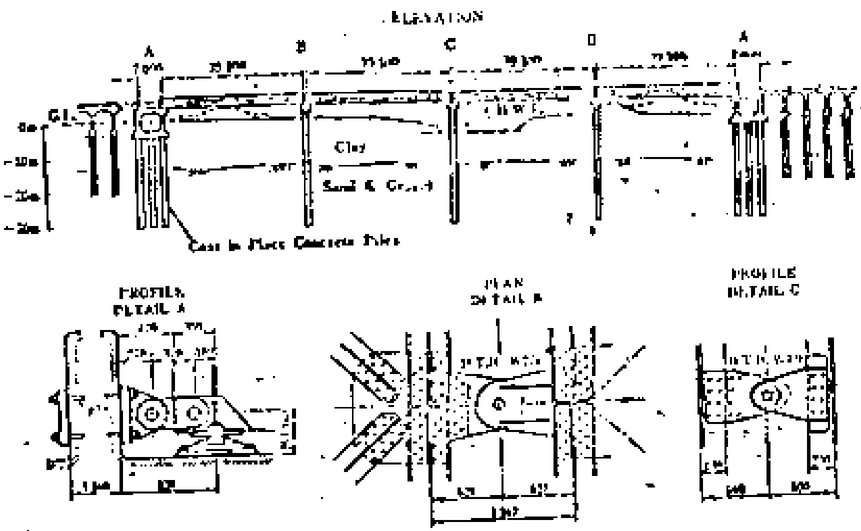


Fig. 9. Ushizugawa Railway Bridge.

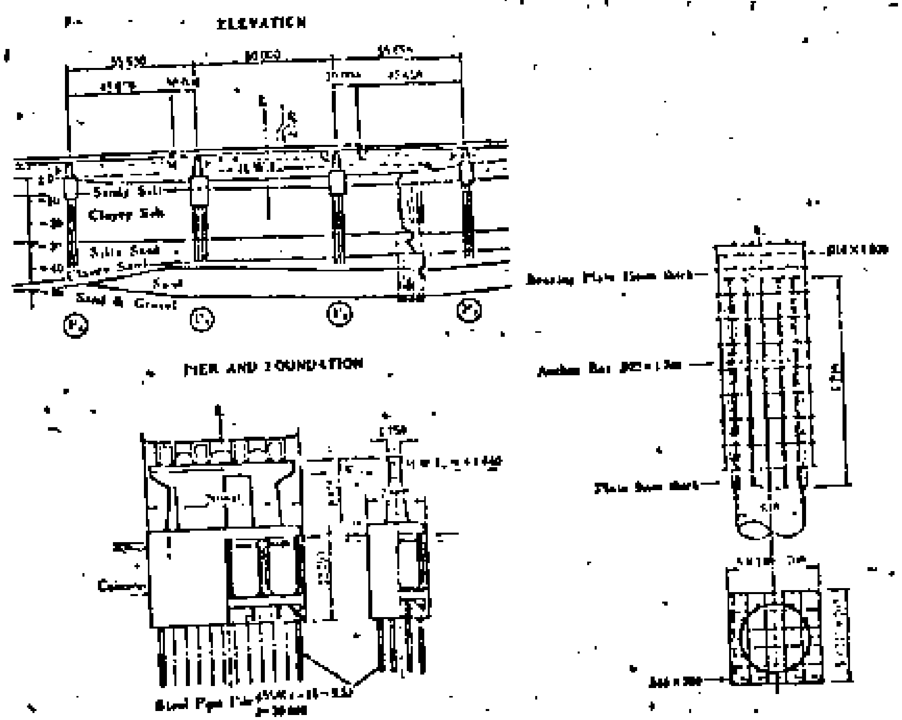


Fig. 10. Hirai Bridge.

3.2.4. Bridges on Bedrock.

In order to increase the stability of a bridge pier on the bedrock, there are cases where a footing is anchored to the bedrock with steel cables. When the cables are prestressed, the resistance of the footing against displacement can be increased.

Fig. 11 shows a pier foundation for the Yokohama-Haneda Expressway. The foundation is anchored by 12 Freyssinet cables of ϕ 12.4 mm.

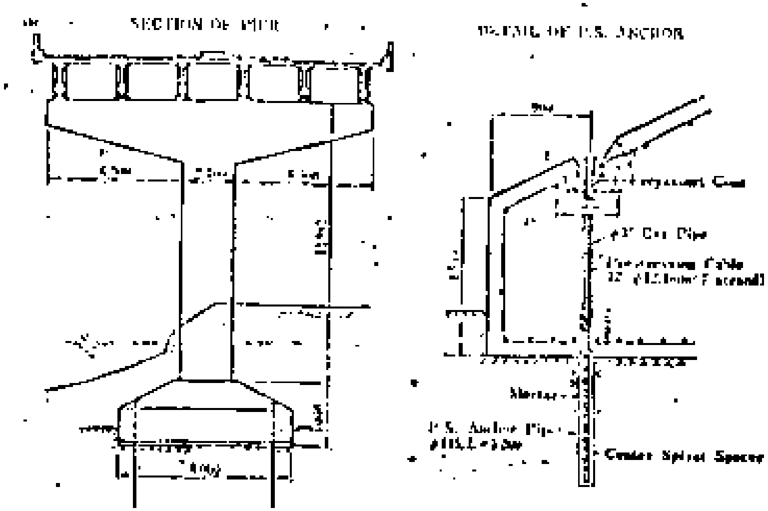


Fig. 11. Haneda Yokohama Expressway.

3.3. EXAMPLES OF ASEISMIC DESIGN OF SUPPORTS AND JOINTS.

3.3.1. Stoppers.

In an aseismic design, it is important to prevent a girder from falling down. In order to achieve this, various devices have been invented and used for the supporting part and the girder ends.

In the case of a prestressed concrete railway bridge[9] over Arakawa River on the Tōhoku Main Line, shown in Fig. 12, a stopper provided on the top of a pier fits in the holes cut in the slabs in order to prevent the girders from falling down. The girders are connected with one another by steel rods and clearances are given for the expansion and contraction of the girders due to temperature variation.

The same idea is adopted to the standard design of the New Tōhoku Line as shown in Fig. 13.

In the case of the Tamagawa Railway Bridge on the Musashino Line, shown in Fig. 14, the movable bearing is so designed as to allow displacement caused by temperature variation, shrinkage and creep of concrete, but not to permit any excessive displacement due to an earthquake.

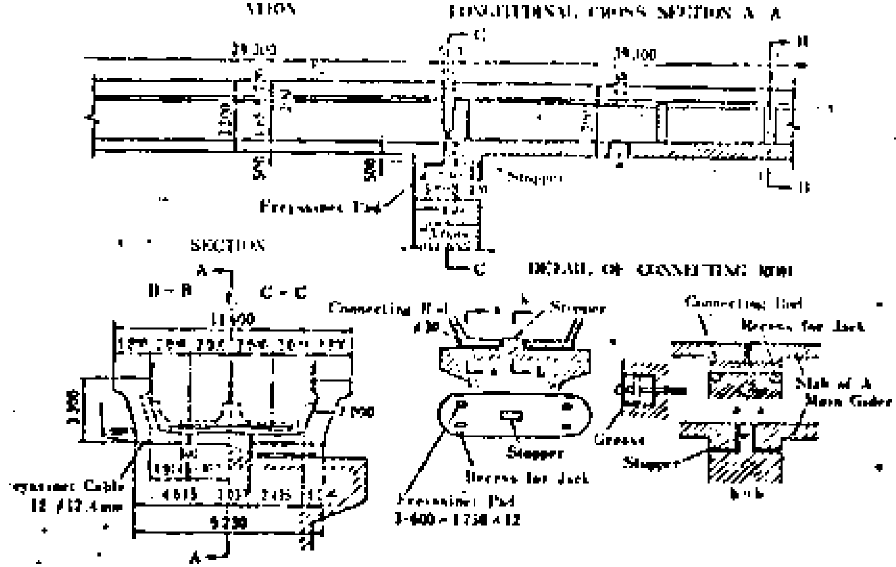


Fig. 12. Arakawa Railway Bridge.

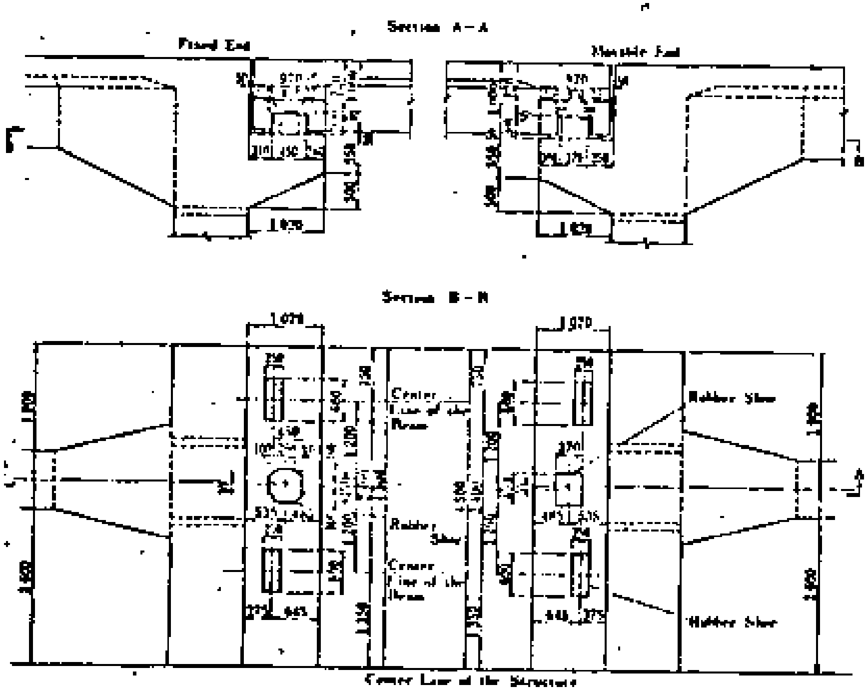


Fig. 13. Standard Design of the New Tohoku Line.

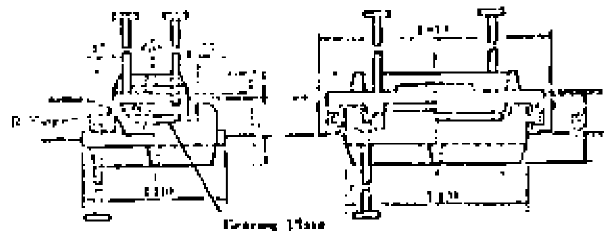


Fig. 14. Shoe of the Tamagawa Railway Bridge.

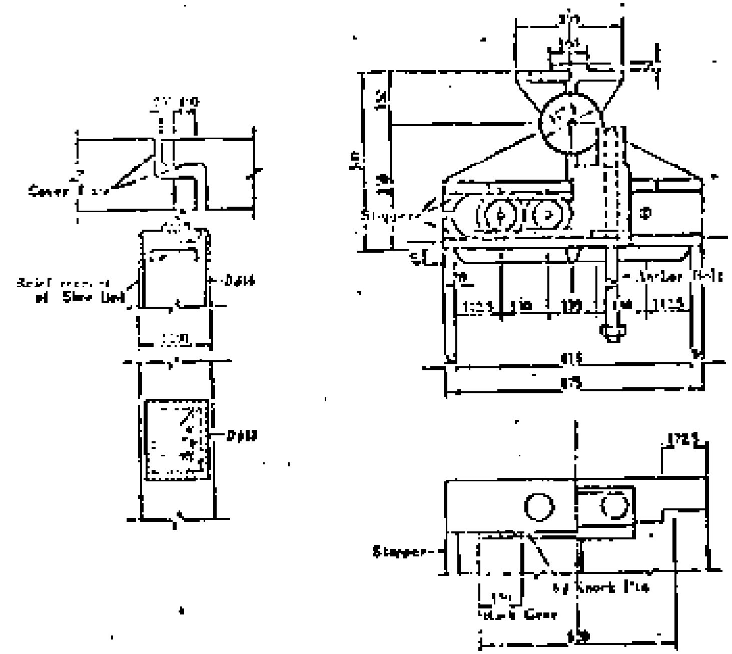


Fig. 15. Shoe of the Shin-Aoyagi Bridge.

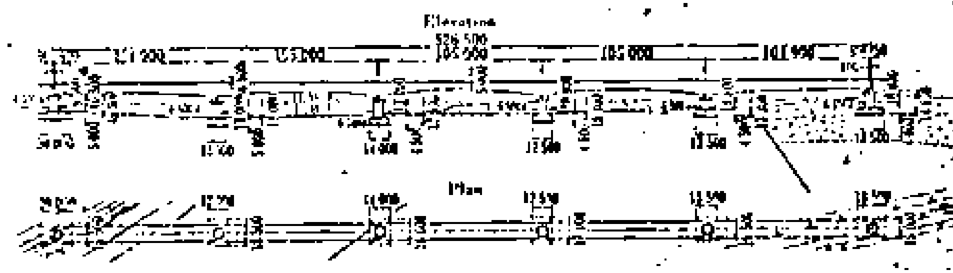


Fig. 16. Abukumagawa Railway Bridge.

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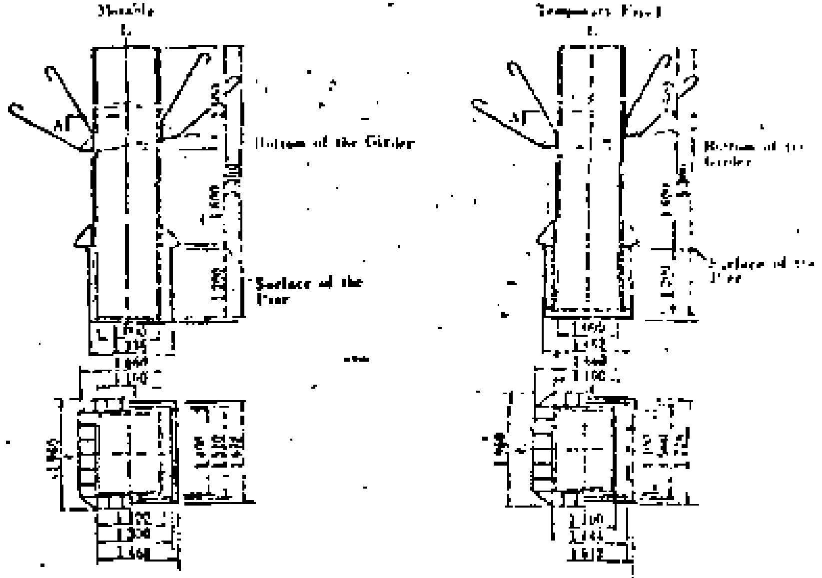

Fig. 17. One Directional Box Stopper.

The bearing shoe of the Shin-Aoyagi Bridge shown in Fig. 15 is one of the movable shoes specially designed for 5-span continuous steel girders. Seismic forces are resisted by the bearing strength of bed concrete at the ribs on the underside of a shoe and the shearing resistance of steel anchor bolts. Stoppers are provided for preventing the fall-down of a girder, and a distance of 60 cm is provided between the support of a girder and the front edge of a pier.

3.3.2. Box Stoppers.

The Abukumagawa Railway Bridge shown in Fig. 16 is a 5-span-continuous prestressed concrete railway bridge on the New Tôhoku Line. One directional box stoppers as shown in Fig. 17 are provided here. They consist of a prismatic bar embedded into the concrete of the superstructure and of a box embedded into that of the substructure. The space between them is filled with viscous material, which has resistance against fast movement during an earthquake, but has no resistance against such a slow movement as elongation or contraction of the bridge due to temperature change. The difference of resistance between fast movement and slow one is adjusted by the amount of gap between the walls. Normally the whole horizontal force is resisted only by the fixed pier whose stopper is provided with strong springs. When a seismic force hits the bridge, its horizontal components are efficiently distributed into each pier by the viscous action of the fluid.

The Enigawa Railway Bridge shown in Fig. 18 is equipped with two directional box stoppers which distribute not only the longitudinal but also the transverse horizontal seismic force. Here the principle of the one directional box stopper is doubly applied, i.e.

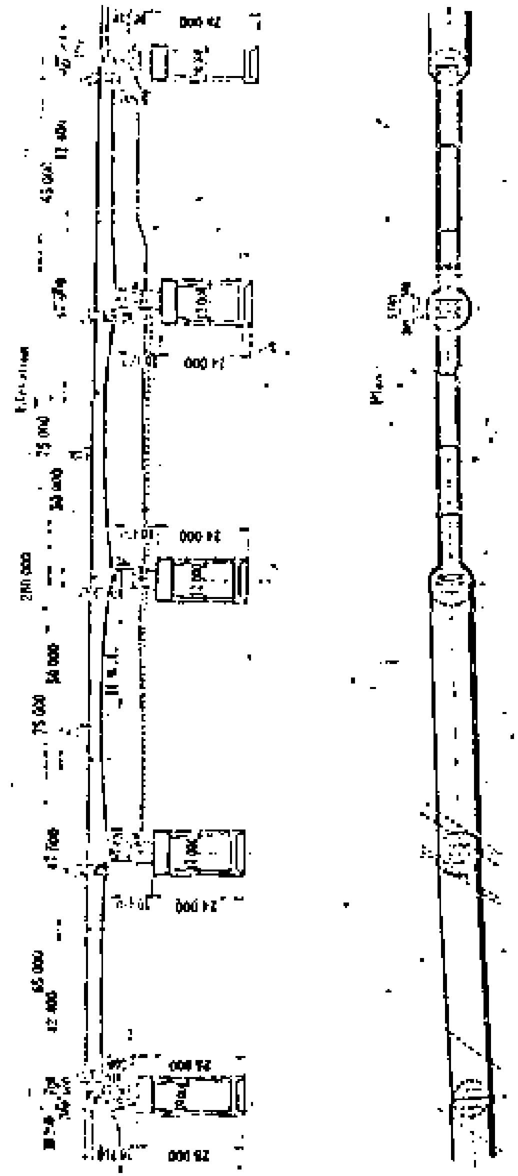

Fig. 18. Enigawa Railway Bridge.

the second box is established between the bar and the first box, where each box allows one directional movement.

The above mentioned stoppers distribute the force through the friction between the bar and the viscous material, while another type of stopper shown in Fig. being put to

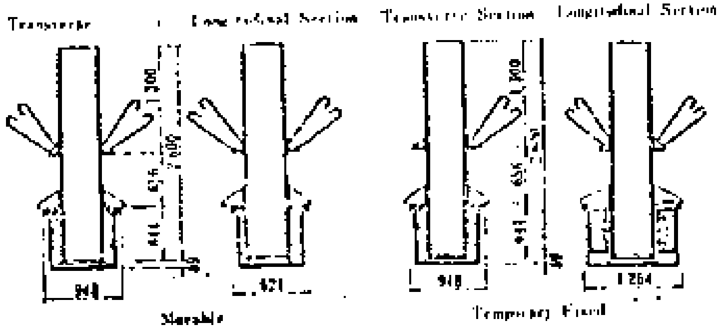

Fig. 19. Two Directional Box Stopper.

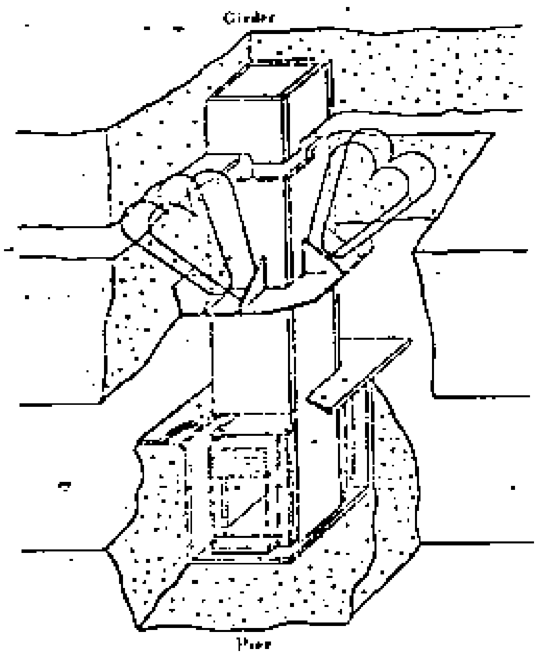

Fig. 20. Box Stopper of Shear Type.

practical use. This stopper, called "Stopper of Shear Type", can react very quickly to the seismic motions.

3.3.3 Dampers.

The Tokyo Expressway Public Corporation adopted a method in which an oil damper* is installed on each support which may be assumed movable for the slow displacement due to temperature variation, while it acts as fixed support producing a strong resistance to the abrupt displacement caused by an earthquake. (See Fig. 21, Photo. 3).

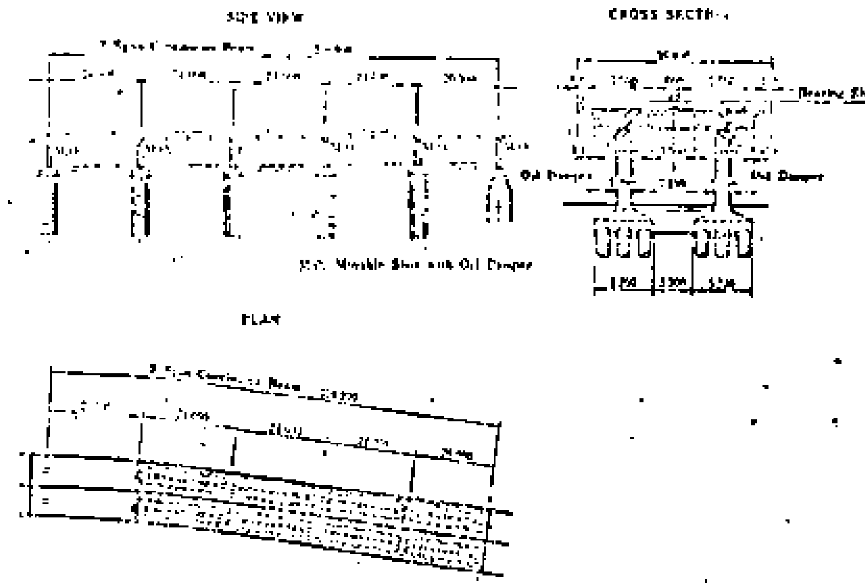

Fig. 21. Route No. 1 of the Tokyo Expressway.

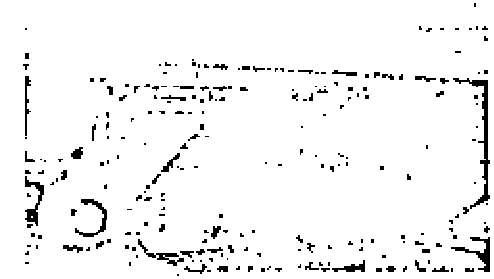

Photo. 3. Oil Damper

Another method applied to the same route of the Tokyo Expressway for the same purpose is shown in Fig. 22. Piers and girders are connected with prestressed wires which are called SU Dampers* (See Photo. 4). The wires have enough cross section and length to absorb the horizontal force due to temperature variation, shrinkage and creep of concrete girders and seismic force. The total seismic force can be distributed almost uniformly to each pier.

3.3.4. Connecting Devices.

In the case of the Shinkatsuragawa Railway Bridge on the Chūō Main Line, shown in Fig. 23, composite girders consisting of steel girders and concrete slabs are placed on piers of about 40 m height, and the girder ends are connected with one another by steel plates to prevent them from falling down. A clearance is left for the relative displacement between girder ends. Moreover, H-shaped steel beams embedded at the top of a pier as

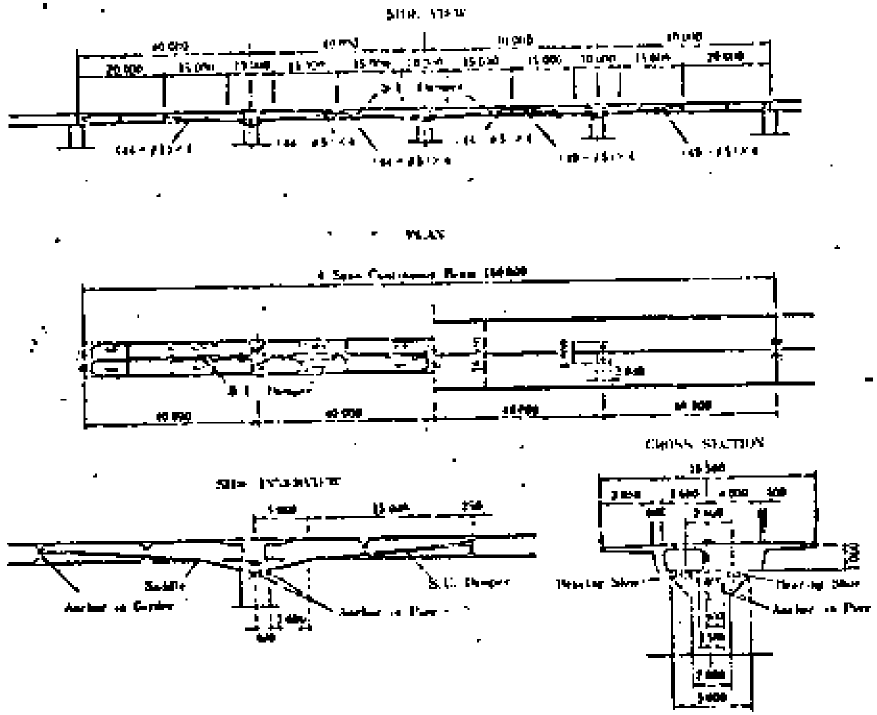

Fig. 22 Route No. 1 of the Tokyo Expressway.

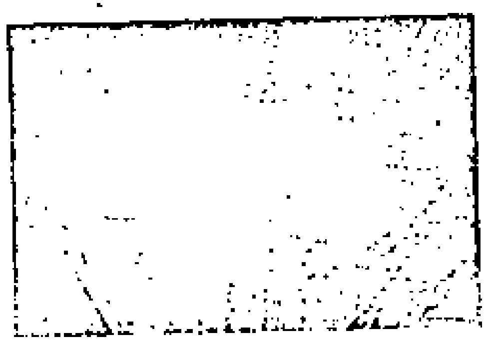

Photo. 4. SU Damper.

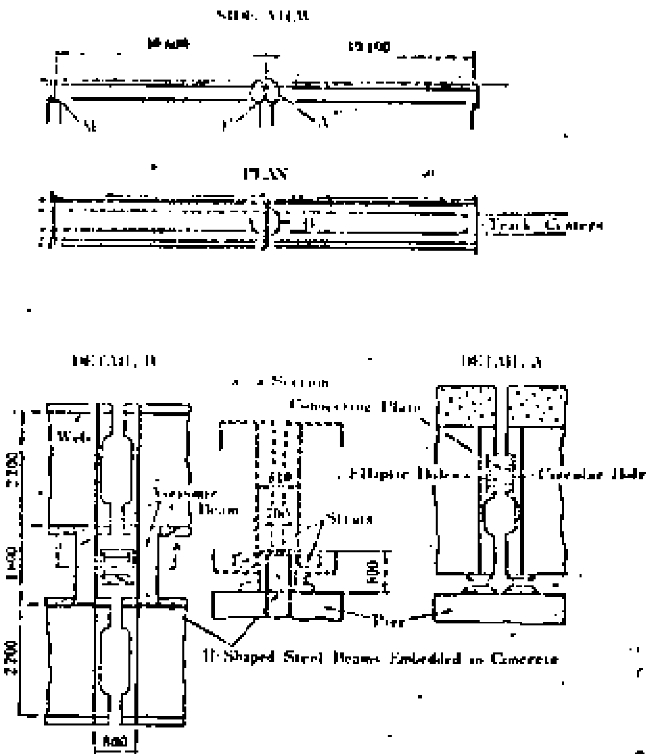

Fig. 23. Shinkatsuragawa Railway Bridge.

shown in the figure are enclosed by the aseismic beams installed at the girder ends to prevent the displacement of a girder beyond a certain limit.

In the Shimoyodogawa Railway Bridge on the Tōkaidō Main Line, shown in Fig. 24, simple trusses of 61.2 m are connected with one another so that even when one pier breaks during an earthquake, the trusses would remain intact as cantilevers.

The method of connecting the ends of girders with connecting plates is adopted for many bridges and elevated roads of the Tokyo Expressway Public Corporation.

There are cases where connecting installations are used to transmit the logitudinal horizontal force due to an earthquake.

In the case of the Ishikawa Elevated Bridge[19] of prestressed concrete structure of a national highway, shown in Fig. 25, these installations are used to connect simple girders to make them act as continuous girders against the horizontal force in the longitudinal direction. Since a prestressed concrete girder shows a small angle of slope at its ends, no expansion joint was provided between the girders and a continuous paving was made.

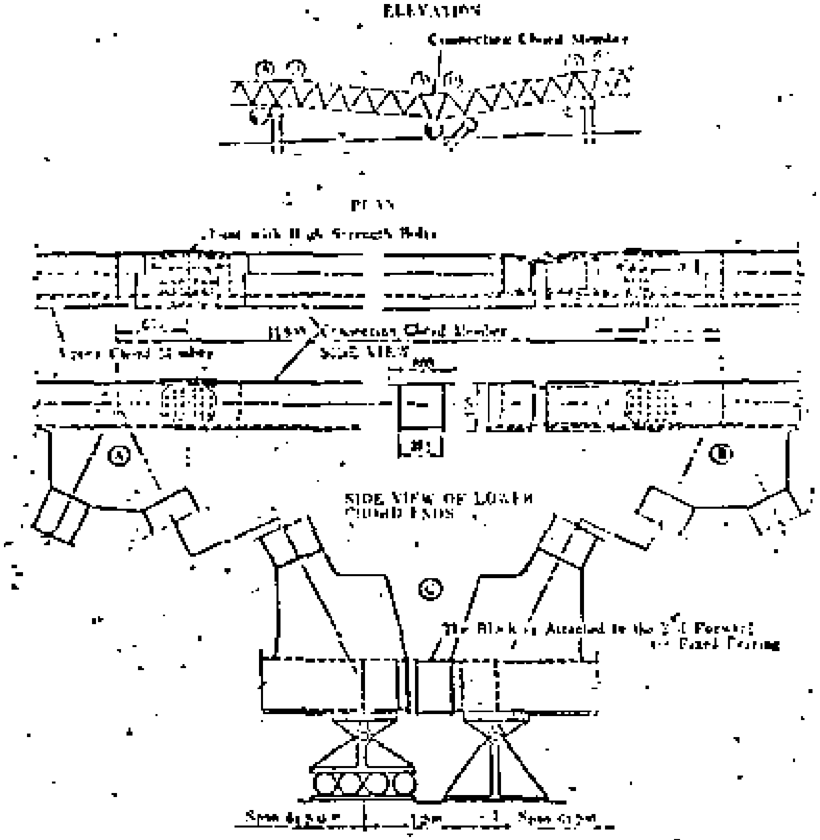

Fig. 24. Shimoyodogawa Railway Bridge.

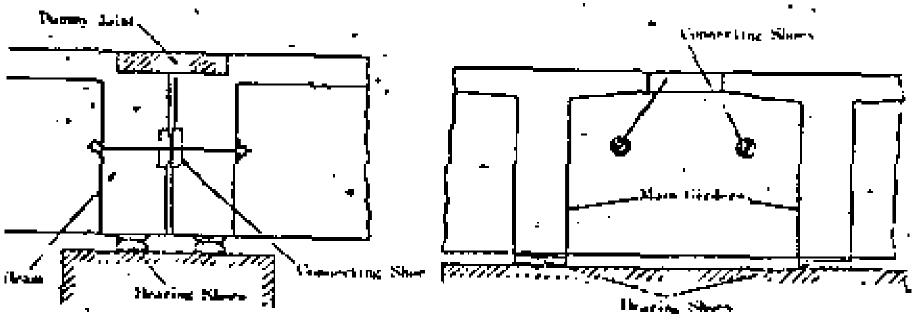

Fig. 25. Ishikawa Elevated Bridge.

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SEISMIC RESPONSE OF MULTI-SUPPORT STRUCTURES

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SUMMARY

Probabilistic models are proposed for determination of design responses in structures sensitive to phase differences among ground motion histories at their supports. Selection of design values is based on approximate proportionalities between responses corresponding to given probabilities of exceedance and standard deviations of those responses at the end of a segment of random noise taken as equivalent to earthquake excitation. Detailed practical rules are proposed for mode and component superposition as well as for definition of internal-load combinations to be used when checking safety conditions at given critical sections.

INTRODUCTION

Design values of internal forces produced by earthquakes are usually taken as those provided by an envelope to the values of those variables caused by the individual ground motion components, one at a time. When dealing with structures having small dimensions in plan, reasonable approximations to maximum response for the superposition of the various ground motion components can be obtained from a linear combination of maximum responses to each individual component at a given point of the ground-structure interface, provided modal responses can be taken as stochastically independent (1).

For structures extended in plan, such as long bridges, or founded on heterogeneous formations or irregular topography, such as dams, differences in ground motion among different supports or zones of the ground-structure interface may give place to quantitative and qualitative differences in internal actions as compared with those produced by in-phase motion of all supports. In addition, strong correlation may exist between pairs of parallel ground motion components at different points, and the correlation among the responses of a given mode to several (correlated) ground motion components can no longer be neglected.

The present paper explores several criteria for determining design responses, on the basis of the variance of the response to the various ground motion components, for a given assumption concerning the covariance structure of the generalized excitation[III]. Simplified rules for practical application are also discussed.

DYNAMICS OF LINEAR SYSTEMS SUBJECTED TO OUT-OF-PHASE SUPPORT DISPLACEMENTS

Let the excitation acting on a multi-support system be described by vector $X(t) = \{x_q\}$ of support displacements. Suppose the system is discretized and represented by the mass, stiffness and damping matrices M, K and C respectively, where support displacements are not taken as degrees of freedom. The vector Y(t) of structural response components (displacements, stresses) can be obtained as follows (2,3).

$$Y(t) = \sum_q x_q(t)\, \tilde{Y}_q + U(t) \qquad (1)$$

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III More general assumptions are covered in Ref 2.

In this equation, $\sum_q$ is taken to all the supports, $\tilde{Y}_q$ is the vector of static response components produced by a unit displacement x_q, and U is the vector of dynamic response components, i.e. those measured at anty given instant with respect to the superposition of the static configurations associated with support displacements x_q at the same instant. U can be expressed in terms of modal vectors $Z_j = \{z_{ij}\}$

$$U(t) = \sum_q \int_0^t \ddot{x}_q(\tau) \sum_j a_{jq} Z_j h_j(t-\tau) d\tau \tag{2}$$

Here a_{jq} is the participation factor of mode j that corresponds to displacements of support q, and $h_j(\tau)$ is the unit impulse response function for the mentioned mode. This factor can be determined from conditions $Y = 0$, $\dot{Y} = 0$, at $t = 0^+$, immediately after application of a concentrated unit-area acceleration pulse $\ddot{x}_q = \delta(t)$. Thus (2,3),

$$a_{kq} = \frac{r_{qk}}{\omega_k^2 Z_k^T M Z_k} \tag{3}$$

where r_{qk} is the external reaction at support q when the system vibrates freely in its k'th mode having shape Z_k and natural frequency ω_k. From Eqs. 1 and 2, a given element Y_i of the response vector of the system can be expressed as follows

$$Y_i(t) = \sum_q x_q(t)\tilde{Y}_{iq} + \sum_q \sum_j a_{jq} Z_{ij} \int_0^t \ddot{x}_q(\tau) h_j(t-\tau) d\tau \tag{4}$$

In this equation, $\tilde{Y}_{iq}$ and Z_{ij} denote the (time independent) values of Y_i associated with vectors $\tilde{Y}_q$ and Z_j, respectively.

DYNAMIC RESPONSE ANALYSIS FOR STOCHASTIC GROUND MOTION

It will be assumed that x can be represented as the product of a Gaussian stationary process $W_q(t)$ with spectral density $G_q(\omega)$ by a deterministic envelope $A_q(t)$:

$$\ddot{x}_q(t) = A_q(t) \, W_q(t) \tag{5}$$

In order to determine the parameters that define $Y_i(t)$ in probabilistic terms, it is advantageous to express that function independently of $x_q(t)$, by means of an integral expressed exclusively in terms of $\ddot{x}_q(t)$ and of a response function $g_{iq}(t)$ wich takes into account both static and dynamic components. For an elementary accelerogram defined by a concentrated impulse $\ddot{x}_q(t) = \delta(t)$, one obtains $\dot{x}_q(t) = H(t)$ and $x_q(t) = tH(t)$, where $\delta(.)$ and $H(.)$ are Dirac delta function and Heaviside step function, respectively. Y_i is then the unit impulse response function for acceleration of support q, and is given by the following equation, obtained from Eq. 4:

$$g_{iq}(t) = tH(t)\,\tilde{Y}_{iq} + \sum_j a_{jq} Z_{ij} h_j(t) \tag{6}$$

If the complete accelerogram at a given support is taken into account by integration of eq. 6 with respect to time, and if the contributions from all supports are added together, the following alternative to Eq. 4 is obtained:

$$Y_i(t) = \sum_q \int_0^t \ddot{x}_q(\tau)\, g_{iq}(t-\tau) d\tau \tag{7}$$

Taking into account Eq. 5, the last equation becomes

$$Y_i(t) = \sum_q \int_0^t A_q(\tau) W_q(\tau) g_{iq}(t-\tau) d\tau \tag{8}$$

Hence, the variance of $Y_i(t)$ is

$$\operatorname{var} Y_i(t) = \sum_r \sum_q \int_0^t \int_0^t A_q(\tau_1) A_r(\tau_2) R_{qr}(\tau_1,\tau_2) g_{iq}(t-\tau_1) g_{ir}(t-\tau_2) d\tau_1 d\tau_2 \tag{9}$$

where $R_{qr}(\tau_1,\tau_2) = E[W_q(\tau_1) W_r(\tau_2)]$ is the cross-correlation function of W_q and W_r.

From Eqs. 5 and 8 and the approximate theory of Ref. 4 it is possible to determine the probability distribution of the maximum absolute value of $Y_i(t)$ during an earthquake, but the amount of computations involved makes application of that theory impractical; adoption of simpler criteria is advantageous. Herein it is assumed that the values of responses which correspond to a given probability of exceedance during an earthquake of given intensity are proportional to the maximum values reached by the respective standard deviations while ground motion lasts. Thus, if σ_0^2 is the maximum variance of ground acceleration during the earthquake and $\ddot{x}_0(p)$ is the value of that acceleration which corresponds to probability p of being exceeded, and if σ_R^2 and $x_R(p)$ are the corresponding values associated with a response variable R, the assumption proposed implies that if the design criterion adopted is based on equal exceedance probabilities for all design responses, then the ratio of the design value of R to the specified peak ground acceleration should equal σ_R / σ_0.

Ref. 2 contains expressions for the variance of a response variable for several alternative assumptions concerning the covariance structure of the excitation. Only one particular case is discussed here: that where ground accelerations at the various supports are represented by segments of stationary white noise travelling undistorted along the ground surface:

$$\ddot{x}_q(t) = W_0(t-\tau_q), \quad q = 1, \ldots, N \tag{10}$$

In this equation, W_0 is white noise with spectral density G_0. The white-noise assumption is not restrictive, and the criterion derived from it can be applied to more general spectral shapes of excitation.

If the natural periods of the systems analyzed are short as compared with the duration of strong ground motion, maximum variances will be reached at the end of the excitation; i.e. for t=s:

$$\operatorname{var} Y_i(s) = \sum_q \sum_r \tilde{Y}_{iq} \tilde{Y}_{ir} I_{qr}(s) + 2\sum_q \sum_r \sum_k a_{kr} \tilde{Y}_{iq} Z_{ik} I'_{kqr}(s) + \sum_q \sum_r \sum_j \sum_k a_{jq} a_{kr} Z_{ij} Z_{ik} I''_{jkqr}(s) \tag{11}$$

where, for the case defined by Eq. 10, making g(t) = tH(t),

$$I_{qr}(s) = \pi G_0 \int_0^s g(s-\tau_q-\tau) g(s-\tau_r-\tau) d\tau \tag{12a}$$

$$I'_{kqr}(s) = \pi G_0 \int_0^s g(s-\tau_q-\tau) h_k(s-\tau_r-\tau) d\tau \tag{12b}$$

$$I''_{jkqr}(s) = \pi G_0 \int_0^s h_j(s-\tau_q-\tau) h_k(s-\tau_r-\tau) d\tau \tag{12c}$$

Analytic expressions for these integrals are given in the Appendix.

SUPERPOSITION CRITERIA

The ground motion models given by Eq. 5 are reasonable representations of earthquake accelerograms, in spite of the fact that they ignore frequency-content variations during a given event. These models lead to sufficiently accurate estimates of the variances of ground accelerations and of responses which are determined mainly by those accelerations; however, they do not lead to accurate estimates of variances of quantities sensitive to ground velocities or displacements (such as the dynamic response of structures possessing moderate or long natural periods, or the stresses produced by phase differences among support displacements), unless the spectral density of ground displacements (and not only of ground accelerations) is closely represented or base-line corrections are applied to accelerograms given by Eq. 5 so as to produce zero final ground velocities. As a consequence, the proportionality of design responses with square roots of variances described above cannot always be directly applied, particularly in those instances in which the proportionality refers simultaneously to quantities sensitive to ground accelerations, velocities and displacements. This difficulty can be overcome if the proportionality assumption is applied individually to each term in Eq. 11: although the displacement variances predicted on the basis of Eq. 5 may be excessive, the ratios of the variances of responses proportional to displacements predicted on the same basis will be good estimates of the ratios of the actual variances. Thus, the following equation results:

$$Q^2 = \sum_q \sum_r \Big(Q_q Q_r \alpha_{qr} D_{oq} D_{or} + 2 \sum_k Q_k a_{kr} Z_k \alpha'_{kqr} D_{oq} D_{kr} + \sum_j \sum_k a_{jq} a_{kr} Z_j Z_k \alpha''_{jkqr} D_{jq} D_{kr} \Big) \tag{13}$$

In this equation, Q is the design response, D_{oq} and D_{or} are the peak ground displacements at supports q and r, respectively; D_{jq} and D_{kr} are the displacement spectral ordinates at the same supports for modes j and k; Q_q is the static response to a unit displacement of support q, and α_{qr}, α'_{kqr}, α''_{jkqr} are proportionality factors obtained as follows:

$$\alpha_{qr} = I_{qr}(s) / (J_{qq} J_{rr})^{1/2} \tag{14a}$$

$$\alpha'_{kqr} = I'_{kqr}(s) / (J_{qq} J''_{kr})^{1/2} \tag{14b}$$

$$\alpha''_{jkqr} = I''_{jkqr}(s) / (J''_{jq} J''_{kr})^{1/2} \tag{14c}$$

J_{qq} and J_{rr} are the values of $I_{qq}(s)$ and $I_{rr}(s)$ when time is measured from the instant when ground motion starts at supports q and r, respectively, and J''_{jk}, J''_{kr} are the corresponding values of I''_{jjqq} and I''_{kkrr} for the same time origins. In other words, the contributions to design responses are expressed in terms of peak ground displacements at all supports as well as of the modal responses to each ground motion component taken as acting simultaneously at all supports.

Figs. 1-4 illustrate the variation of α, α' and α'' (determined with Eqs. A1-A3) for several combinations of natural frequencies and time-lags. The latter are expressed as fractions of the duration s of the white noise segment used to represent the excitation. On firm ground, s can be taken as 20 sec.

DESIGN CRITERIA

It is assumed that design consists in determining probable combinations of internal forces at critical sections and verifying if those combinations lie within the specified safe regions. The criterion advocated herein for the determination of the mentioned combinations is based on the same concepts as that recommended in Ref. 1, but unlike the latter it takes into account the statistical correlation among support displacements. Both criteria assume that the joint probability distribution of the internal forces which determine the most unfavorable condition at a critical section is Gaussian multi-dimensional, and that a set of multidimensional ellipsoids can be built with centers at the expected values of the internal forces and principal axes in directions which are functions of the correlation coefficients, so that to each ellipsoid corresponds a number of standard deviations of each internal force from its expected value and hence a given probability of containing the load combination of interest. A given design is adequate if the ellipsoid which corresponds to a specified probability lies within the safe region and is tangent to its boundary (1). Because contructing the mentioned ellipsoid would be excessively difficult for practical densign, it is proposed to substitute theellipsoid with a set of 2^N N points (combinations) where N is the number of cartesian components in each combination (IV). The design value of component Y_k in the j-th combination would equal $\gamma_{kj} y_k$, where y_k is the design value which would be assumed for Y_k if design were based exclusively on it, γ_{kj} equals $\pm$ 1 for k = j and $\gamma_{ij}(\rho_{kj} \pm 0.3(1-\rho_{kj}^2))$; for k $\neq$ j, and ρ_{kj} is the correlation coefficient between $Y_k(s)$ and $Y_j(s)$. An expression for cov $(Y_k(s), Y_j(s))$ of the type of Eq. 13 can readily be derived starting from Eq. 4.

Take for instance the case illustrated in Fig. 5 for the cross section of a reinforced concrete column subjected to axial load P and bending moment M with respect to one of its principal axes of inertia. The cases when the correlation coefficient ρ_{MP} equals 0 and 0.5 will be considered. Assume also that the dotted line represents the section's interaction diagram, and that p and m are the design values of M and P should each of them be considered separately. For each value of ρ_{MP}, N = 2 and the number of internal-forces combinations to be considered equals $2^2\ 2 = 8$. For $\rho_{MP} = 0$, $\gamma_{MP} = \pm 0.3$, and for $\rho_{MP} = 0.5$, $\gamma_{MP} = \pm (0.5 \pm 0.225)$; hence Figs. 5a, b.

APPLICATIONS

Fig 6 shows some results of applying Eq. 13 to define the design responses of a fixed-end arch when both supports move out of phase. Responses to vertical and horizontal ground displacements are analyzed separately. In each case, results are expressed in terms of both the ratio τ_q/s of the time-lag between support displacements to the duration of the white-noise segment and the apparent wave propagation velocity along the ground surface. Both qualitative and quantitative deviations with respect to the case of in-phase support motion are evident.

Another case of interest is shown in Fig. 7, which represents a bridge built of simply supported spans resting on columns with ends fixed on the ground surface. Variables studied include bending moments at column bases and tensions on tie-bars connecting beam spans. Again, the sensitivity of design responses to phase differences is obvious.

CONCLUSIONS

The seismic response of extended-in-plan structures can be very sensitive to phase differences among the motions of different points in the foundation. Under the assumption of linear behavior it is possible to formulate approximate criteria to obtain design values and response superposition models which account for all ground motion components. The resulting expressions are determined by the covariance structure of the ground motion histories at the different supports.

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APPENDIX. VARIANCE INTEGRALS FOR SHIFTED IDENTICAL WHITE NOISE COMPONENTS

Analytical expressions for I_{qr}, I'_{kqr}, I''_{jkqr}, according to Eqs. 12a - c:

$$I_{qr}(t)=\frac{1}{3}(t^3-t_1^3)-\frac{1}{2}(z_q+z_r)(t^2-t_1^2)+z_q z_r(t-t_1) \tag{A1}$$

$$\begin{aligned} I'_{kqr}(t)=\frac{1}{D_1^2\omega'_k}\Big\{ & e^{-\zeta_k\omega_k(t-z_r)}\Big[\big(B_1(1+\zeta_k\omega_k t)-A_1\omega'_k t-\omega'_k z_q D_1\big)\cos\omega'_k(t-z_r)+\\ & +\big(A_1(1+\zeta_k\omega_k t)+B_1\omega'_k t-\zeta_k\omega_k z_q D_1\big)\sin\omega'_k(t-z_r)\Big]\\ & -e^{-\zeta_k\omega_k(t_1-z_r)}\Big[\big(B_1(1+\zeta_k\omega_k t_1)-A_1\omega'_k t_1-\omega'_k z_q D_1\big)\cos\omega'_k(t_1-z_r)+\\ & +\big(A_1(1+\zeta_k\omega_k t_1)+B_1\omega'_k t_1-\zeta_k\omega_k z_q D_1\big)\sin\omega'_k(t_1-z_r)\Big]\Big\} \end{aligned} \tag{A2}$$

$$\begin{aligned} I''_{jkqr}(t)=\frac{e^{\gamma}}{2\omega'_j\omega'_k}\Big\{ & e^{-At}\Big[\frac{A\cos(Z_1-W_1t)+W_1\sin(Z_1-W_1t)}{A^2+W_1^2}-\\ & -\frac{A\cos(Z_2-W_2t)+W_2\sin(Z_2-W_2t)}{A^2+W_2^2}\Big]-\\ & -e^{-At_1}\Big[\frac{A\cos(Z_1-W_1t_1)+W_1\sin(Z_1-W_1t_1)}{A^2+W_1^2}-\\ & -\frac{A\cos(Z_2-W_2t_1)+W_2\sin(Z_2-W_2t_1)}{A^2+W_2^2}\Big]\Big\} \end{aligned} \tag{A3}$$

In these equations,

$t_1=\max(z_q, z_r)$	$A=\zeta_j\omega_j+\zeta_k\omega_k$
$A_1=\zeta_k^2\omega_k'-\omega_k'^2$	$Z_1=\omega'_j z_q+\omega'_k z_r$
$B_1=2\zeta_k\omega_k\omega'_k$	$Z_2=\omega'_j z_q-\omega'_k z_r$
$D_1=\zeta_k^2\omega_k^2+\omega_k'^2$	$W_1=\omega'_j+\omega'_k$
$\gamma=\zeta_j\omega_j z_q+\zeta_k\omega_k z_r$	$W_2=\omega'_j-\omega'_k$

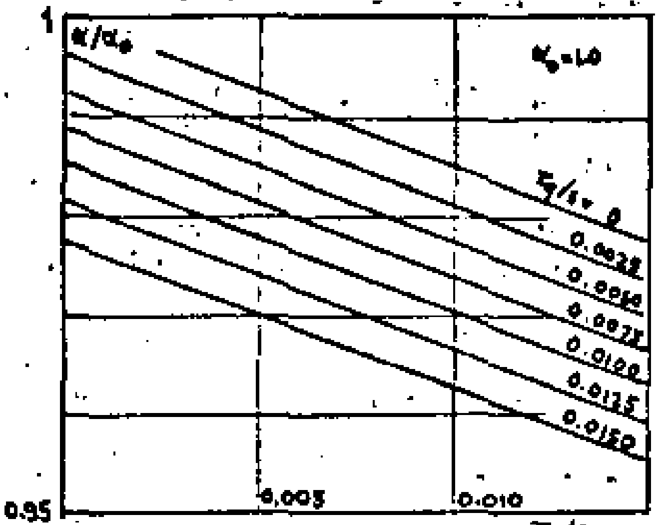

Fig. 1 Values of α

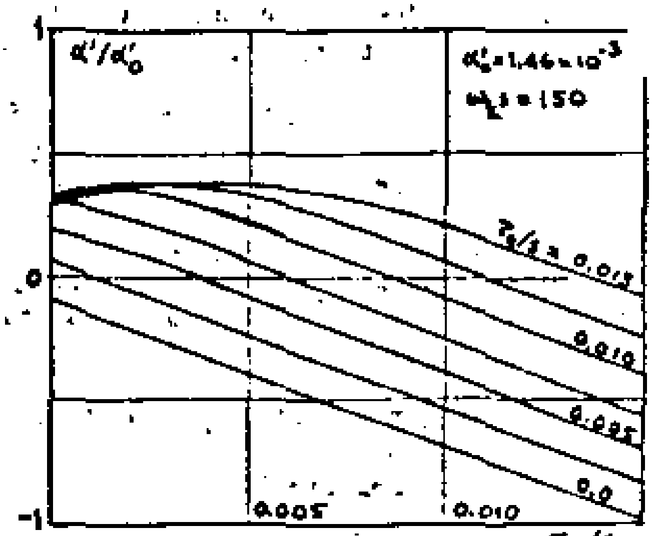

Fig. 2 Values of α'

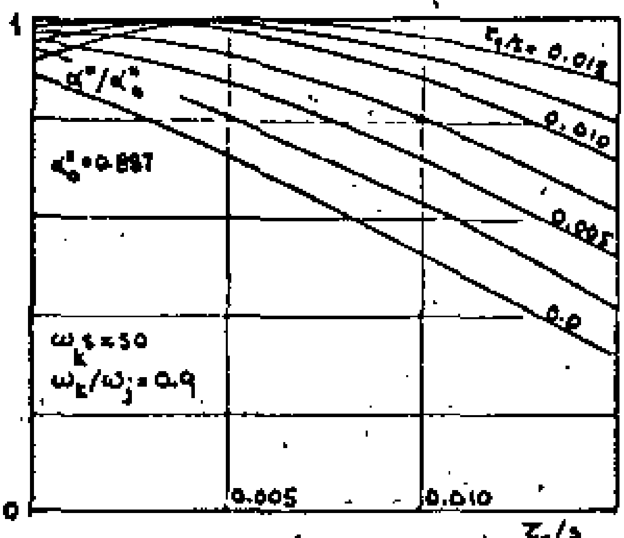

Fig. 3 Values of α''

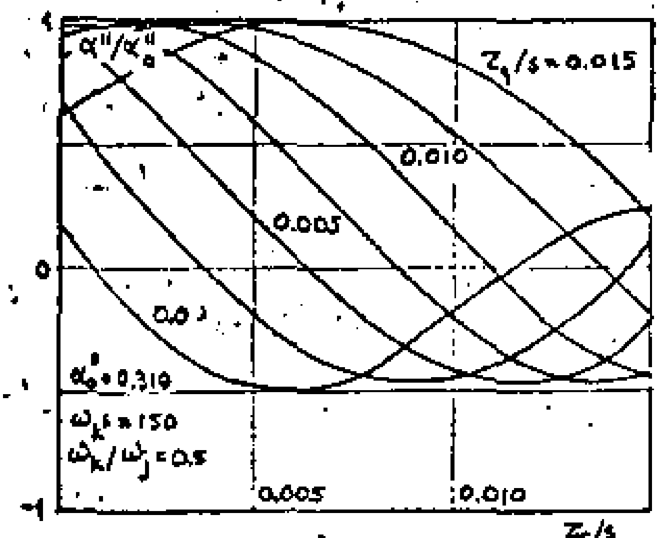

Fig. 4 Values of α''

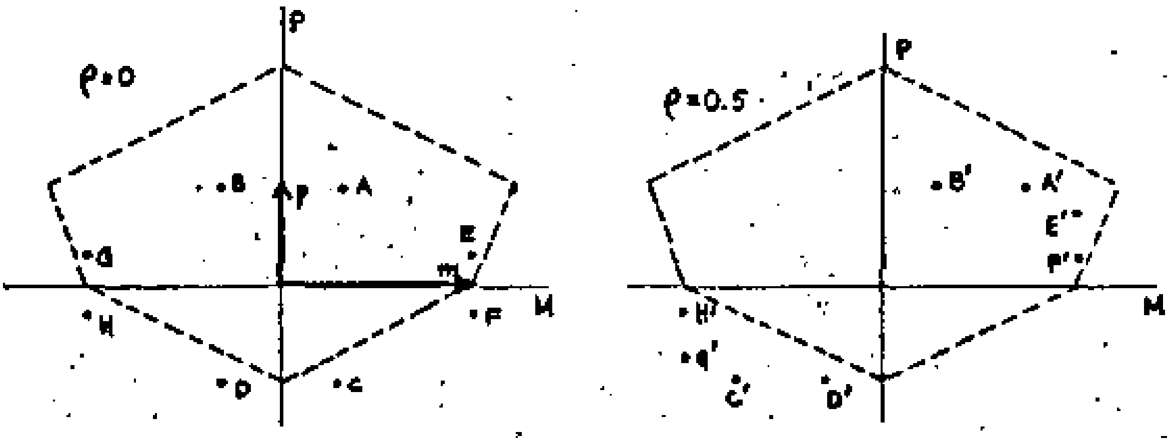

Fig. 5 Superposition of correlated responses

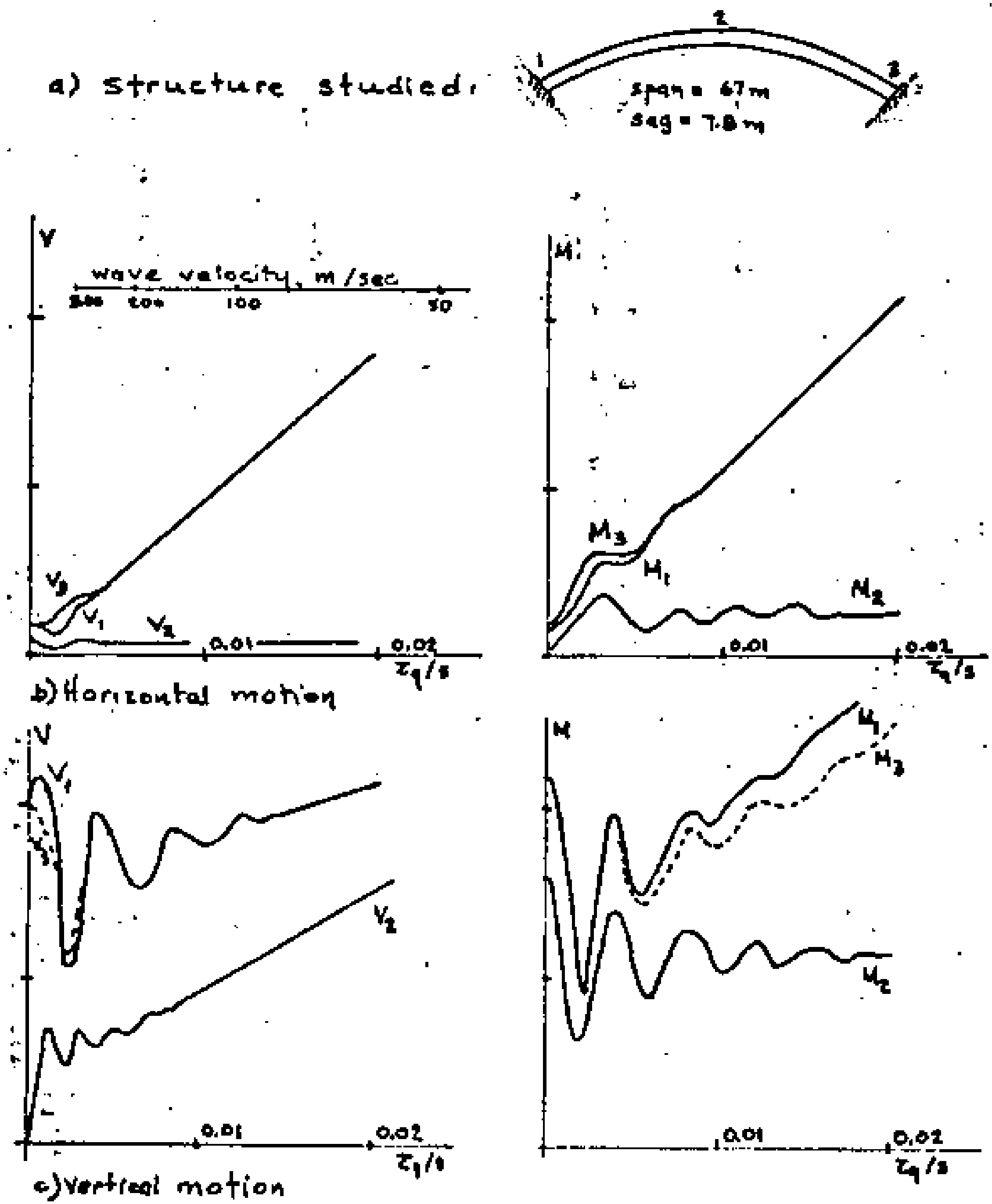

Fig.6 Influence of phase differences in the seismic response of an arch

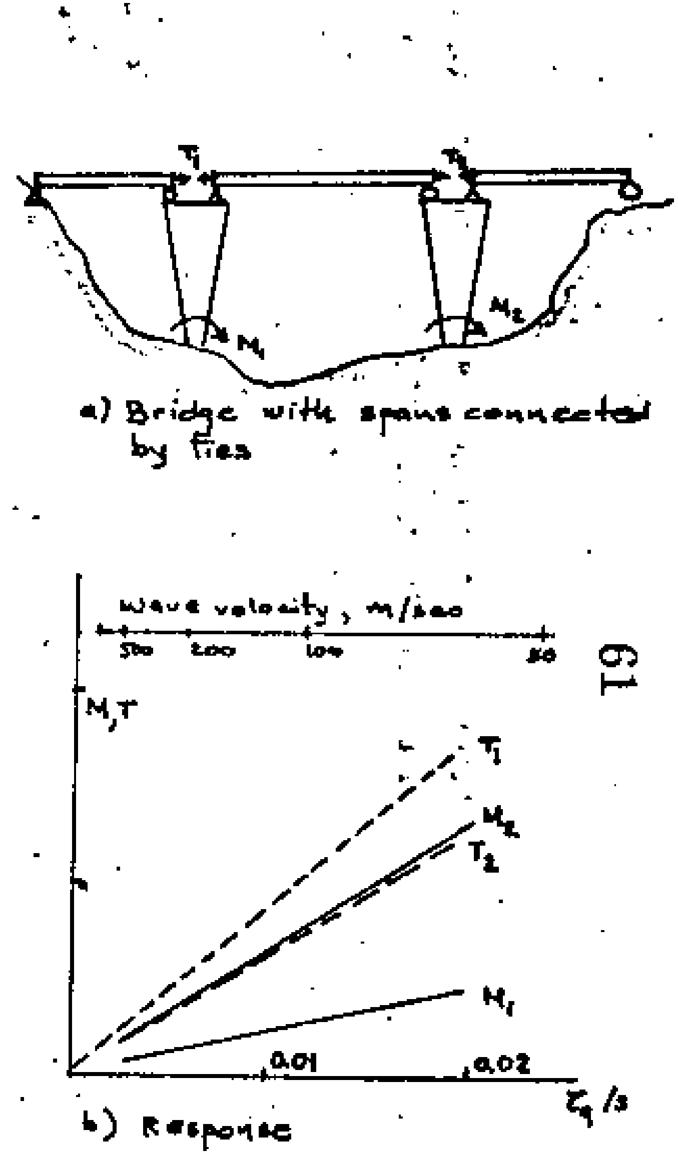

Fig.7 Bridge response

INFLUENCIA EN LA RESPUESTA SISMICA DEL PUENTE COATZACOALCOS II DE LAS DIFERENCIAS DE FASE EN LOS MOVIMIENTOS DE SUS APOYOS

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INTRODUCCION

La Secretaría de Asentamientos Humanos y Obras Públicas proyecta construir un puente de 772 m de largo total sobre varios apoyos, sobre el río Coatzacoalcos, en la carretera Nuevo Teapa-Minatitlán en el Estado de Veracruz. Las características geométricas más importantes del puente se extrajeron del plano 4775 proporcionado por SAHOP modificando las longitudes del claro central a 288m, y de los dos adyacentes a 98.35m cada uno. Estos tres claros están suspendidos de las dos torres centrales mediante sendos sistemas de cables atirantados colocados en el plano longitudinal de simetría del puente. Los apoyos se desplantan sobre zapatas y pilas que atraviesan las formaciones sedimentarias del fondo del cauce, y que se apoyan a su vez sobre terreno firme a profundidades que fluctúan entre 6.6 y 30 m.

La configuración del puente y los valores de las distancias entre apoyos hacen pensar en la posibilidad de que las diferencias de fase entre los movimientos de dichos apoyos afecten en forma importante las respuestas sísmicas de diseño en algunas secciones críticas, en relación con las que se obtendrían bajo la hipótesis de que los movimientos de las bases ocurren en fase. La importancia del efecto se acentúa si se tiene en cuenta que en el sitio de interés el riesgo sísmico depende en gran parte de temblores de gran magnitud a distancias hipocentrales grandes, y que por tanto las ondas superficiales contribuirán de manera importante a la excitación sísmica. Además teniendo en cuenta que los elementos de apoyo de los claros centrales atraviesan formaciones sedimentarias, las velocidades de propagación de las ondas superficiales serán relativamente bajas, lo que habrá de traducirse en mayores diferencias de fase.

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El criterio que en este estudio se adopta se basa en la ref 1; es de tipo probabilístico y supone que los movimientos de los distintos apoyos son iguales en forma, pero difieren en los tiempos de desfasamiento; es decir, que se deben a un tren de ondas que viaja sin distorsionarse a lo largo de la superficie del terreno. Se adopta este modelo en ausencia de información experimental (registros de temblores reales) sobre las relaciones entre las características del movimientos del terreno en sitios cercanos.

SELECCION DE LA RESPUESTA DE DISEÑO

Según la ref 1 la selección de los valores de diseño de las respuestas se basa en la proporcionalidad aproximada entre las respuestas correspondientes a probabilidades dadas de excedencia y las desviaciones estándar de dichas respuestas al final de la excitación aleatoria que se emplea para representar al movimiento del terreno. A fin de obtener expresiones simples para las desviaciones estándar que se mencionan, en la ref 1 se introducen algunas hipótesis simplificadoras, entre las que destaca la que consiste en suponer que en cada apoyo el acelerograma es un segmento de ruido blanco con duración δ e intensidad $S(\omega) = S_0$; la variación de s con ω para fines de considerar adecuadamente la contribución de cada uno de los modos naturales de vibración a la variancia de la respuesta se toma en cuenta mediante la introducción de los factores de proporcionalidad α, α' y α'' que se definen en las ecs 2a-c.

En resumen, las hipótesis citadas conducen, de acuerdo con la ref 1, a la siguiente expresión para determinar el valor de diseño de la respuesta Q:

$$Q^2 = \sum_q \sum_r \{Q_q Q_r \alpha_{qr} D_{oq} D_{or} + 2\sum_k Q_q a_{kr} Z_k \alpha'_{kqr} D_{oq} D_{kr} + \sum_j \sum_k a_{jq} a_{kr} Z_j Z_k \alpha''_{jkqr} D_{jq} D_{kr}\} \quad (1)$$

En esta ecuación, D_{oq} y D_{or} son los valores máximos (en este caso iguales) del desplazamiento del terreno en las bases de los apoyos q y r, respectivamente; D_{jq} y D_{kr} son las ordenadas del espectro de desplazamiento en dichos apoyos para los modos j y k; Z_j y Z_k son los valores que toma la respuesta de interés para las configuraciones modales j y k; a_{jq} y a_{kr} son los factores de participacion de los modos j y k para las configuraciones que resultan de aplicar gradualmente (en forma estática) desplazamientos unitarios a los apoyos q y r, respectivamente; Q_q y Q_r son las respuestas estáticas a dichos desplazamientos unitarios y α_{qr}, α'_{kqr}, α''_{jkqr} son factores de proporcionalidad que se obtienen como sigue:

$$\alpha_{qr} = I_{qr}(s)/(J_{qq}J_{rr})^{1/2} \tag{2a}$$

$$\alpha'_{kqr} = I'_{kqr}(s)/(J_{qq}\ J''_{kr})^{1/2} \tag{2b}$$

$$\alpha''_{jkqr} = I''_{jkqr}(s)/(J''_{jq}\ J''_{kr})^{1/2} \tag{2c}$$

Aquí, $I_{qr}(s)$, $I'_{kqr}(s)$, $I''_{jkqr}(s)$, en donde s es la duración del segmento de ruido blando que representa a la excitación, resultan al formular expresiones para las variancias de las respuestas de interés incluyendo las componentes estáticas y dinámicas. Las funciones I se refieren a las covariancias entre las respuestas estáticas producidas por los desplazamientos de los diversos apoyos, las I'' a las covariancias entre las respuestas dinámicas asociadas con los distintos apoyos y modos naturales de vibración y las I' a las covariancias entre las respuestas estáticas y las dinámicas. J_{qq} y J_{rr} son los valores de $I_{qq}(s)$ e $I_{rr}(s)$ cuando el tiempo se mide desde el instante en que el movimiento del terreno se inicia en los apoyos q y r respectivamente, y J''_{jq}, J''_{kr} son los valores correspondientes de I''_{jjqq} e I''_{kkrr} para los mismos orígenes del tiempo. En otras palabras, las contribuciones a las respuestas de diseño se expresan en términos de los desplazamientos máximos del terreno en los distintos apoyos, así como de las respuestas modales a cada componente del movimiento del terreno como si actuara simultáneamente en todos los apoyos (1).

ESTRUCTURA ANALIZADA

Dada la complejidad del sistema estructural de interés y las aproximaciones y simplificaciones implícitas en el criterio de análisis propuesto, se consideró justificable llevar a cabo un estudio en un modelo simplificado de la estructura, con un número reducido de grados de libertad, pero que preservara los rasgos importantes que podrían ser significativos en la influencia de las diferencias de fase sobre las respuestas de diseño. Así, en el presente trabajo se determinan, para el modelo simplificado que se describe más adelante, los valores de las fuerzas internas de diseño en las diversas secciones críticas, obtenidas respectivamente para las condiciones en que se incluyen y en que se ignoran las diferencias de fase. Comparando ambos grupos de resultados es fácil deducir los incrementos que deben aplicarse a los valores de las fuerzas de diseño obtenidas mediante un análisis dinámico convencional de un modelo detallado de la estructura sujeto a movimiento simultáneo de sus apoyos.

Como se mencionó antes, sólo se estudiaron las respuestas a las componentes longitudinales y verticales del movimiento del terreno. Los sistemas de cables de la estructura real se sustituyeron por cables únicos con rigideces lineales equivalentes; estos elementos pueden tomar incrementos positivos y negativos de carga axial. Los elementos de flexión se sustituyeron por unos cuantos elementos finitos, las masas distribuidas se sustituyeron por unas cuantas concentraciones, y la interacción dinámica entre la cimentación y la estructura en el desplante del apoyo 18 se tomó en cuenta mediante resortes y amortiguadores cuyas constantes de obtuvieron de la ref 2. En los demás apoyos la cimentación se consideró infinitamente rígida.

A fin de reducir aun mas el número de grados de libertad y el número de apoyos que se mueven fuera de fase, el sistema simplificado se transformó en el de la fig 1, eliminando los claros y apoyos extremos, y representando sus rigideces en los nudos 1 y 22 de la fig 1 mediante resortes lineales y angulares cuyas constantes se obtuvieron a partir de los miembros iniciales mediante los criterios convencionales del análisis estructural lineal. Además, se supuso que los dos apoyos que quedaron en cada extremo de la nueva estructura simplificada de la fig 1 se movían en fase, como un solo apoyo, para fines de aplicar la ec 1 (fig 2).

La determinación de las respuestas estáticas Q_q, Q_r y dinámicas modales Z_j,

z_k, así como de los factores de participación a_{kr} se llevó a cabo empleando el programa SAP IV para calculadora digital (3). En las figs 3 a 7 se presentan las configuraciones de los primeros 5 modos naturales. El periodo fundamental resultó ser 1.97 seg, que se compara razonablemente bien con el de 2.56 seg, obtenido en la ref 2 empleando un modelo detallado de la estructura.

Para fines de calcular los coeficientes dados por las ecs 2a-c se supuso un coeficiente de amortiguamiento en cada modo igual a 0.2 del crítico. Este valor toma en cuenta la disipación de energía por comportamiento inelástico.

EXCITACION DE DISEÑO

Se adoptó para cada apoyo el espectro de diseño empleado en la ref 4. De las características de este espectro y las distancias a las fuentes sísmicas que más contribuyen al riesgo sísmico en el sitio de interés se concluyó que debería tomarse para el desplazamiento horizontal máximo del terreno un valor de 20 cm, y para el vertical de 14cm. Por otro lado, considerando la influencia de los sedimentos en las características de los temblores, se tomó s = 35 seg, en vez de valores comprendidos entre 15 y 20 seg, que normalmente se recomiendan para la duración efectiva de segmentos de ruido blanco que se emplean en aplicaciones similares a la presente cuando se trata de temblores en terreno firme (5).

Se analizó la influencia de las diferencias de fase que provienen de suponer trenes de ondas superficiales que viajan en una dirección paralela a la longitudinal del puente. Las velocidades de propagación consideradas fueron ∞ (movimiento en fase), 50, 100, 200, 300, 500, 1000, 2000 y 5000 m/seg. A fin de estimar la velocidad de las ondas superficiales aplicable al caso en estudio se analizaron varios modelos idealizados de la estratigrafía en el punto medio entre los dos apoyos centrales. El primer modelo consideró un estrato de 13 m de espesor con módulo dinámico de cortante igual a 1025 ton/m^2 y peso volumétrico de 1.7 ton/m^3, de acuerdo con la ref 6; por debajo de dicho estrato se consideró un medio semi-infinito con un módulo de cortante igual a 5 veces el de arriba, lo que es congruente con las rigideces reportadas en los primeros metros de la formación en que se apoya la cimentación. Considerando que es probable que a pocos metros se encuentren formaciones bastante más rígidas, se analizó un segundo modelo que difiere del primero únicamente en que la formación inferior es 20 veces más rígida que el estrato superior. Para ondas de Rayleigh con periodo de un segundo las respectivas velocidades de propagación resultaron 120 y 230 m/seg.

RESULTADOS

Las tablas 1 y 2 resumen los resultados de los estudios efectuados para movimiento horizontal y vertical respectivamente. La última columna corresponde al caso en que los apoyos se mueven en fase.

El número que precede a cada grupo de seis renglones es el de un miembro, de acuerdo con la numeración de la fig 1. Los seis renglones corresponden a fuerzas axiales, fuerzas cortantes y momentos flexionantes en los extremos izquierdo y derecho o superior e inferior, en unidades de ton y ton/m.

Los dos últimos renglones de cada tabla corresponden a las tensiones de los cables, elementos 7 y 8 de la fig 1.

En las tablas 1 y 2 se observa que en el caso vertical la mayor parte de los miembros las diferencias de fase pueden tener un efecto apreciable sobre las fuerzas internas de diseño, en ciertos casos incrementándolas y en otros reduciéndolas. El efecto citado es muy sensible a las velocidades efectivas* de propagación de las ondas en la dirección paralela al puente. Las diferencias de fase en el movimiento horizontal no ocasionan amplificaciones importantes.

DISCUSION Y RECOMENDACIONES

La información disponible sobre características del terreno en los sitios de

* Se usa aquí este término para tener en cuenta que una onda que viaja con una velocidad *nominal*, en dirección oblicua al eje del puente, tiene una cierta velocidad *efectiva* en la dirección paralela a dicho eje.

los apoyos cubre solo las capas de material blando y unos cuantos metros de material de rigidez moderada que yace debajo de ellas (en total, una profundidad de 35m); por ello no es posible estimar con precisión las velocidades de propagación de las ondas superficiales. Los cálculos de la sección anterior muestran que las velocidades en cuestión pueden encontrarse entre 120 y 230 m/seg, de acuerdo con las hipótesis que se hagan relativas a las características del terreno por debajo de los 35 m de profundidad.

A la incertidumbre sobre las velocidades nominales de propagación de ondas superficiales debe sumarse la asociada con el modelo adoptado y con la hipótesis conservadora que consiste en considerar que las ondas viajan precisamente en la dirección del puente. Si en vez de esto se considerara, por ejemplo, un ángulo de incidencia de 45° con respecto al plano longitudinal de simetría, la velocidad efectiva se obtendría multiplicando la nominal por la secante de 45°, es decir, por 1.41.

Los resultados que se presentan provienen de un análisis de respuesta dinámica lineal. De acuerdo con la práctica usual de diseño sísmico, pueden tomarse como valores de diseño si provienen de espectros reducidos que correspondan al nivel de ductilidad que la experiencia aconseja para el tipo de estructura en estudio.

Sin embargo, es razonable esperar que el comportamiento no lineal y la correspondiente redistribución de esfuerzos contribuyan a desenfatizar la influencia de las diferencias de fase. En la literatura técnica no se cuenta con información cuantitativa sobre este efecto.

En vista de las incertidumbres citadas, y tomando en cuenta consideraciones económicas, se propone tomar como respuestas de diseño las que muestran las tablas 1 y 2 para velocidades de propagación de ondas de 300 m/seg. Dichas respuestas en ningún caso deberán tomarse menores que las que corresponden a desplazamiento en fase en todos los apoyos, según el criterio convencional; por lo tanto, en este estudio sólo se tendrán incrementos en las respuestas ocasionadas por el defasamiento en el movimiento vertical.

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TABLA 1

MOVIMIENTO LONGITUDINAL

L.M/S	50	100	200	300	500	1000	2000	5000	EN FASE
	15.52	11.39	15.99	15.31	14.96	20.47	23.07	23.94	24.11
	15.52	11.39	15.89	15.31	14.96	20.47	23.07	23.94	24.11
	30.59	25.20	17.97	17.25	15.71	20.89	23.88	24.63	24.50
	30.89	25.20	17.97	17.35	15.71	20.89	23.88	24.63	24.50
	384.06	312.75	222.52	215.69	195.56	258.86	295.95	305.42	303.03
	1489.50	1205.64	856.03	825.63	746.90	994.49	1136.95	1172.67	1166.27
	139.48	101.59	142.55	115.31	105.13	174.42	200.70	209.30	211.75
	139.48	101.57	142.55	115.31	105.13	174.42	200.70	209.30	211.75
	275.42	197.19	212.50	203.60	193.78	264.81	304.95	319.61	322.98
	275.42	197.19	212.50	203.60	193.79	264.81	304.95	319.64	322.98
	4256.84	2832.56	3200.40	3027.39	2820.41	3994.35	4601.05	4823.59	4875.92
	4897.17	3510.75	3417.40	3310.24	3180.16	4250.53	4893.47	5127.77	5177.87
	257.21	130.10	117.70	98.42	90.92	133.22	151.83	158.09	159.56
	257.21	130.10	117.70	98.48	90.92	133.22	151.93	158.09	159.56,PARE.
	145.56	97.84	140.99	117.16	108.30	173.61	199.94	208.99	211.55
	145.56	97.81	140.99	117.16	108.30	173.61	199.94	208.99	211.55
	3804.15	2112.26	2710.13	2376.38	2431.31	3431.32	4157.60	4337.47	4442.92
	1341.44	1243.19	1310.64	1047.73	913.62	1593.33	1825.56	1899.52	1899.52
	170.91	126.45	173.55	141.90	129.20	196.46	227.81	239.59	241.03
	170.91	126.45	173.55	141.90	129.20	196.46	227.81	239.59	241.03
	38.58	41.44	31.64	30.03	27.23	38.00	43.27	44.46	45.15
	38.58	41.44	31.64	30.03	27.23	38.00	43.27	44.46	45.15
	1341.44	1243.19	1310.64	1047.73	913.62	1593.33	1823.56	1889.52	1899.52
	1752.83	1911.76	1566.67	1656.02	1591.19	2005.10	2295.41	2304.62	2303.25
	0.11	0.12	0.12	0.14	0.12	0.09	0.05	0.02	0.01
	0.11	0.12	0.12	0.14	0.12	0.08	0.05	0.02	0.01
	26.47	23.27	28.97	28.22	23.75	33.80	39.13	41.36	42.20
	26.47	23.27	28.97	28.22	23.75	33.80	39.13	41.36	42.20
	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	505.46	443.20	551.96	537.34	452.11	642.77	745.40	787.81	803.91

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CONTINUACION TABLA 1

6									
AXI	223.64	207.94	226.19	160.37	121.42	259.99	303.87	315.78	317.21
AXJ	223.64	207.94	226.19	160.37	121.42	259.99	303.87	315.78	317.21
COI	157.90	157.36	141.20	146.51	125.59	185.40	216.41	226.98	228.64
COJ	157.90	157.36	141.20	146.51	125.59	199.40	216.41	226.98	228.64
MOI	506.46	443.68	551.86	537.56	452.41	643.93	745.49	787.81	807.91
MOJ	3175.21	3220.44	2734.99	2051.09	2669.51	3580.78	4158.68	4346.01	4347.85
7									
AXI	224.32	207.32	227.09	161.20	132.52	261.69	305.89	317.92	319.40
AXJ	224.32	207.32	227.09	161.20	132.52	261.69	305.89	317.92	319.40
COI	86.71	83.19	61.43	59.31	56.89	73.60	83.42	86.08	85.89
COJ	86.71	83.19	61.43	59.31	56.89	73.60	83.42	86.08	85.89
MOI	3175.21	3220.44	2734.99	2051.09	2669.51	3580.78	4158.68	4346.01	4347.95
MOJ	4771.85	4784.37	3858.31	3953.63	3746.78	4926.79	5753.53	5992.36	6006.49
8									
AXI	239.44	219.40	240.28	173.81	139.77	291.76	330.78	344.25	346.00
AXJ	239.44	219.40	240.29	173.81	132.77	291.76	330.78	344.25	346.00
COI	931.96	841.39	796.36	825.70	789.06	1043.43	1206.22	1262.68	1271.29
COJ	931.96	841.39	796.36	825.70	789.06	1043.43	1206.22	1262.68	1271.29
MOI	8961.87	9276.06	7320.96	7435.51	6791.04	9467.03	11047.66	11527.92	11555.07
MOJ	9877.54	7471.91	9110.32	8961.04	8716.61	11993.51	13691.35	14344.34	14495.09
9.PAGE.									
AXI	243.29	217.71	244.69	178.05	142.76	290.01	340.68	354.76	356.71
AXJ	243.29	217.71	241.69	178.05	142.36	290.01	340.68	354.76	356.71
COI	1013.06	928.30	887.44	921.73	875.93	1144.23	1348.27	1412.34	1422.38
COJ	1013.06	928.30	887.44	921.73	875.93	1144.23	1348.27	1412.34	1422.38
MOI	9877.54	7471.91	9110.32	8961.04	8716.61	11993.51	13691.35	14344.34	14495.09
MOJ	27224.44	23406.60	24250.91	24257.11	23973.52	31008.61	36722.61	38480.84	38709.09
10									
AXI	903.84	765.70	600.20	598.60	565.05	743.42	853.22	896.85	899.47
AXJ	903.84	785.70	600.20	598.60	565.05	743.48	853.22	896.85	899.47
COI	52.06	52.55	48.28	43.64	26.97	61.27	75.12	79.24	79.91
COJ	52.06	52.55	48.29	43.64	26.97	61.27	75.12	77.21	79.91
MOI	2457.82	2669.57	2152.82	2026.19	1571.37	2760.61	3319.54	3402.47	3501.78
MOJ	1515.02	1220.77	1510.46	1367.05	906.55	1091.10	2200.82	2427.55	2452.02
11									
AXI	767.29	520.91	393.43	386.41	371.25	454.97	511.78	530.38	530.94
AXJ	767.29	520.91	393.43	386.41	371.25	451.97	511.78	530.38	530.94
COI	21.30	17.98	22.68	21.60	11.73	21.41	30.00	31.83	32.32
COJ	21.30	17.98	22.68	21.60	11.73	21.41	30.00	31.83	32.32
MOI	1515.02	1220.77	1510.46	1349.05	906.55	1071.10	2200.82	2427.55	2452.02
MOJ	1756.73	1625.47	1526.38	1672.55	1061.55	1729.48	2115.71	2281.46	2289.63.PAGE.
CAB	256.57	179.22	[illegible]	221.77	[illegible]	[illegible]	268.23	321.21	357.40
CBD	254.18	266.38	220.37	171.22	122.72	229.59	305.73	317.58	318.24

TABLA 2

MOVIMIENTO VERTICAL

VEL.M/S	50	100	200	300	500	1000	2000	5000	EN FA…
1									
AXI	11.57	16.20	14.51	11.27	5.43	5.50	5.30	3.29	2.04
AXJ	11.57	16.20	14.51	11.27	5.43	5.50	5.30	3.29	2.04
COI	17.14	20.09	16.15	13.01	14.70	15.97	15.52	15.00	14.97
COJ	17.14	20.09	16.15	13.01	14.70	15.97	15.52	15.00	14.97
MOI	207.56	242.72	194.82	157.11	173.09	193.44	188.26	181.94	181.57
MOJ	821.04	962.48	774.28	623.57	604.70	764.79	743.19	718.34	715.54
2									
AXI	162.81	206.15	174.29	136.00	130.66	134.66	130.92	126.66	124.16
AXJ	162.81	206.15	174.29	136.00	130.66	134.66	130.92	126.66	124.16
COI	95.08	133.45	114.48	87.41	64.42	55.35	51.13	48.02	44.85
COJ	95.08	133.45	114.48	87.41	64.42	55.35	51.13	48.02	44.85
MOI	1645.70	2267.65	1947.24	1489.72	1140.63	1039.71	979.49	931.80	882.93
MOJ	1524.18	1893.07	1620.18	1235.86	850.29	692.67	616.24	563.18	512.89
3									
AXI	82.73	113.66	97.04	74.32	55.97	51.98	47.37	40.92	37.66
AXJ	82.73	113.66	97.04	74.32	55.97	51.98	47.37	40.92	37.66
COI	145.95	186.84	158.13	122.94	115.49	117.20	113.76	110.01	107.54
COJ	145.95	186.84	158.13	122.94	115.19	117.20	113.76	110.01	107.54
MOI	2354.69	3086.36	2620.70	2029.79	1502.40	1765.38	1704.92	1615.09	1595.10
MOJ	1816.05	2249.46	1880.23	1478.12	1491.07	1560.59	1527.29	1478.29	1456.93
4									
AXI	185.44	238.39	186.95	131.54	119.81	129.66	130.84	128.76	130.63
AXJ	185.44	238.39	186.95	131.54	119.81	129.66	130.84	128.76	130.63
COI	39.30	45.46	37.29	29.94	33.76	35.94	35.04	33.87	33.67
COJ	39.30	45.46	37.29	29.94	33.76	35.94	35.04	33.87	33.67
MOI	1816.05	2247.46	1880.23	1478.12	1491.03	1560.59	1527.29	1470.29	1456.93
MOJ	937.60	1053.75	822.75	651.32	800.99	961.67	932.89	896.51	903.41
5									
AXI	0.23	0.21	0.22	0.16	0.29	0.34	0.32	0.20	0.14
AXJ	0.23	0.21	0.22	0.16	0.29	0.34	0.32	0.20	0.14
COI	28.03	29.60	29.13	22.94	26.90	31.97	33.10	34.13	34.37
COJ	28.03	29.60	29.13	22.94	26.90	31.97	33.10	34.13	34.37
MOI	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MOJ	534.05	567.95	574.91	475.07	[illegible]	[illegible]	650.56	650.12	654.69
6									
AXI	292.42	377.85	[illegible]	[illegible]	[illegible]	195.77	200.95	204.49	211.52
AXJ	292.42	377.85	[illegible]	[illegible]	[illegible]	195.77	200.95	204.49	211.52
COI	45.09	54.72	76.17	71.81	73.71	77.17	74.27	75.92	74.90
COJ	45.07	54.72	76.17	71.81	73.71	77.17	74.27	75.92	74.90
MOI	584.05	583.95	611.71	[illegible]	[illegible]	[illegible]	650.54	650.12	654.49
MOJ	1292.35	1453.09	[illegible]	843.57	800.60	849.05	917.28	907.74	930.18

CONTINUACION TABLA 2

7									
AXI	292.09	378.69	293.81	210.[illegible]	199.75	195.18	200.85	206.62	211.48
AXJ	292.09	378.69	293.81	210.19	199.75	195.18	200.85	206.62	211.48
COI	51.38	62.30	42.46	73.90	36.19	37.36	38.29	39.35	40.27
COJ	51.38	62.30	42.46	73.90	36.19	37.36	38.29	39.35	40.27
MOI	1202.25	1459.09	995.74	842.53	899.69	949.05	917.29	907.76	930.18
MOJ	2167.46	2637.71	1789.29	1477.39	1559.56	1611.77	1613.42	1641.72	1693.44
6									
AXI	368.31	397.58	311.71	219.75	192.43	204.54	213.57	220.61	226.76
AXJ	368.31	397.58	311.71	219.75	192.43	204.54	213.57	220.61	226.76
COI	186.85	262.93	184.73	163.92	127.53	119.95	106.39	103.19	109.91
COJ	186.85	262.93	184.73	163.92	123.53	119.95	106.39	103.19	109.91
MOI	4730.50	5801.73	4149.29	3329.29	3423.83	3475.14	3427.57	3460.01	3577.59
MOJ	4118.16	5801.14	4909.06	3695.05	2412.59	1931.33	1701.05	1554.10	1443.99
9.PAGE.									
AXI	310.58	401.75	316.53	222.04	191.57	203.31	212.52	219.53	225.69
AXJ	310.58	401.75	316.53	222.04	191.57	203.31	212.52	219.53	225.69
COI	197.71	279.07	197.61	175.96	134.65	131.54	112.69	104.88	110.92
COJ	197.71	279.07	197.61	175.96	134.65	131.54	112.69	104.88	110.92
MOI	4118.16	5801.14	4909.06	3695.05	2412.59	1931.33	1701.05	1554.10	1443.99
MOJ	6067.77	9118.33	7461.32	5954.90	3403.00	2550.09	1611.55	779.81	534.21
10									
AXI	353.43	438.05	294.31	236.41	243.10	251.73	262.38	273.16	283.44
AXJ	353.43	438.05	294.31	236.41	243.10	251.73	262.38	273.16	283.44
COI	45.84	60.66	49.80	36.76	26.17	22.30	22.25	22.89	23.63
COJ	45.84	60.66	49.80	36.76	26.17	22.30	22.25	22.89	23.63
MOI	1967.19	2440.95	1508.56	1409.29	1210.76	1143.36	1193.90	1219.82	1253.49
MOJ	1430.60	1977.89	1697.06	1277.28	714.75	426.19	390.06	382.79	400.26
11									
AXI	156.55	200.16	125.64	113.09	104.76	98.51	114.29	142.26	155.30
AXJ	156.55	200.16	125.64	113.09	104.76	98.51	114.29	142.26	155.30
COI	16.15	25.34	21.96	17.50	10.04	6.99	2.95	1.67	1.09
COJ	16.15	25.34	21.96	17.50	10.04	6.99	2.95	1.67	1.09
MOI	1430.60	1977.89	1697.06	1277.29	714.75	426.19	390.06	382.79	400.26
MOJ	1476.57	2030.23	1692.47	1477.59	1072.36	753.60	642.66	585.72	550.05.PAGE.
CAB	263.42	321.69	293.77	220.60	196.56	190.75	190.53	192.07	194.03
CMB	290.92	367.69	279.[illegible]	[illegible]	187.70	209.[illegible]	220.62	221.10	239.24

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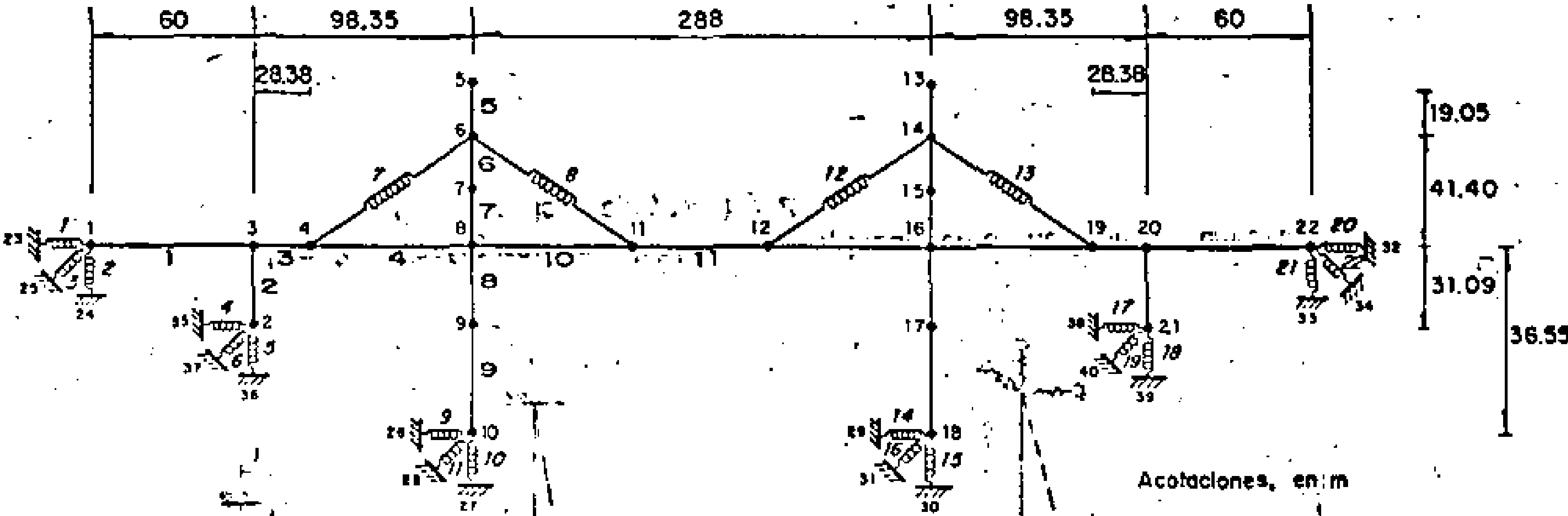

Fig 1. Modelo simplificado. Puente Coatzacoalcos II

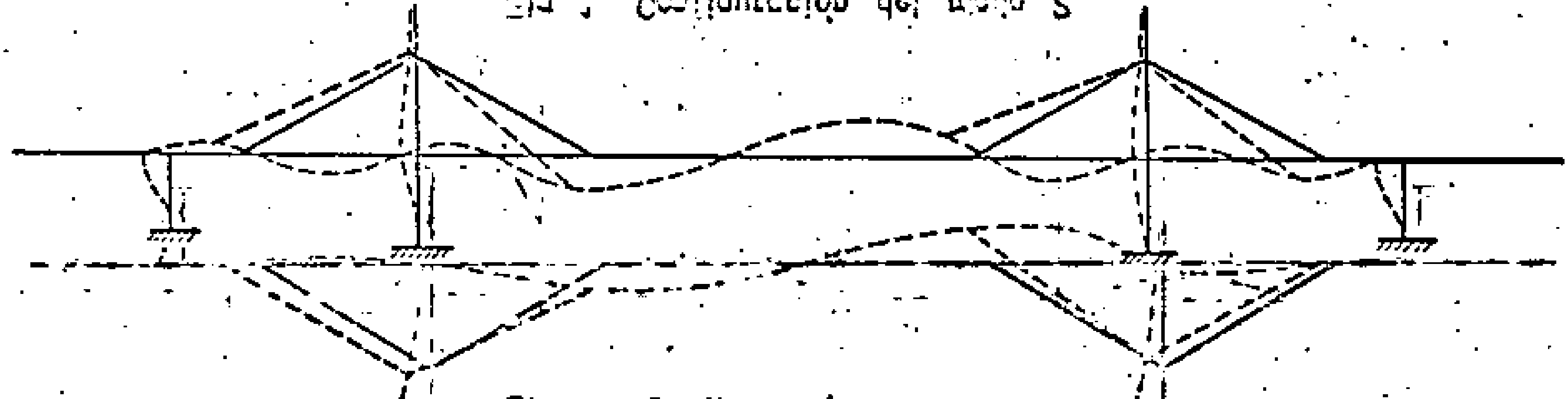

Fig 5. Configuración del modo 3

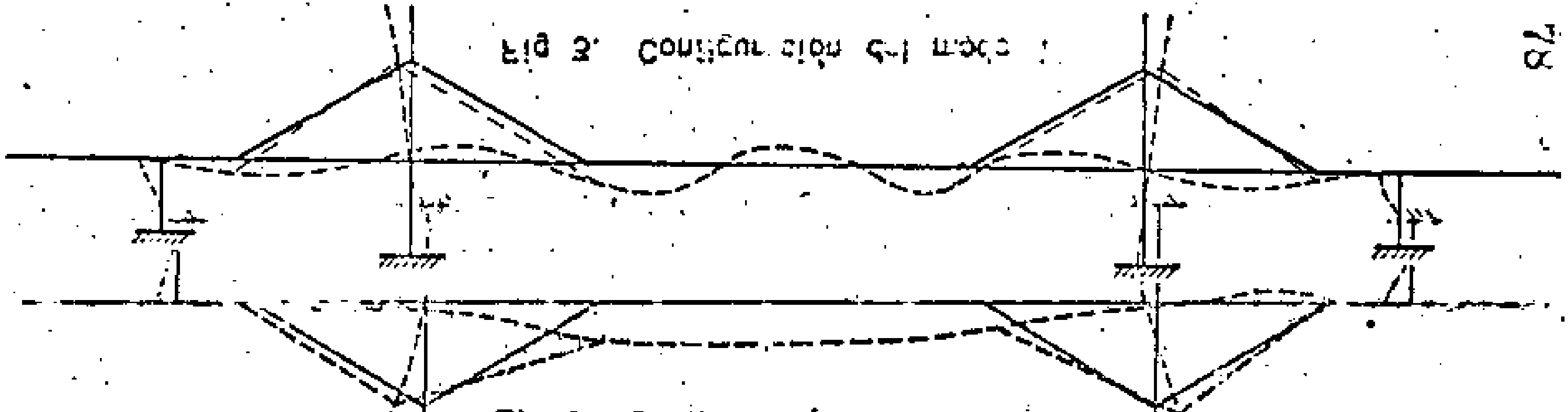

Fig 6. Configuración del modo 4

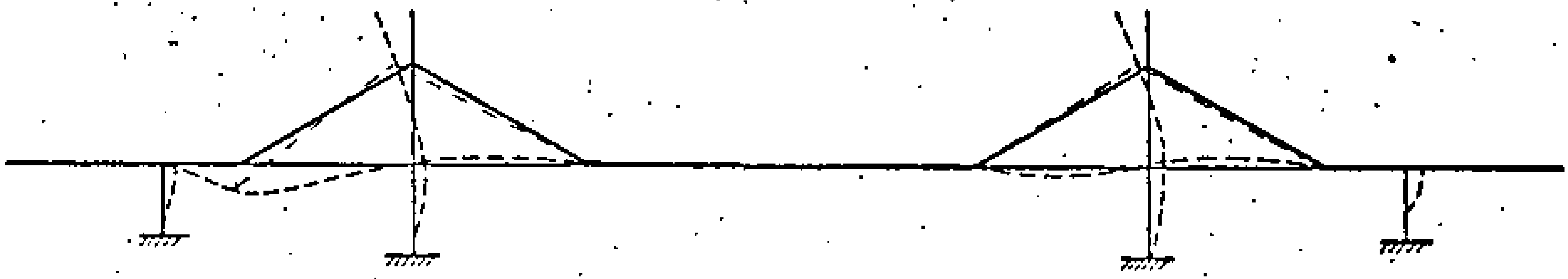

Fig 7. Configuración del modo 5

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INFLUENCIA DE LA RESPUESTA SISMICA DEL PUENTE COATZACOALCOS II DE LAS DIFERENCIAS DE FASE EN LOS MOVIMIENTOS DE SUS APOYOS

SEGUNDA PARTE

INFLUENCIA DE LA RESPUESTA NO LINEAL

L. Esteva*
D. de León**

Elaborado para

DIRECCION FEDERAL DE CARRETERAS FEDERALES
SECRETARIA DE ASENTAMIENTOS HUMANOS Y
OBRAS PUBLICAS

Proy 0754

DICIEMBRE 1980

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INFLUENCIA DE LA RESPUESTA SISMICA DEL PUENTE COATZACOALCOS II DE LAS DIFERENCIAS DE FASE EN LOS MOVIMIENTOS DE SUS APOYOS

SEGUNDA PARTE

INFLUENCIA DE LA RESPUESTA NO LINEAL

L. Esteva y D. de León

Antecedentes

En diciembre de 1980 el Instituto de Ingeniería entregó a la Secretaría de Asentamientos Humanos y Obras Públicas el informe de un estudio que realizó por su encargo sobre la influencia en la respuesta sísmica del puente Coatzacoalcos II de las diferencias de fase en los movimientos de sus apoyos (1). Los resultados mostraron que para el caso en que se consideran diferencias de fase en la componente vertical del movimiento de los apoyos se encuentran amplificaciones excesivas del momento en el apoyo central izquierdo en relación con el valor de dicho momento para el caso en que los apoyos se ven sometidos a movimiento vertical en fase. En consecuencia, la Secretaría citada solicitó que se revisaran los cálculos y resultados del informe antes mencionado.

Después de una revisión muy detallada y rigurosa de los cálculos se concluyó que estaban correctos y que representaban adecuadamente la respuesta dinámica del puente sujeto a excitaciones fuera de fase en sus apoyos bajo la hipótesis de comportamiento lineal. Se concluyó que dadas las características del sistema y de la excitación era indispensable obtener las fuerzas de diseño a partir de un análisis que considerara abiertamente el comportamiento no lineal de la estructura. Aquí se presentan los resultados de un análisis aproximado que incluye este concepto.

Respuesta no lineal de sistemas sujetos a movimientos fuera de fase en sus apoyos.

El diseño sísmico de estructuras considera que ante temblores intensos dichas estructuras responderán haciendo uso de su capacidad de disipar energía mediante comportamiento inelástico. Esto permite reducir los espectros de diseño teniendo en cuenta los factores de ductilidad que se consideran aceptables para cada tipo de estructura. Si los apoyos no se mueven en fase, no basta con reducir los espectros de diseño: es necesario reducir las rigideces de los miembros estructurales a fin de estimar con la misma aproximación y bajo un criterio unificado la parte de la respuesta que se debe al movimiento de los apoyos como si actuaran en fase y la que se debe a las diferencias de fase. Para ello se propuso el criterio de analisis que se describe a continuación.

Del estudio de la respuesta dinámica de sistemas con curva carga-deformación elasto-plástica se ha deducido el criterio para reducir los espectros de diseño en términos de la ductilidad que se emplea para las estructuras convencionales: en el intervalo de periodos naturales moderados y largos se reducen las ordenadas del espectro elástico dividiéndolas entre el factor de ductilidad Q, y para periodos cortos se impone la condición de que para T = 0 la ordenada del espectro de aceleraciones es igual a la aceleración máxima del terreno, a_0, independientemente del factor de ductilidad o del amortiguamiento. El espectro así obtenido es el que en la fig 1 se designa como espectro reducido por ductilidad: para emplearlo se entra con los periodos naturales de las estructuras calculados en función de las rigideces elásticas iniciales de todos sus miembros.

Para nuestros fines es necesario trabajar con un sistema equivalente con rigideces reducidas en forma inversamente proporcional a la rigidez tolerable (2). Los resultados de este análisis son congruentes con el criterio convencional del espectro elasto-plástico si se desarrolla un nuevo espectro, que en la fig se designa como espectro para sistema equivalente y que es igual al espectro reducido por ductilidad excepto porque la esca horizontal está transformada de manera que los periodos están multiplicados por $\sqrt{Q}$. A este espectro se entra con los periodos naturales equivalentes del sistema no lineal, que se obtienen tomando en cuenta las rigideces del sistema equivalente, es decir, las elásticas iniciales divididas entre el factor de ductilidad.

Resultados y conclusiones

Los resultados de analizar el sistema equivalente para $Q \doteq 4$ con los criterios descritos en el informe de diciembre de 1980 se muestran en la fig 2. En ella se observa que los factores de amplificación de la respuesta para excitación fuera de fase con respecto a la que se tiene en fase no exceden de 2 en las secciones de esfuerzos más elevados, aunque se encuentran valores mayores en zonas de esfuerzos pequeños o casi nulos. Los factores de amplificación de esta figura se consideran adecuados para diseño sísmico del puente bajo la hipótesis que es aceptable una ductilidad de 4.

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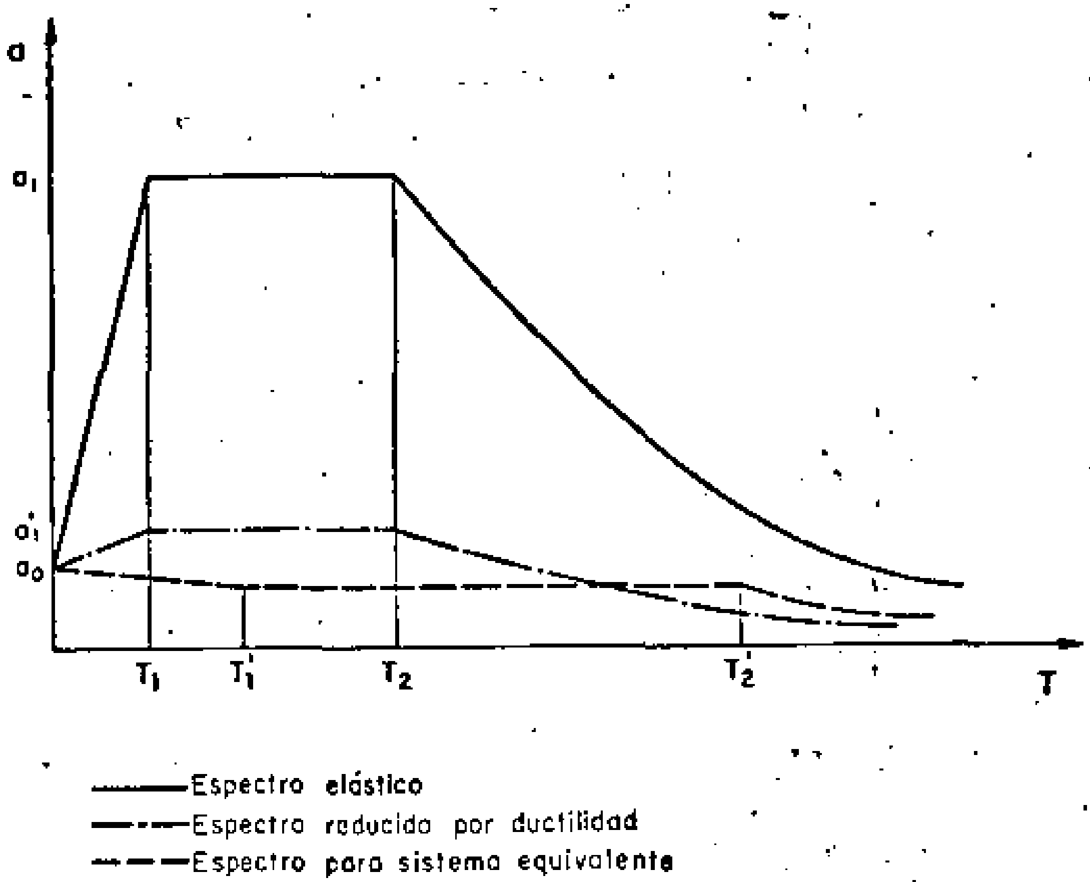
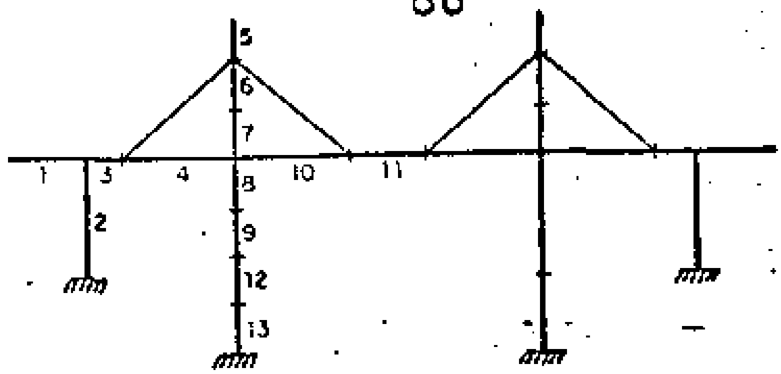


Fig 1. Espectros para diseño de estructuras inelásticas



Elemento	Fuerza interna	Factor de amplificación
1	3.49	1.06
	24.19	1.25
	419.00	1.25
	1032.00	1.25
2	166.7	1.35
	52.2	1.70
	1077.0	1.58
	546.0	1.95
3	58.46	1.27
	142.20	1.37
	2108.90	1.41
	1927.20	1.32
4	294.7	0.91
	45.5	1.28
	1927.0	1.32
	1257.0	1.23
5	1.12	0.78
	54.20	0.89
	0	0
	1032.00	0.89
6	396.0	0.90
	35.7	0.80
	1032.0	0.89
	703.0	1.11
7	398.0	0.90
	39.8	0.99
	703.0	1.11
	1170.0	1.15
8	405.0	0.85
	101.0	4.70
	2858.0	1.26
	2244.0	1.31
9	416.0	0.86
	85.0	4.19
	2244.0	1.31
	1407.0	2.19
10	399.0	0.93
	13.0	1.62
	683.0	1.30
	246.0	2.57
11	109.00	1.04
	0.38	23.50
	246.00	2.57
	301.00	2.80
12	407.0	0.85
	103.0	1.10
	1293.0	1.37
	260.0	4.07
13	392.0	0.85
	331.0	1.35
	990.0	1.12
	2500.0	1.58

Fig 2. Fuerzas internas y factores de amplificación

DIVISION DE EDUCACION CONTINUA
FACULTAD DE INGENIERIA U.N.A.M.

IX CURSO INTERNACIONAL DE INGENIERIA SISMICA

DISEÑO SISMICO DE ESTRUCTURAS ESPECIALES

INSTALACIONES ESPECIALES

M. en C. JORGE PRINCE ALFARO

INSTALACIONES ESPECIALES

* Earthquake design of structures with brittle members and heavy artificial damping by the method of direct integration

**: Seismic qualification tests of electric equipment for caorso nuclear plant: comments on adopted test prodedure and results

*** Las especificaciones sísmicas de la ENDESA para el equipo de alta tensión

Especificaciones sísmicas para el equipo de alta tensión (220 kv)

**** Countermeasures for earthquakes in the electric utility industry of japan

***** Electrical power and communication lifelines

Protecting a power lifeline against earthquakes

Advances in mitigating seismic effects on power systems

Aseismic design of 500kv air circuit breaker with friction dampers

julio, 1981

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EARTHQUAKE DESIGN OF STRUCTURES WITH BRITTLE MEMBERS AND HEAVY ARTIFICIAL DAMPING BY THE METHOD OF DIRECT INTEGRATION

D. A. Winthrop* and H. C. Hitchcock**

Introduction

This paper outlines an investigation by the New Zealand Electricity Department into possible methods of increasing the seismic strength of 220kV airblast circuit breakers. The circuit breaker was idealised as a two mass vibrating system whose behaviour in different earthquakes was examined by the method of direct integration of the equations of motion.

History

The circuit breakers had been purchased under a specification requiring a seismic design factor of 0.25g. However, earthquake damage to similar breakers in a number of countries, together with a more detailed understanding of the response of structures to earthquakes lead to the adoption by the New Zealand Electricity Department of standard design spectrum curves based on a set of single mass response spectra drawn up by Skinner[1].

These spectra were produced from the eight components of four well known earthquakes, El Centro 1940 and 1934, Olympia 1949 and Taft 1952. The records were scaled to the 1940 El Centro N-S size then their spectra were averaged and smoothed. The present N.Z.E.D. specification for equipment with brittle components requires them to withstand earthquake forces obtained from the appropriate design spectral curve with a factor of safety of at least two.

The La Ligua earthquake of 1965 (Chile), magnitude 7-7.5 on the Richter scale had its epicentre 80 miles from San Pedro substation where 8 airblast circuit breakers of a type in common use in New Zealand were extensively damaged. Out of a total of forty-eight 110kV support columns, thirty-eight were fractured at their base. Although the ground acceleration did not exceed an estimated 0.16g, the maximum response of the airblast heads was considered to be in the region 1.5-2.5g (ref.2). The New Zealand Electricity Department has over eighty of this type of breaker installed throughout the country, rated from 80kV to 245 kV. The maximum acceleration the 220kV circuit breaker porcelain columns can withstand is approximately 0.6g. The need for some form of strengthening of the columns became apparent after the San Pedro and similar incidents.

The manufacturer offered to supply vibration damping devices for fitting at the insulator column bases, but because of the type of construction, the existing porcelain columns would also require replacing. The total cost of modifying each circuit breaker was such that alternative means of increasing the circuit breaker seismic resistance was sought.

One such proposal was to provide the circuit breaker base with a more flexible mounting together with heavy viscous damping. An elementary treatment of this arrangement was described in reference 3 and this suggests that large values of viscous damping in the support structure could to an appreciable extent make up for the lack of damping in the porcelain columns. When computer facilities became available the present more detailed study, representing the circuit breaker as a two mass system was undertaken.

Method of Analysis of Circuit Breaker Response

Since the flexibility of the porcelain columns was not negligible and since their strength was limited, it was desired to obtain more definite information on the deflection and stressing of the porcelain by representing each pole of the breaker as a two mass system. The upper mass represents the three sets of airblast heads which are mechanically coupled, so that they move virtually as one body and the lower mass represents the compressed air tank which forms the base of the circuit breaker.

It is difficult to determine the earthquake response of a two mass system with widely varying damping by modal analysis. Instead we studied the response of the two mass system to four of the earthquake records used by Skinner in obtaining his original response design curves.

The Department has available a package computer program known as Continuous System Modeling Program, or C.S.M.P. This is designed to simulate any continuous systems represented by one or more linear differential equations. C.S.M.P. has facilities for both linear and quadratic interpolation of ground acceleration from the digitised earthquake records. The first method was used by us primarily to limit computer operating time. The machine used for the operation was an IBM360/40 shared jointly by the New Zealand Electricity Department and the Ministry of Works.

Scaling of Accelerograms

Several methods of measuring the 'size' of an earthquake have been used.

(a) according to maximum ground acceleration

(b) according to the mean value of the velocity response spectrum between 0.1 and 2.5 seconds natural period for a value of damp-

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ion appropriate to the structure being considered (Housner).

(c) according to the mean value of acceleration response spectrum between 0.1 and 2.5 seconds for 2 percent critical damping (Skinner).

(d) according to the r.m.s. value of the strong motion portion of the accelogram (Jennings).

(e) according to the mean value of the velocity spectrum for 20% damping between 0.3 and 3 cycles per second plotted logrithmically (Plichon ref. 4).

The scaling factors indicated by the different methods are shown in Table I. For our study we used the scaling factors generally as derived by Skinner but calculated from the period range applicable to our structure, namely 0.1 to 1.2 seconds. Table I shows that the difference between Skinners and the authors figures are small.

Application of Accelerograms

The two body arrangement representing the circuit breaker is shown in Fig. I. The equations of motion governing the systems are:

$$\ddot{X}_1 = -\ddot{Z} - \frac{X_1(K_1+K_2)}{M_1} - \frac{\dot{X}_1(C_1+C_2)}{M_1} + X_2\left(\frac{K_2}{M_1}\right) + \dot{X}_2\left(\frac{C_2}{M_1}\right)$$

$$\ddot{X}_2 = -\ddot{Z} + X_1\left(\frac{K_2}{M_2}\right) + \dot{X}_1\left(\frac{C_2}{M_2}\right) - X_2\left(\frac{K_2}{M_2}\right) - \dot{X}_2\left(\frac{C_2}{M_2}\right)$$

The maximum stress to which the columns is subjected are proportional to the maximum displacement of the circuit breaker head relative to the tank.

The structure was first subjected to the 1940 El Centro N-S acceleration record and the response of the airblast heads is shown on Fig III for different values of support structure stiffness and damping.

Scaled up versions of the other three earthquakes were then applied to the structure for the values of support structure stiffness most likely to be of interest. The circuit breaker head response is shown on Fig. IV.

Conclusions

(1) Fig II compares Skinners response spectrum for 2% damping with the response spectra recalculated by us for four of the eight individual earthquakes which were scaled up by Skinner when deriving the standard. It suggests that the use of these accelerograms in direct integration methods is reasonably equivalent to the use of the design spectrum but is perhaps slightly on the optimistic side.

(2) Fig III shows that the greatest reduction in loading on the porcelain columns was given by high flexibility and damping. Damping even as high as 0.6 of critical gives an improvement. The curves showing the response of airblast heads to four earthquakes in Fig IV indicate:

(a) that the scatter of response for the four earthquakes decreases for heavy damping and low spring stiffness.

(b) that with 12000 ft/lb stiffness in the support and damping between 0.4 and 0.6 of critical, the response of the circuit breaker top to an El Centro sized earthquake is likely to be below 0.4g.

(c) That with 24,000 lb/foot stiffness a [illegible] .35 damping which represents the recommendation of the earlier elementary study (3) based on the treatment of the circuit breakers as a single mass system, the response to two of the test earthquakes is as predicted by that study and to the other two is about 30% larger.

To limit the deflection of the circuit breaker due to wind forces, to about 1 inch at 90-100 m.p.h. it would be necessary to limit the stiffness of the supports to not less than 12000 lb/foot.

The results of this investigation confirm that by choosing supports of low stiffness and heavy viscous damping, a substantial improvement in circuit breaker seismic strength is possible. Preliminary investigations of the mechanical design indicate that the provision of these features is both practical and economic.

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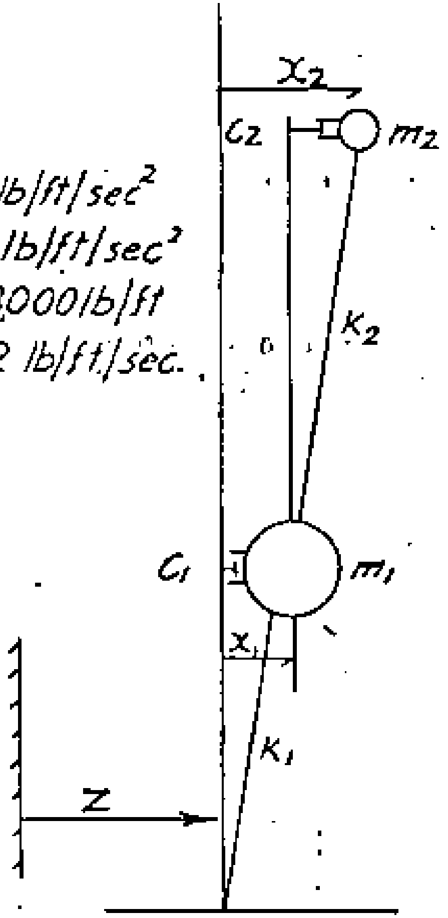
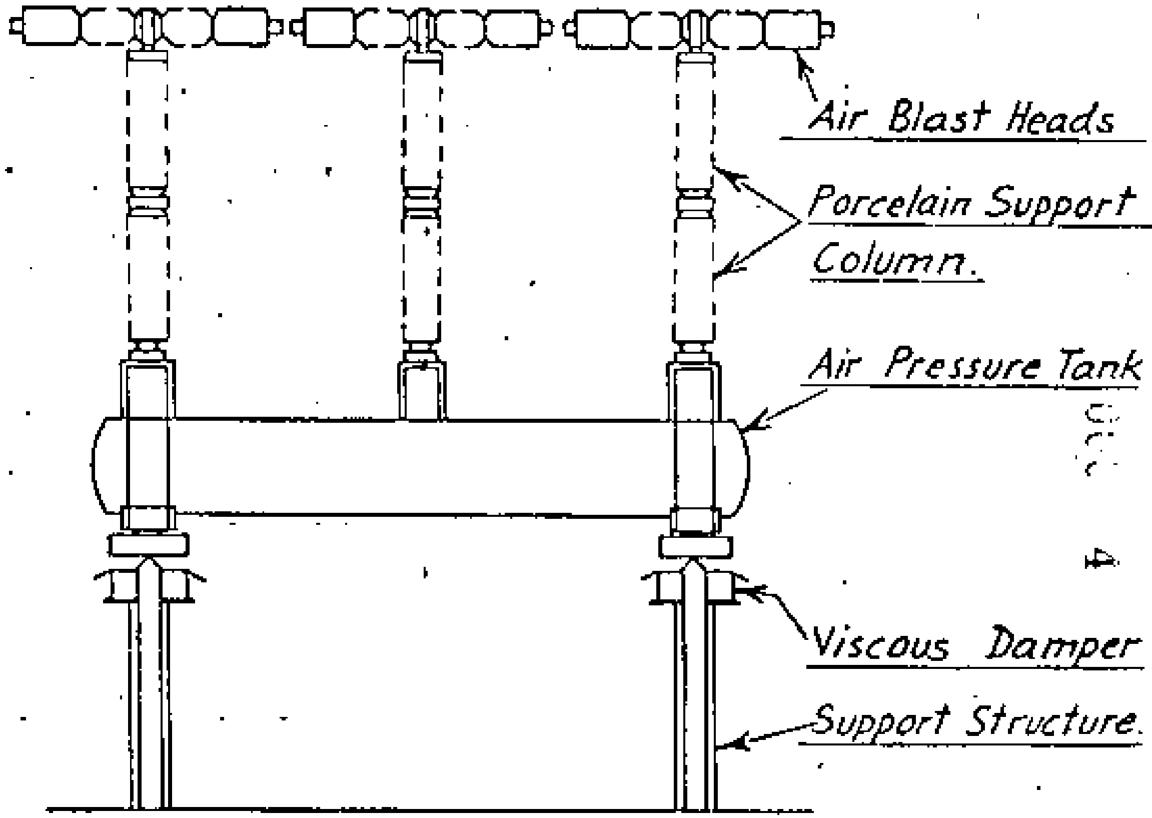
TABLE 1

	EL CENTRO 1940 N.S.	OLYMPIA 1949 N10W	OLYMPIA 1949 NE0E	TAFT 1952 S21W
Max Ground acceln	0.33	0.17	0.32	0.18
FACTOR	1.00	1.94	1.03	1.84
Spectral Intensity (Housner) Zero damping, 0.1-2.5 secs	6.94	5.39	6.03	4.53
FACTOR	1.0	1.6	1.48	1.97
Mean accel. response (Skinner) 2% damping 0.1-2.5 secs	1.3	0.74	0.91	0.59
FACTOR	1.00	1.75	1.42	2.19
Mean accel. response (Authors) 2% damping 0.1-1.2 secs	0.809	0.446	0.612	0.386
FACTOR	1.00	1.62	1.32	2.10
R.M.S. Accel. (Jennings)	2.20	1.57	1.95	1.42
FACTOR	1.00	1.4	1.13	1.55

$m_1 = 59\, lb/ft/sec^2$
$m_2 = 78\, lb/ft/sec^2$
$k_2 = 68{,}000\, lb/ft$
$c_2 = 92\, lb/ft/sec.$





Single Pole of Circuit Breaker

m_1, m_2 mass constants
k_1, k_2 stiffness constants
c_1, c_2 damping constants,
where $c = 2N\sqrt{m.k}$.
N is fraction of critical damping
x_1, x_2 displacements of m_1 and m_2.
z " of ground.

220kV. AIR BLAST CIRCUIT BREAKER

FIG. I

RESPONSE SPECTRA (0.02 DAMPING FACTOR)

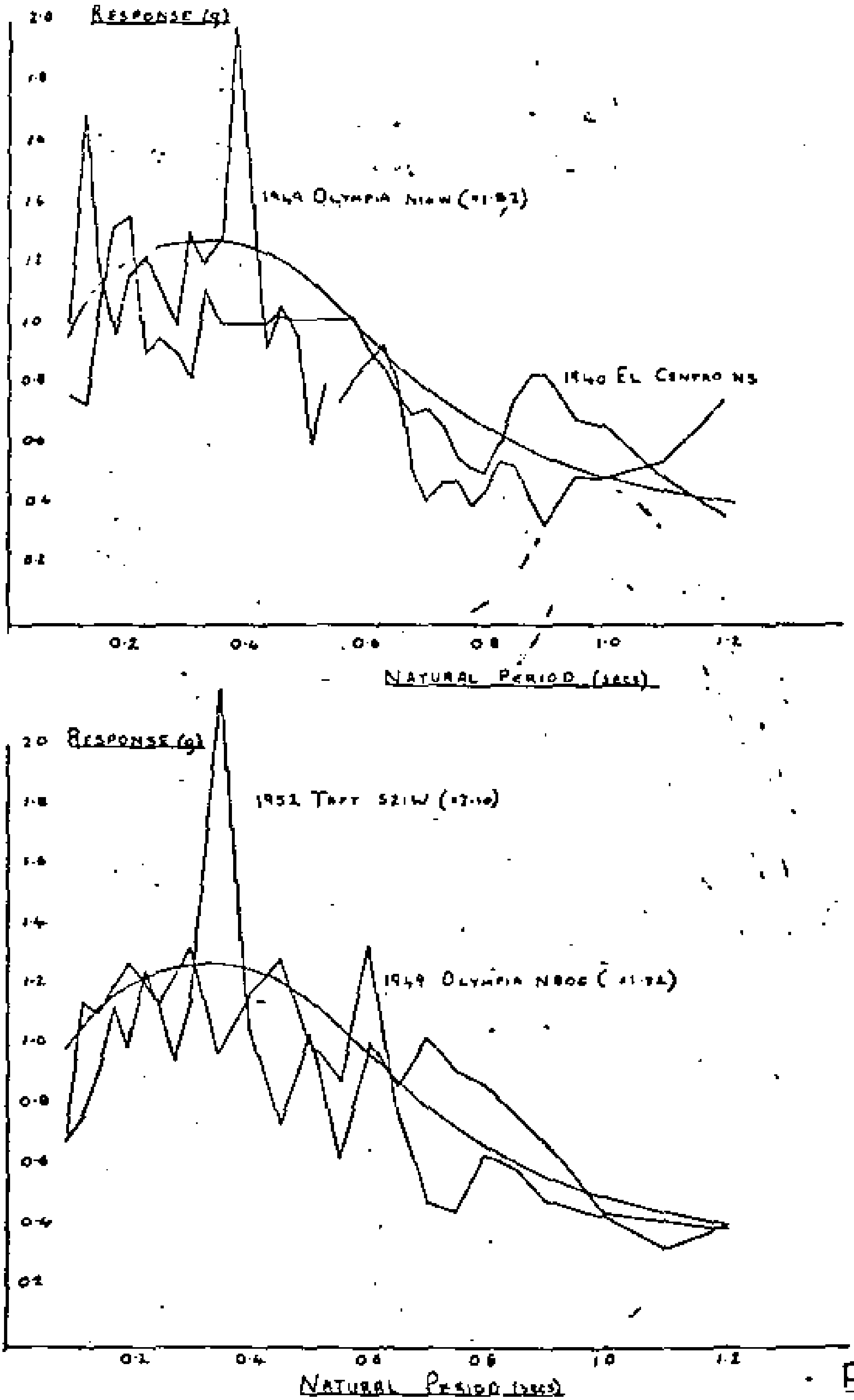


FIG II

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Response of airblast heads to 1940 El Centro conditions (N-S)

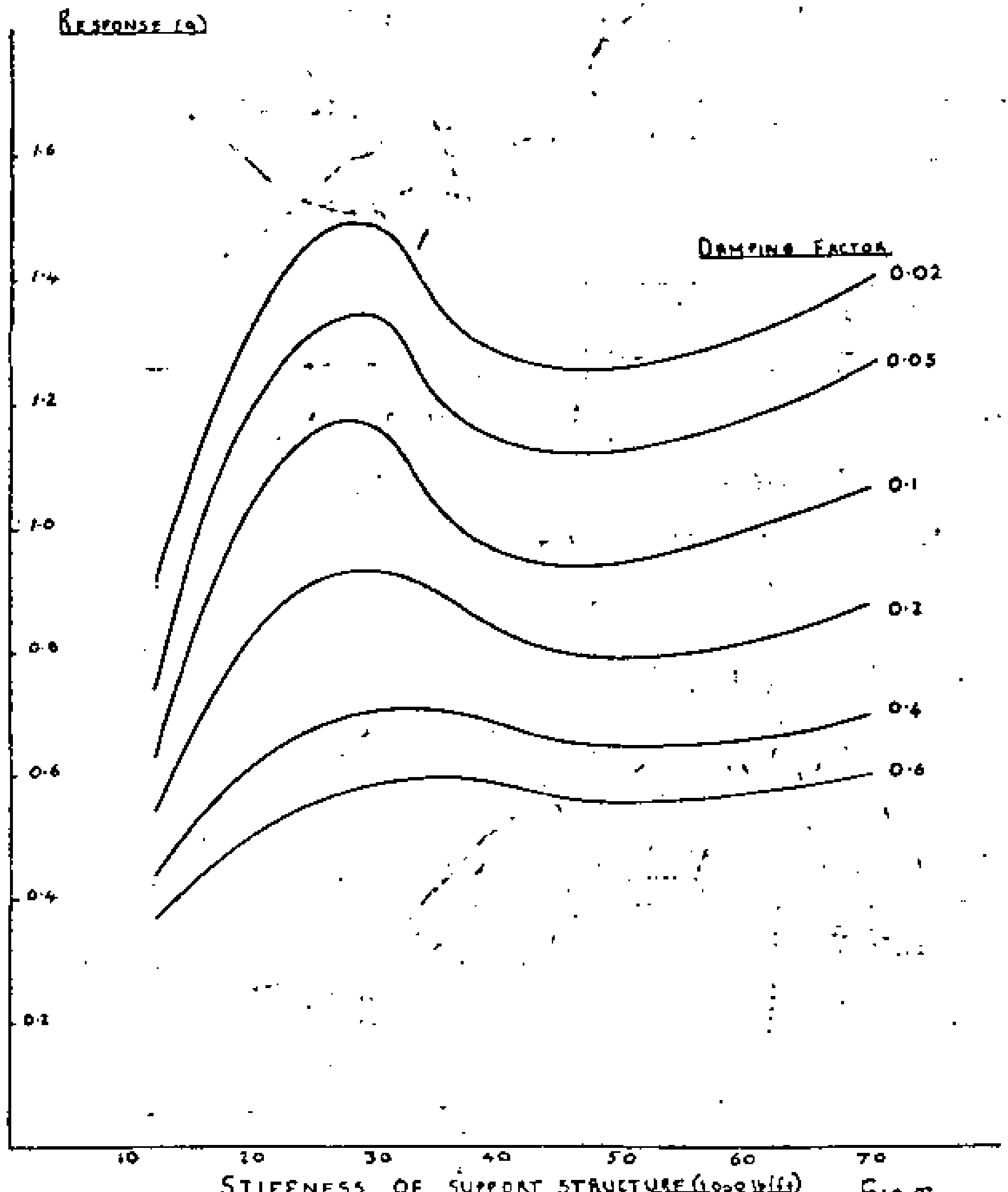

Fig III

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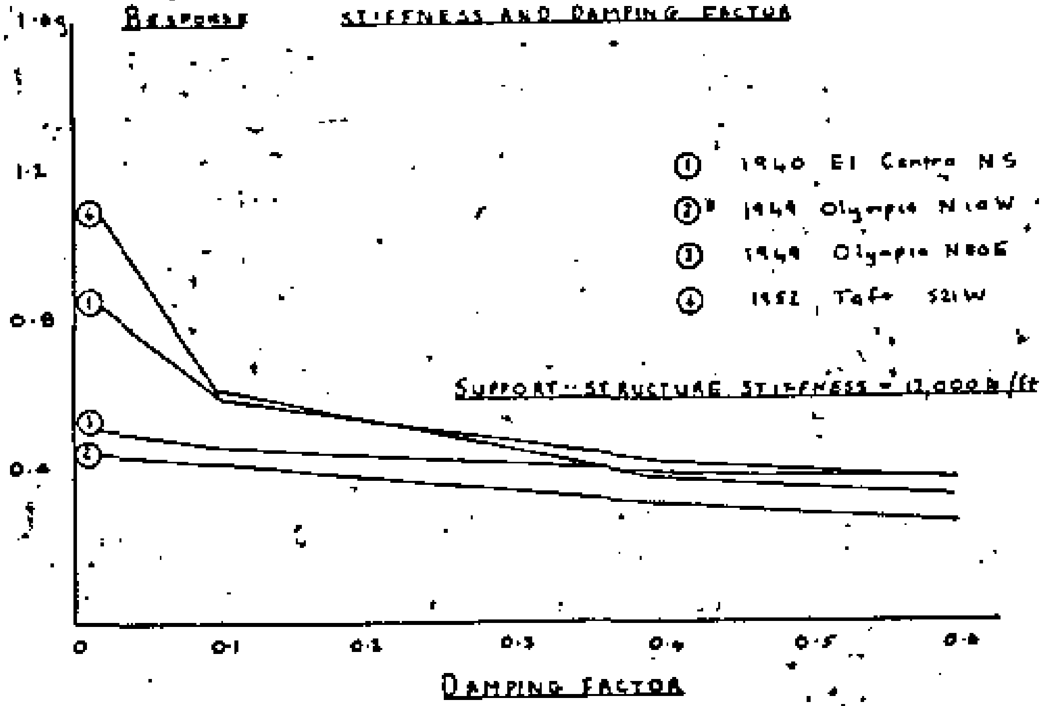
MAXIMUM RESPONSE OF AIRBLAST HEADS FOR VARIATIONS IN SUPPORT STRUCTURE STIFFNESS AND DAMPING FACTOR



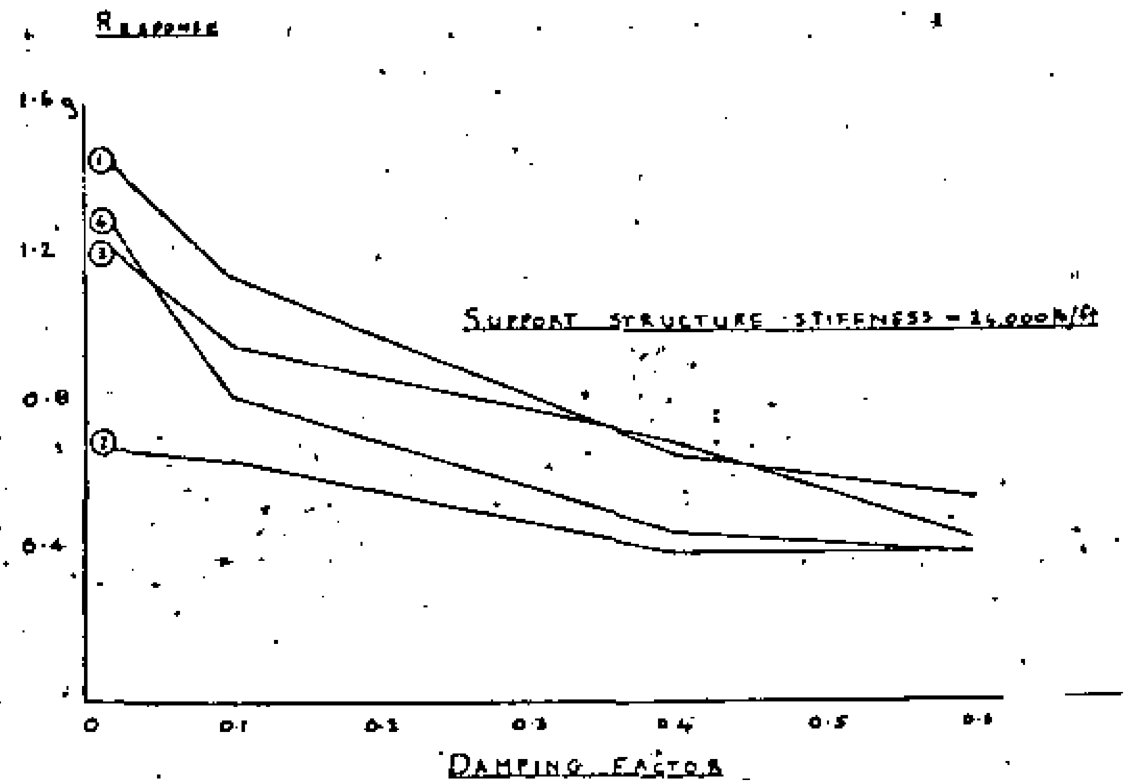


Fig IV

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SEISMIC QUALIFICATION TESTS OF ELECTRIC EQUIPMENT FOR CAORSO NUCLEAR PLANT: COMMENTS ON ADOPTED TEST PROCEDURE AND RESULTS

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ISMES

Seismic qualification tests of electric equipment for caorso nuclear plant: comments on adopted test procedure and results

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SEISMIC QUALIFICATION TESTS OF ELECTRIC EQUIPMENT FOR CAORSO NUCLEAR PLANT: COMMENTS ON ADOPTED TEST PROCEDURE AND RESULTS

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SUMMARY

The seismic tests on Class I electric equipment for the nuclear power plant of ENEL at Caorso have been carried out at ISMES in the last two years. The testing procedure, supplied by the customers, is mainly based on the "IEEE Guide" Std. 344-1971, and is summarized as follows:

— complete panels (fully equipped with operated and non operated devices): resonance search at low acceleration level; determination of amplification and damping at the found resonances; vibration tests at the resonance frequencies (continuous sine) with acceleration to be related to the floor response spectrum (f.r.s.) on the basis of the results of the previous tests;

— devices (operated and monitored): resonance search as above; vibration tests at the resonance frequencies, or at 33 cps if resonances are not found, with increasing acceleration until the device ceases to perform.

Without going into details as to the reliability of the f.r.s. a proper correlation between the results of the resonance search tests and the f.r.s. is easily made could the panel be considered a single degree of freedom, linear and viscously damped system. Actually, almost all the panels show a very complex behaviour with numerous resonances, which, in turn, is highly non-linear. To determine the damping value is then an almost unsolvable problem, and the correlation between the test acceleration and the f.r.s. is very difficult. As regards the responses recorded in points of the panel, high accelerations are due to local resonances of small non-structural parts, so that the choice of the gage points should be done very carefully when the overall behaviour is to be determined. As to the meaning of the "structural integrity" as a criterion of seismic adequacy, the question is debatable. Structural damage occurs very seldom, and affects small parts of scarce importance unless fatigue effects, unlikely to play a noticeable role during an earthquake, are introduced. On the other hand, structural damage may not affect the performance of the devices placed in the panel, whereas their bad performance may occur without structural damage to the panel itself. Moreover, the influence of the mounting (weldings, boltings, connections) greatly affects the dynamic behaviour of the panels, especially when the first resonance frequency is concerned.

On the basis of these considerations, a more reliable testing procedure should be: a continuous frequency sweep at low acceleration (sinusoidal), to record the behaviour of the most significant points; a random excitation whose spectrum matches with the given f.r.s., or—for a more general test procedure—, envelops a number of expected earthquake spectra; to repeat the frequency sweep to make evident possible differences in the panel behaviour due to important structural failures.

Criteria for a correct choice of a specimen panel to be tested are discussed in detail.

As regards the devices, the determination of the resonances frequencies is sometimes difficult (due to the small dimensions of the device its behaviour might be modified by the presence of the pick-up mass), and in most cases, it has no meaning. In effect, a bad performance of the device is mainly due to local resonances (of springs, contacts, connections) which can be discovered only by monitoring.

The tests at 33 cps when resonances are not found have little meaning, because, mainly due to the non-linear behaviour of contacts and other moving parts of the devices (switch on—switch off positions), bad performance may occur suddenly at any frequency for lower acceleration levels.

A better testing technique seems then to be to test the device, its performance being monitored, with continuous sweeps in the frequency range of interest and increasing the acceleration, until it ceases to perform correctly.

1. FOREWORD

Due to the large forces in play, and to the fact it affects the whole structure, a strong earthquake is indeed one of the most important and dangerous accidents which may occur to a nuclear power plant. Of great importance as regards the seismic safety of the plant is the good performance of all the electric and electronic equipment installed. Its continuous performance must be always guaranteed: for the most dangerous condition in order to control and; if necessary, to shut down the plant; during emergency in order to avoid the activity of the plant itself being interrupted. Despite its importance, the seismic verification of this equipment is based on a testing procedure still unsatisfactory.

The problem has two distinct aspects. The first is related to the knowledge of the characteristics of the motion generated by the earthquake at the point of the structure at which the equipment is connected. In normal practice, this information is supplied by the so called "floor responde spectrum" (f. r. s.). The second is to find out a suitable and reliable testing method, whatever the technique used in order to obtain the f. r. s., and however reliable it is.

This paper is an analysis of the problems arisen: a) from the choice of the testing technique adopted for the seismic check, carried out by means of the testing facilities of ISMES, of panels and electrical components to be installed, by Fan Electric and others, at the ENEL IV power plant; b) for the interpretation of the results obtained from the tests.

2. SEISMIC TESTING PROCEDURE ADOPTED FOR ENEL IV NUCLEAR PLANT

The seismic qualification of the electric equipment for ENEL IV nuclear plant has been carried out according to the following two steps:

a) qualification tests of modular panels, monitoring acceleration levels at component installation points;

b) qualification tests of components.

According to the adopted procedure, panels which may be different from the one tested (due to possible slight changes in number and type of the installed components) are considered to be qualified by just testing the new components without requalifying the entire electrical board. Among the different testing methods (sinusoidal, sine beat, multifrequency motion) suggested by some standards - as, for example, the IEEE Guides - a sinusoidal excitation has been chosen to carry out the tests. In particular, the following procedure has been used:

a) as regards the panels, they were tested fully equipped with non operated devices, or dummy loads, and were mounted on a vibrating table in the same way as they will be mounted in the plant. Motion was applied in the three orthogonal directions separately.

A first test consisted of a resonance search carried out with a sinusoidal vibration at low acceleration level (0.05 ÷ 0.10 g) and with variable frequency between 2 and 33 cps approximately. In many points of the structure, selected on the basis of the probable behaviour of the panel and near the mounting points of the components, accelerometers were placed, and the relevant response curves were recorded. When these curves showed a clearly delineated first mode, that is, when the amplification factor had a pattern as shown in fig. 1 a), on their basis a "representative point" of the panel was chosen, in most cases located near the center of gravity of the panel. Besides the value A_{max} of the amplification, the natural period $T_o = 1/f_o$ and damping factor of the structure under test were determined from the amplification curve. These values enabled to obtain, on the f. r. s. curve (fig. 1 b) the value a_s of the acceleration that the representative point has to reach during the qualification test. This test was then carried out at a frequency slightly varying around f_o, and with a base acceleration slowly increasing until reaching, at the representative point of the structure, the value a_s previously determined. During this test the acceleration levels at the component mounting points were monitored in order to compare them with the level each component can withstand without damage. The Pan Electric panels were tested according to this procedure.

When the response curves showed more than one important resonance, the same procedure was applied at each of these frequencies. In turn, when clear resonances could not be found, the maximum acceleration value at floor level was adopted as test input.

b) As regards the components, tests were quite similar and also consisted of two steps. The component was first fixed on the shaking table either directly or by means of a stiff supporting structure. The mounting of the component was carried out in the same way as when in operating conditions on the electric panel. A continuous sweep was performed over the entire frequency range in order to determine, separately for the three principal axes, the resonance frequencies of the component. At the resonance frequencies or, if they are not found, at 33 cps, a vibration test was carried out with input acceleration slowly increasing until the level is reached at which the component no longer operates correctly. During this test the performance of the component under normal operating conditions was monitored.

As regards the quality control tests, a certain percentage of the total amount of a given component has been tested at the same frequencies as before (resonance frequencies or 33 cps) with lower acceleration levels.

3. COMMENTS ON THE TESTING PROCEDURE

In the course of the tests, and depending on the results obtained from time to time, a number of problems arose about the correct application to the single cases of the recom mended test procedure. The three main points under discussion are:

a) the correlation between the adopted test excitation and the given f. r. s.;

b) the criteria used for the analysis of the test results and the consequent assessment as to the seismic adequacy of the panels and components;

c) the generalization of the test results, that is the capability of the "specimen" tested to represent the standard behaviour..

3.1. From a theoretical point of view, the correlation between the f. r. s. (which is a description of a random motion), and the sinusoidal vibration (which excites only a single frequency at a time) is never possible, unless only one vibration mode is considered to be of importance for the dynamic response of the panel. In this case, the correct correlation is given, in resonance conditions, by the simple relationship:

$$a_{r_i} = C \cdot \phi_i \cdot \frac{1}{2\xi} \cdot a_b$$

where:

a_{r_i} is the amplitude of the response at point i of the structure

a_b is the amplitude of the base motion

C is the coefficient of participation of the mode considered

ϕ_i is the "amplitude" of the normalized mode shape at point i

ξ is the percentage of critical damping.

The "representative point" of the structure under test has to be determined through the knowledge of the mode shape and the relevant coefficient of participation, as the point at which $C \cdot \phi = 1$. In fact, at this point, the amplification factor of the panel equals that of a single degree of freedom system. Generally, this point is higher than the center of gravity; it follows that to adopt the center of gravity as the representative point means to increase the base acceleration and the test becomes more conservative. Anyway, as the results of the resonance search supply all the necessary data to evaluate the response, the testing method could be, in this case, fully satisfactory.

However, many tests carried out on a number of different panels show that their actual behaviour does not meet the above stated assumptions, as the panels cannot be considered linear single degree of freedom systems. In fact, a panel is generally a framed structure made of steel bended sheets, connected by bolts or weldings in a vertical arrangement. It supports steel doors and walls, in and on which are mounted many electrical components of various types, such as relays, switches, transformers, etc. It is then a rather complex system of parts with different masses and stiffnesses (figs. 2 - 3) and its dynamic behaviour is usually involved. From the tests, in many cases response curves are obtained which show a large number of resonances even in narrow frequency intervals. Too many tests would then be required, and the use of an impractical and expensive quantity of gauge points, in order to determine the value of the participation coefficient. On the other hand, as to the influence on the components more than the first mode (which is usually related to the overall structural behaviour of the panel) higher modes are of importance, which depend on local vibrations of nonstructural parts (fig. 5).

The dynamic response of the panels is made more involved by their strongly non-linear behaviour, as clearly shown in fig. 6. Increasing input acceleration levels generally cause decreasing values of the peak frequencies and of the amplification, and produce deep modification in the response. The values obtained for resonance frequencies, damping and amplification factors during the frequency sweep at low acceleration are no more the ones in play during the qualification tests, carried out with higher acceleration levels. This is the main reason for the attempts which have been made during the qualification tests, in order to adjust the value a_b of the input acceleration to the value a_s required by the f.r.s., instead of using directly the simple ratio $a_b = a_s/A_{max}$.

3.2. As regards the interpretation of the test results, it is necessary to take into account that the eventual task of the panel is to allow the performance of the components. Its seismic adequacy has then to be judged on the basis of the operative integrity of the devices it must assure. Of course, the integrity of the component depends on the amplification of the panel at the mounting point, which is in turn dependent on possible structural degrading of the panel, due to fatigue effects. This occurred on some occasions during the course of the tests, owing to the uncertainties about the correlation between input acceleration and f.r.s. In fact, in many cases - as mentioned above - the qualification value was reached after a number of tentative tests, and, in other cases, a very high input value was adopted. Despite this, the tests might have not been conservative, as the determined amplification may be less than that occurring on an intact panel.

The structural integrity of the electric panel during and after the seismic test is not a decisive criterion for judging its seismic adequacy. Structural damage may not affect the performance of the devices, whereas their malfunction may occur without any structural fault of the panel. Nevertheless, an essential requirement for the seismic qualification of the panel should be that its overall behaviour before and after the test remains unchanged.

Another important point for the evaluation of the results, is that, even for input motion in a single direction, a point on the panel may have a response along three axes. The choice of the gauge points has then to be made with care, and, at least for the most important devices, should give a complete picture of the response.

As regards the results of component tests, the following considerations can be drawn. The resonance search is sometimes practically difficult, due to the reduced size of the devices under test; when possible, it usually puts in evidence the resonances of the frame or of the box into which the device is placed. However, generally these resonances have small influence, if any, on the malfunction of the component. Its performance depends in fact on the possible vibrations of the springs, contacts, and connections the component contains; the resonances of these parts can be discovered only by means of electrical monitoring. However, should the input excitation give rise to a motion lower than the stroke of the contacts, which have a discontinuous behaviour (switch on - switch off positions), even the electric monitoring may not show resonances which actually are present. Of course, the malfunction search at resonance frequencies or at 33 cps, even at very high acceleration levels, does not ensure that malfunction may not occur suddenly for lower acceleration at any other frequency.

3.3. The necessity of selecting a single specimen to be tested among the numerous panels of a set, caused the problem of the choice of a panel representing the "average" behaviour, taking into account the possible differences. Among these should be mentioned:

- differences due to manufacturing, such as welding, bolting, other connecting operations, whose uniformity should be guaranteed. Weldings having the same external appearance should have also the same strength; bolts or screw connections should be tightened in the same way and have uniform mechanical properties, to ensure the same performance;

- differences in number, type, weight and arrangement of components which panels of the same structure hold. Considerable changes in weight and position may be of noticeable importance, as they may give rise to new vibration modes, such as torsional resonances, which may modify the response amplitude in a way difficult to forecast, unless the weight and arrangement are not very different from those of the tested panel. Similar panels can be considered qualified on the basis of the test results of the specimen only when the amount and distribution of the masses are "more or less" the same even with different components, and new devices are mounted on rigid supports, without modifying the main structures of the panel;

- differences of mounting conditions of the panels. Sometimes many similar panels are joined together to obtain a single electrical board of large size; in other cases a board already tested has to be completed with an additional panel for another installation plant. Then the problem of extrapolating the behaviour of the complete board on the basis of the results of a partial test arises. In particular, the minimum number of panels to be tested together and their arrangement is a debatable question. It is difficult to provide a general answer to these problems, and, owing to lack of regulations, at the present level of knowledge, they should be examined one at a time.

As to the components, the tests are generally made on a specimen which is considered representative of the set of equipment to be tested. Modularity problems arise also in this case: they may sometimes be solved by means of a previous detailed examination of the modifications introduced into the mechanical structure of the component by the variation of some characteristics, as its nominal current and voltage, number of auxiliary contacts, etc. The quality control tests (carried out - as mentioned above - in a statistic way on a percentage of the total amount of the equipment set) may simply require, especially when mass-produced components are concerned, the control of the main characteristics of the device during vibration.

4. CONCLUSIONS

4.1. On the basis of the considerations laid out in chapter 3, which rise from the experience presently available, it is possible to infer that the problem of seismic qualification of electric equipment for nuclear plants has not yet received a satisfactory definition.

Besides the important problems related to the choice of a reliable f.r.s. and of a suitable testing equipment, which are beyond the scope of this paper, it should be underlined that the use of a sinusoidal excitation makes impossible a theoretically correct correlation with the f.r.s. From a practical point of view the problem has been overtaken adopting, in a number of cases, undue input acceleration values.

The use of a single frequency excitation at the resonant frequencies to check seismic adequacy has generally as it consequence the overtesting of the panels with respect both to the amplitude of the input motion, and to the test duration, as tests have to be repeated at the main resonances, or in oder to adjust the input to the required f.r.s. acceleration. Fatigue effects may then arise and sometimes damage the panel.

The use of a sine beat motion of given characteristics, as suggested by some standards, does not overcome these problems. The resonance search has to be made also in this case and qualification tests should be repeated for each found resonance. As regards the correlation with the f.r.s., the same consideration for the sinusoidal input can be made. The slight advantage of a less marked influence of an incorrect damping determination is balanced by the more involved testing equipment necessary to generate the sine beat.

For qualification tests, a more convenient procedure, which is in turn more correct from a theoretical point of view, and supplies at the same time more reliable results should be, in our opinion, the application to the panel of a random excitation whose spectrum matches with the given f.r.s. This method requires the use of a rather sophisticated testing equipment; on the other hand it does not depend on parameters of uncertain, if not impossible, determination and makes immaterial the previous knowledge of the characteristics of the structure. Tests become simpler, as it is enough to measure the acceleration levels reached at component mounting points, thus reducing test duration and avoiding possible overtesting.

The sinusoidal input should be used better as a research step during the design stage of the prototype panel, the correct determination of the structural behaviour of which can in this case be justified.

The problem of finding one or more time histories having a given f.r.s. is still open; from a practical point of view, however, a too detailed matching of the f.r.s. is not strictly necessary. In fact, a completely reliable f.r.s. can never be obtained as the methods used in order to determine it involve generally a number of assumptions regarding a proper definition of the earthquake, a suitable structural schematization of the plant building, an estimate of the values of the parameters in play (soil characteristics, damping etc.). In turn, the problem becomes easier if it is considered that the correlation between time history and f.r.s. should be achieved for a narrow band of damping values, and, eventually, that the equipment manufacturers - for economical reasons - aim to qualify their products for more than a single plant. In this case, time histories should be used whose spectra envelop a number of possible f.r.s., and consequently show smooth shapes.

4.2. As far as the components are concerned, taking into account their different operating conditions, as they are mounted on panels having very different behaviour, the simplest and most reliable procedure seems to be to adopt a sinusoidal sweep over the entire frequency range, disregarding the resonance search and the vibration test at resonance or at 33 cps. As fatigue effects are unlikely to occur, each sweep should be repeated with increased acceleration levels, until malfunction occurs, which has to be shown by means of a continuous electric monitoring.

4.3. The effect of an excitation in three directions simultaneously is at present a problem far from being theoretically solved, due to the complexity and nonlinearity of the panels, even on the basis of the knowledge of the single direction behaviour; from an experimental point of view it should require the use of very sophisticated and expensive testing facilities, which in reality is not justifiable.

4.4. As mentioned above, also the problems which arise as to the representative nature of the specimen under test, can not be completely solved at the present stage of available experience. Whereas a statistic test for the components seems to be suitable in order to assure their quality (different devices may require, however, different percentages of the total amount to be tested), the problem is more involved as regards the panel tests. The standards should establish proper criteria in order to judge when the differences arising from different number and arrangement of the devices, addition or connection of modular panels in the electric boards, modify the behaviour with respect to the specimen panel in such a way as to make necessary supplementary tests.

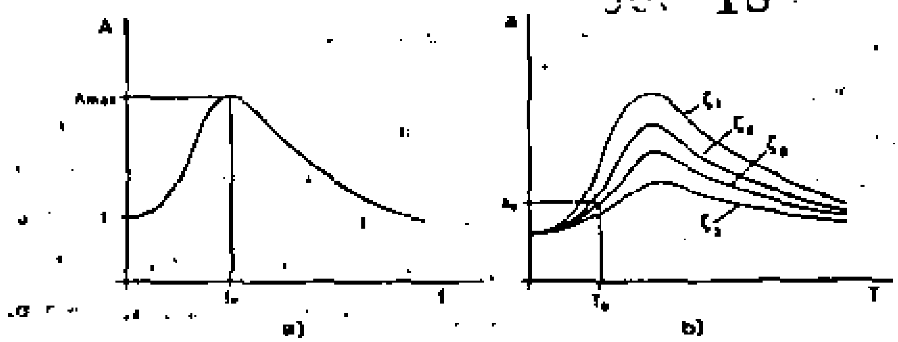

Fig. 1 a) Schematization of the amplification curve.

b) Schematization of the floor response spectrum curves (ζ = percentage of critical damping).

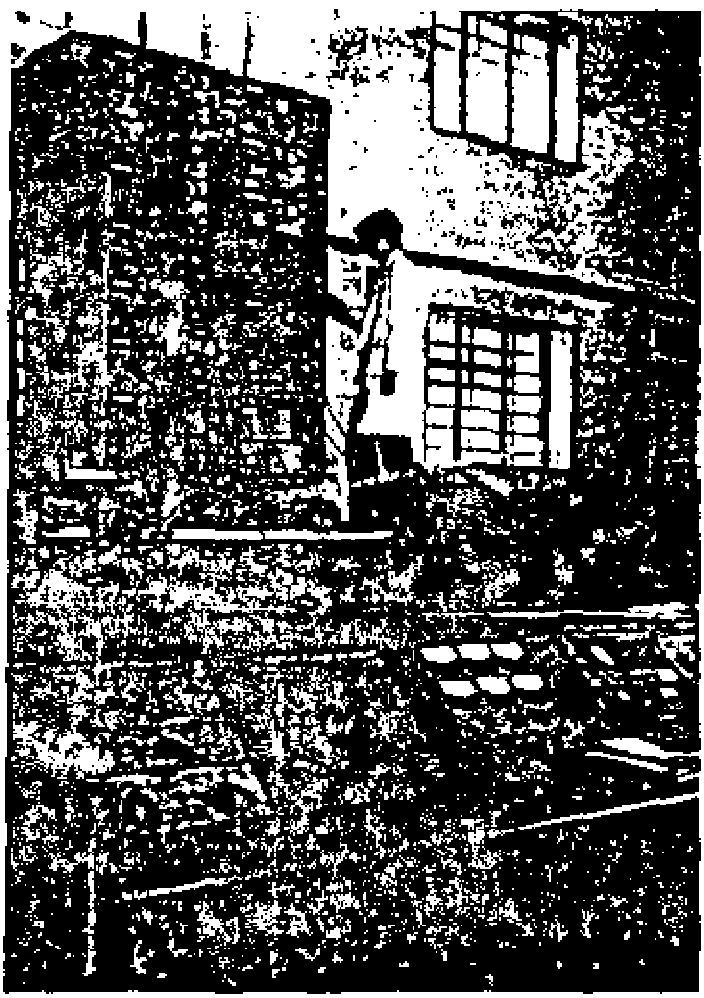

Fig. 2 Pan Electric motor control center, DECABLOC type, during the seismic tests.

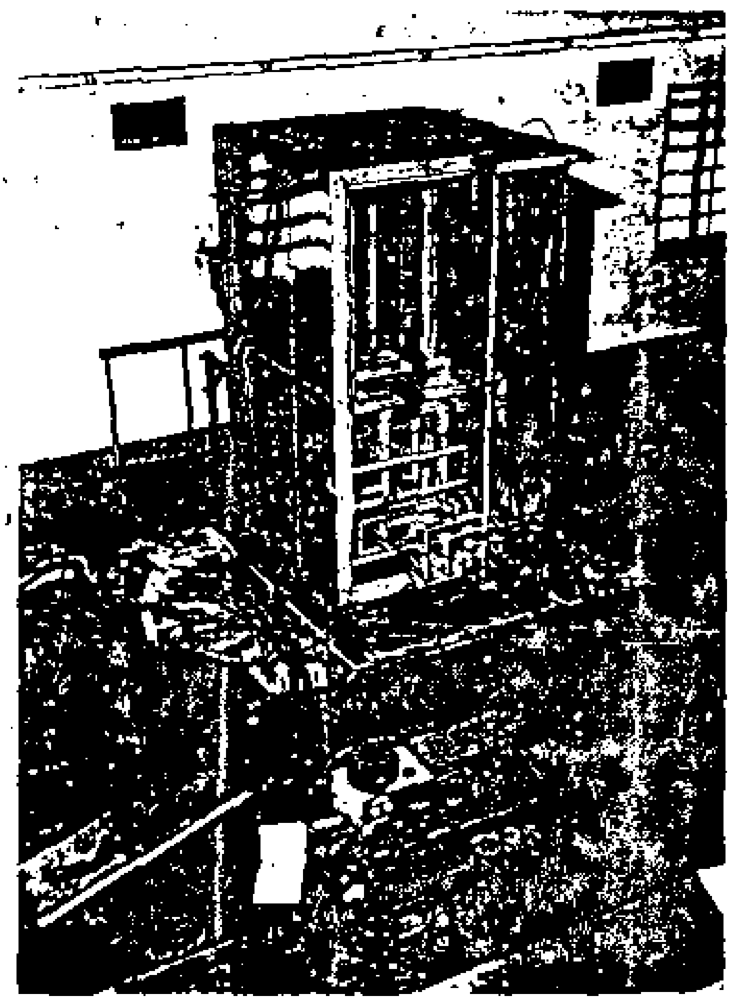

Fig. 3 Pan Electric power center on the ISMES shaking table.

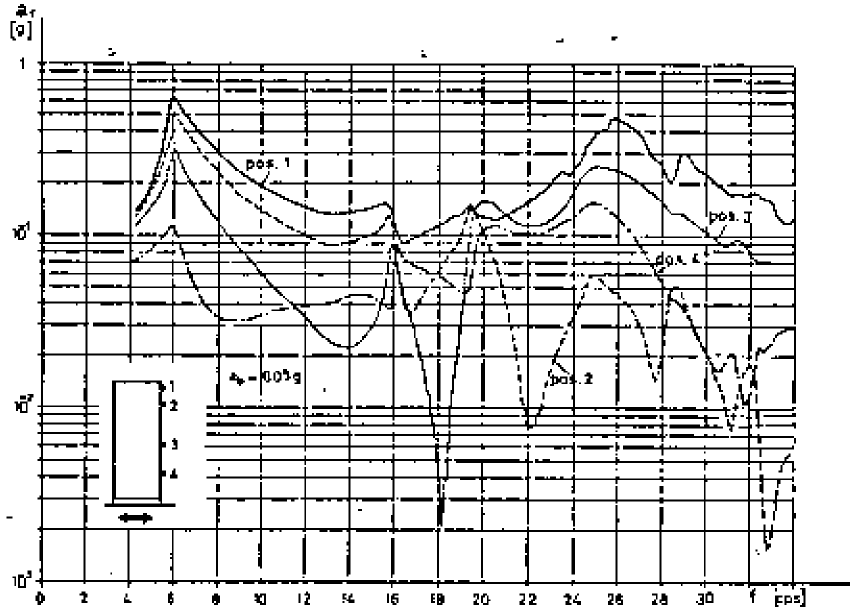

Fig. 4 Response curves recorded in different points of a power center.

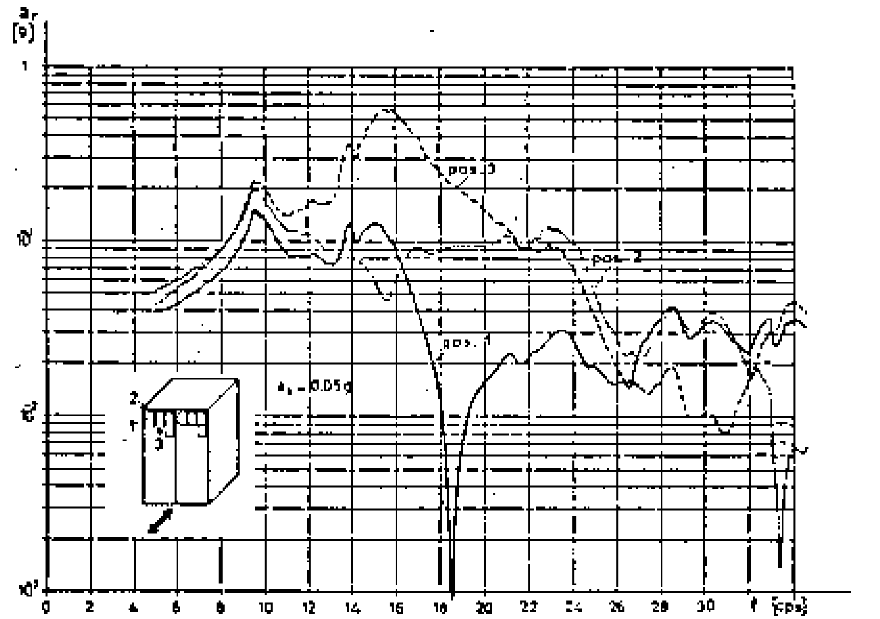

Fig. 5 Response curves of a medium voltage switchgear board, recorded on the structural frame (pos. 1 - 2)

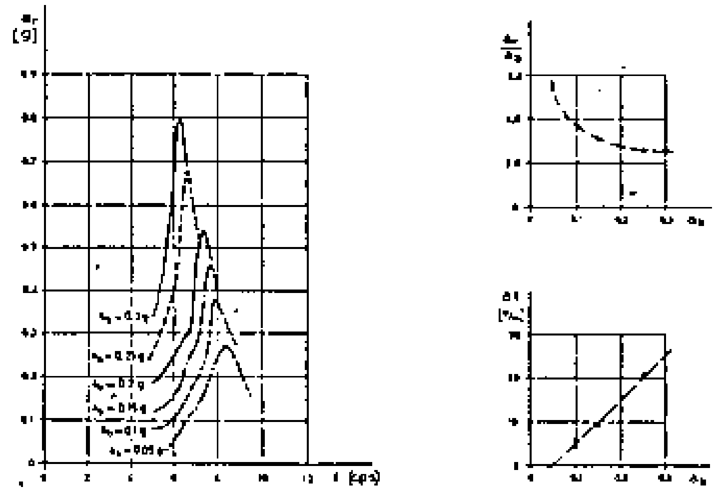

Fig. 6 Non-linear behaviour of an electric panel.

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* Esaurito - Out of print.

LAS ESPECIFICACIONES SISMICAS DE LA ENDESA PARA EL EQUIPO DE ALTA TENSION (1)

En los últimos 25 años, el diseño del equipo eléctrico de alta y muy alta tensión ha ido afirmando una tendencia hacia el empleo de elementos normalizados, de volumen relativamente reducido y que, con características eléctricas funcionalmente bien definidas, puedan ser combinados en aparatos de diferentes voltajes y capacidades. Una tendencia en este sentido puede detectarse, en mayor o menor grado, en el desarrollo de equipos tan diferentes como son los interruptores de potencia, los transformadores de medida, los desconectadores, los pararrayos, etc.; es decir, todos aquellos aparatos que pueden ser englobados bajo el nombre de equipo primario de maniobra y de protección.

Esta tendencia hacia la composición de aparatos a partir de unidades normales se ha unido a la de racionalización de los elementos aisladores, para reducir en lo posible sus dimensiones y disminuír el número de piezas diferentes de porcelana.

Ambas tendencia puede decirse que han provocado una convergencia, en los diseños de equipos de alta y de muy alta tensión, hacia disposiciones estructurales esbeltas y flexibles, con sus elementos dispuestos de manera que resulta, en general, propensa a la formación de respuestas elevadas a las oscilaciones de un temblor.

Los primeros equipos de hasta 154 kV con esta disposición, comenzaron a ser instalados por la ENDESA hacia 1950. Por entonces creíamos suficiente especificar, como condiciones sísmicas para estos equipos, que fueran capaces de resistir "una aceleración sísmica horizontal de 0,3 g, y una vertical de 0,1 g", en que "g" es la aceleración de gravedad, combinadas en la dirección mas desfa-

(1) Versión revisada de un artículo presentado por F. Novoa al Congreso Panamericano de Ingeniería Mecánica, Eléctrica y Ramas Afines, 4a. reunión efectuada en Lima (Perú), del 7 al 13 de noviembre de 1971.

favorable. Es decir, elegíamos simplemente un factor sísmico relativamente severo entre los usuales de los códigos de entonces para la asismicidad de los edificios.

Los equipos, los seleccionábamos entre los que tuvieran una estructura favorable, en relación con las masas soportadas y adoptábamos, en general, disposiciones de instalación que evitaran la transmisión de esfuerzos durante un temblor, a través de las conexiones, introduciendo uniones flexibles o deslizantes, en las de tubos IPS, y dejando una huelga en las de cable.

Disponíamos el equipo con sus partes de alta tensión en un plano horizontal, soportado en estructuras reticulares de perfiles livianos de acero, relativamente rígidas. Intuitivamente, tendíamos a disponer una estructura separada para cada elemento importante, como ser, cada interruptor de 66 kV, cada polo de interruptor de 110 o de 154 kV y cada transformador de medida. Los desconectadores, y algunos pararrayos, en razón de su peso más reducido, los disponíamos en una estructura común para los tres polos.

Hasta 1960, la experiencia acumulada a través de diversos sismos menores había confirmado que, en general, dichas medidas parecían acertadas, pero que, en determinados casos, era necesario considerar el efecto dinámico de las oscilaciones del temblor ya que, típicamente, por ejemplo, los relés sistema Buchholz de los transformadores, operaban incorrectamente debido únicamente a las sacudidas del mercurio de los contactos utilizados en ellos. Para tomar en cuenta estos efectos, y otros que podrían temerse en los mecanismos de operación de los interruptores, por ejemplo, agregamos en nuestras especificaciones la frase : "el equipo deberá operar correctamente bajo la acción de oscilaciones de frecuencia entre 1 y 10 Hz y amplitud no mayor que 10 mm".

Los terremotos de mayo de 1960 en la zona central - sur del país, cuatro de los cuales alcanzaron magnitudes Richter entre 7,5 y 8,5, causaron ciertos daños en nuestras instalaciones, siendo los principales, la destrucción de un banco de condensadores con unidades dispuestas en columnas, y la saltadura de sus vías de dos transformadores, con riesgo evidente de volcamiento. El análisis de éstos y su comparación con desperfectos constatados en instalaciones bien proyectadas, como las de la acerería de Huachipato, nos hizo agregar a nuestras prácticas :

1) Aumento de los factores sísmicos de especificación a 0,5 g en horizontal y a 0,2 g en vertical, para tener en cuenta la disposición del equipo sobre estructuras algo flexibles y poco amortiguadas.

2) Exclusión efectiva de toda transmisión de esfuerzos entre partes sujetas a oscilaciones diferentes, por mejora de las conexiones flexibles y holgadas entre los equipos, y destierro definitivo de las conexiones en tubos IPS.

3) Consideración más cuidadosa de las disposiciones en columna, para evitar las de resistencia horizontal insuficiente.

4) Montaje de los transformadores de potencia sobre bases planas de concreto a ras del suelo, con anclajes dimensionados según 1).

En marzo de 1965, sin embargo, un temblor de magnitud Richter entre 7 y 7,5 en la zona central, produjo daños extensos en las instalaciones existentes en San Pedro, cerca de Quillota, y alcanzó a producir algunos daños en Cerro Navia y otras subestaciones en Santiago, en donde se midió una aceleración máxima horizontal en terreno de fundación, de 0,16 g.

En San Pedro, 38 columnas sobre 48 de los interruptores de aire comprimido de la compañía concesionaria de la distribución, se quebraron en la base. Estos interruptores se encontraban, cada 3 polos y 3 transformadores de corriente, sobre una estructura de entramado liviano, aparentemente rígida, proyectada por el fabricante. De la ENDESA, se derrumbaron dos polos de un interruptor de pequeño volúmen de aceite, de disposición vertical muy elevada, arrastrando consigo dos transformadores de corriente, y se quebraron en la base 3 transformadores de potencial con aisladores muy débiles.

En Cerro Navia y otra subestación de la empresa distribuidora en Santiago, 3 columnas de interruptores como los de San Pedro, en estructuras similares, se quebraron en la base. Sin embargo, 6 columnas idénticas de la ENDESA a no más de 50 metros , y 30 columnas más desfavorables de 154 kV a no más de 150 metros de distancia, no sufrieron daños, estando sus polos correspondientes sobre estructuras separadas.

Concluímos, por lo tanto :

1) Que los factores sísmicos especificados para el equipo destruído habían resultado absolutamente inadecuados para representar el efecto dinámico del temblor.

2) Que, en Santiago, era evidente una influencia del tipo de estructura, que era favorable al sistema empleado por la ENDESA.

Esta, tenía colocada una orden importante por interruptores hasta 220 kV del mismo tipo que los afectados en San Pedro. Requerimos, por lo tanto, de los fabricantes, que investigaran en una mesa vibratoria, cuál era la acción dinámica de la mesa capaz de reproducir las rupturas constatadas, y determinar los medios necesarios para impedirlas, bajo esa misma acción. Mientras tanto, estudiaríamos especificaciones que estábamos dispuestos a discutir con ellos, para determinar el grado final de seguridad que se alcanzaría contra los terremotos reales.

Una discusión más completa de los factores involucrados puede encontrarse en otro lugar (1); baste indicar aquí, que la solución sólo pudo ser encontrada a través de la disminución de la respuesta sísmica de las columnas, mediante el agregado de amortiguadores en la base de cada una de ellas. Como dichos amortiguadores involucraban una considerable elasticidad, debió suprimirse todo elemento de restricción lateral de las columnas, y establecer solamente conexiones flexibles entre ellas.

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(1) CIGRE, Sesión de 1970, informe N° 23-02.

Para representar la acción dinámica de los temblores, adoptamos un "espectro de respuesta" de aceleración. Esta curva representa, con sus cordenadas, la máxima aceleración instántanea que llegará a tomar un oscilador lineal de un grado de libertad, de la frecuencia propia dada por la abscisa, cuando es sometido a una determinada curva de aceleraciones en función del tiempo. Se obtiene, descomponiendo la curva de aceleraciones en una sucesión de impulsos "a_t Δt", cada uno de los cuales, aislado, produciría una oscilación transiente amortiguada del oscilador. La oscilación resultante de éste, será la suma de dichas respuestas transientes que, en términos del cálculo operacional, está dada por la integral de convolución de la función de aceleraciones sobre la función de respuesta del oscilador, cuyo valor máximo dependerá fundamentalmente de la frecuencia propia y de la amortiguación de éste, y puede ser determinado por computador.

Comparando los espectros de respuestas calculados para marzo de 1965 en Santiago, los de otros dos temblores en Chile central y los efectos de los sismos en Huachipato en 1960, se resolvió adoptar la curva empírica :

$$a_{10\,\%} = 0{,}4\ \sqrt{T_o} \leq 0{,}8\ g$$

equivalente, grosso modo, al doble del espectro medio calculado por Housner para amortiguación 10 %, para el sismo de El Centro, en 1940, como representativa para comparar con la resistencia a la ruptura de las columnas.

Se pidió determinar la frecuencia propia y la amortiguación de las columnas mediante una prueba de oscilación libre, y aplicar luego en mesa vibratoria al equipo, la excitación sinusoidal que fuese capaz de producir en ellas la respuesta que les correspondería según la curva. Previamente, se habría verificado en la misma mesa, la frecuencia dinámica de resonancia de las columnas, mediante una prueba de excitación reducida, a frecuencia variable entre 0,5 y 20 Hz.

Los interruptores adicionados de los amortiguadores en la base de las columnas resultaron dotados de un factor de seguridad superior a 2, con respecto a la curva de respuesta mencionada. Sometidos a prueba los desconectadores y los transformadores de medida, se pudo ver que, en los primeros, la respuesta crítica de un buen diseño tendía a ser la de desplazamiento de los contactos, y no la de aceleración horizontal; y en los segundos, la frecuencia y amortiguación relativamente altas de los buenos diseños, no los hacían peligrar, en general.

Mientras se realizaban estudios más completos y se obtenían los resultados de las pruebas, debieron emplearse, para los equipos del sistema El Toro, ordenados entre 1967 y principios de 1969 especificaciones parecidas.

Los estudios fueron demostrando, sin embargo, que la especificación de la respuesta de 1966, adolecía de defectos serios:

1) La respuesta constante supuesta para períodos inferiores a 0,5 s, no permitía coordinación lógica con el hecho evidente de que un elemento suficientemente rígido, tendría una respuesta no mayor que la del suelo.

2) La respuesta límite adoptada para dichos períodos, que abarcaron prácticamente toda la gama de diseños usuales de interruptores, desconectadores y otros, junto con la fórmula de extrapolación para amortiguaciones inferiores a 10% (1), no explicaba satisfactoriamente la ruptura en masa de las columnas en San Pedro.

Estas contradicciones pudimos notar que tendían a desaparacer, si se adoptaba el método propuesto por Newmark y Hall en 1969 (2) para fijar respuestas máximas de diseño.

En dicho procedimiento, se hace uso del resultado del estudio comparativo de muchos espectros de respuesta de terremotos registrados en lugares cercanos al epicentro. Cuando dichos espectros de respuesta son representados en un gráfico logarítmico triaxial, cuyas ordenas de máxima aceleración A, velocidad V y desplazamiento D hayan sido trazadas en función de la frecuencia propia f de manera de que satisfagan la condición del movimiento armónico simple de un oscilador:

$$A/g = 2\pi f \cdot V = 4\pi^2 f^2 \cdot D \quad (g = 980.665\ cm/s^2)$$

puede observarse en dicho gráfico que los espectros de máxima respuesta tienden a quedar representados por una máxima aceleración constante,

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(1) $a_{n\%} = a_{10\%}(10/n)^{1/4}$

(2) "Seismic Design Criteria for Nuclear Reactor Facilities, IV th World conf. Earthq. Eng., Santiago, 1969, B-4, pp 37 - 50.

para las frecuencias elevadas, por una máxima velocidad constante para las frecuencias medias y por un máximo desplazamiento constante o decreciente, para las frecuencias bajas del oscilador.

Estudios como los de Kanai y otros muestran la relación que existe entre las máximas respuestas de aceleración, de velocidad y de desplazamiento en el terreno y la magnitud absoluta (magnitud Richter) y distancia al epicentro del temblor que se considere posible. Es posible, entonces, elegir, para las condiciones de sismicidad que se requieran, un juego de valores de respuestas máximas de aceleración, de velocidad y de desplazamiento en el terreno de fundación, del cual podrán ser obtenidos, mediante factores de multiplicación adecuados, según el método propuesto en el artículo citado, los espectros respuestas máximas para osciladores con diferentes factores de amortiguación.

De acuerdo con nuestra experiencia anterior y considerando las condiciones sísmicas en Chile, hemos llegado a que es aconsejable adoptar, como valores de máxima respuesta en el terreno de fundación, los de 0,5 g para la aceleración, de 60 cm/s para la velocidad y de 46 cm para el desplazamiento. Estos valores han sido considerados por los autores del artículo citado como recomendables en países sísmicos con registros insuficientes de terremotos destructivos, como sería el caso de Chile. Los valores coordinan, además, con los factores sísmicos utilizados hasta ahora por la ENDESA para elementos rígidos, que se seguirán utilizando.

En cuanto a la influencia posible de las estructuras soporte, nuestro estudio procuró abordarla a través del acoplamiento entre el oscilador que representa una columna del equipo y el oscilador que representa la estructura, soportando las masas de la base (CIGRE, ref. cit.). Aplicando los conceptos desarrollados por la teoría de las comunicaciones para el estudio de la propagación de señales aleatorias a través de circuitos oscilantes dados, puede llegarse, como se muestra en un estudio académico del autor, en preparación, a la valoración cuantitativa del efecto que tiene, sobre la respuesta del oscilador superior, la razón entre las frecuencias propias de dicho oscilador solo y del oscilador inferior solo, y la razón entre las masas de ambos.

Para una razón de frecuencias propias en las vecindades de 1, el factor de amplificación que representa para la respuesta del oscilador superior (la columna), su acoplamiento con el oscilador inferior (las masas de la base y la estructura), alcanza valores numéricos principalmente limitados por el valor de la razón entre las masas. Cuando dicha relación es, por ejemplo, de 1/6, el factor de multiplicación de la respuesta del oscilador superior puede alcanzar valores entre 4 y 5, para amortiguaciones como las que se presentan en las columnas o en las estructuras. Se creyó, por lo tanto, que circunstancias como éstas podrían dar la explicación de las rupturas de los interruptores de San Pedro, en 1965, en atención a que la masa del oscilador inferior, representante de la estructura, podría ser elevada, en razón del agrupamiento de 3 polos y de 3 transformadores de corriente sobre la misma estructura.

Estas ideas teóricas encontraron inesperada y dramática verificación en 1971.

Después del sismo de marzo de 1965, los interruptores de aire comprimido en San Pedro fueron mantenidos, por razones económicas, en las condiciones en que se encontraban antes del sismo. Aparte del reemplazo de las columnas destruídas por otras del mismo tipo, no sufrieron modificaciones en su disposición. En Cerro Navia, en cambio, 4 de los interruptores más importantes desde el punto de vista del servicio, fueron desmontados, para reemplazar sus estructuras soporte por otras rígidas, individuales para cada polo y para cada transformador de corriente.

El 8 de Julio de 1971, un nuevo terremoto afectó las provincias centrales del país. La intensidad en Santiago, fué ligeramente inferior a la de marzo de 1965; en San Pedro, fué igual que la de entonces. Los interruptores de San Pedro volvieron a sufrir prácticamente los mismos daños que en 1965 (1) y, en Santiago, en Cerro Navia, dos columnas de los interruptores cuyas estructuras no tuvieron modificaciones, se rompieron.

Nuevamente, sin embargo, no se constataron daños en los interruptores de 110 y de 154 kV de la ENDESA, a corta distancia. La nueva subestación de 220 kV de esta empresa en el mismo lugar, con interruptores de columnas amortiguadas, no sufrió tampoco

(1) De un total de 54 columnas, 46 resultaron quebradas en la base.

el más ligero daño o perturbación. Una subestación más pequeña de la ENDESA en la zona epicéntrica de este temblor, tampoco sufrió daños.

La investigación que se iniciara por este motivo, ha demostrado lo siguiente :

Los 3 polos de los interruptores afectados en San Pedro y Cerro Navia, van apoyados sobre dos vigas que corren a lo ancho de la estructura soporte. Dichas vigas trabajan libremente a la flexión, de modo que cuando se excita la oscilación de un polo lateral, oscilan con la misma amplitud, en dirección contraria al polo del centro, y en la misma dirección, el polo opuesto. Los tres polos forman por lo tanto un oscilador, con las vigas trabajando a flexión como elemento elástico. La frecuencia propia de este oscilador es muy similar a la de oscilación de las columnas sobre la base del polo. Se cumplen, por lo tanto, todas las condiciones para que exista un factor de amplificación del orden de 3,5 de la respuesta de las columnas.

Aceptando que, en Santiago, la máxima aceleración en el suelo, fué del orden de 0,15 g, la respuesta de una columna de estos interruptores, debido a su período propio y a su amortiguación, sería unas 4 veces superior, es decir, de 0,6 g. El factor de amplificación de la respuesta, por el apoyo flexible de los polos sobre las vigas mencionadas, llevaría sin embargo la respuesta total a 3,5 x 0,6 = 2,1 g. Esta aceleración es justamente igual a la mínima de ruptura de las columnas, lo que explicaría las dos columnas rotas en Cerro Navia en esta ocasión.

En San Pedro, bastaría suponer una aceleración máxima del suelo de 0,25 g, para llegar a 3,5 g de respuesta total en las columnas, suficiente para explicar la ruptura en masa de ellas.

Este factor de amplificación por acoplamiento entre osciladores, resulta también de fundamental importancia para verificar el equipo solo. Esto es lo que se hace, cuando está asegurado que toda oscilación de la estructura de soporte se efectúe con frecuencia propia de 15 Hz o más, ya que entonces, la respuesta máxima de tal oscilador no podrá ser mayor que la respuesta en el terreno. Pero, aún en este caso, si se presenta una base de un grupo de columnas que no pueda ser considerada rígida en comparación con ellas, o un elemento flexible liviano con su base fija sobre otro elemento flexible o, aún una parte superior de columna considerablemente más liviana que una inferior y flexible, pueden presentarse, dentro del equipo mismo, condiciones suficientes para la aparición de un factor de amplificación considerable en las respuestas.

Es necesario concluír, por lo tanto, que los factores de verdadera importancia para la verificación de la asismicidad del equipo eléctrico de alta tensión serían :

1) Amplificación de las solicitaciones sísmicas por la respuesta de un elemento flexible que quede dentro del rango de frecuencias típicas de este equipo.

2) Amplificación de las respuestas sísmicas por acoplamiento mutuo entre elementos flexibles de frecuencias propias similares combinados estructuralmente.

3) Transmisión indebida de esfuerzos entre elementos flexibles separados, por exceso de desplazamiento relativo entre los extremos de uniones eventuales entre ellos.

Las especificaciones sísmicas que hemos preparado, junto con tomar en cuenta nuestra experiencia descrita más arriba, dan la debida prominencia a dichos tres factores, en un procedimiento aproximado de verificación. El margen de duda y de variación que permite dicha aproximación, hacen necesario considerar, en la mayoría de los casos, pruebas tipo de vibración del equipo. Las pruebas exigibles, así como sus condiciones mínimas de ejecución han sido también consideradas en estas especificaciones.

ESPECIFICACIONES SISMICAS PARA EL EQUIPO ELECTRICO DE ALTA TENSION (≤220 kV)

0.- ALCANCE.

0.1.- Esta especificación se aplica al equipo eléctrico de tensión igual o inferior a 220 kV, en especial, al de disposición estructural relativamente esbelta, en columnas aislantes, como interruptores, transformadores de medida, desconectadores y todos aquellos equipos o partes de alta tensión cuyas condiciones estructurales puedan considerarse asimilables a éstas.

0.2.- Otras especificaciones sísmicas que, en la presente, se consideran específicamente excluídas, son :

a) Especificaciones sísmicas para el equipo eléctrico de muy alta tensión (mayor que 220 kV) ;

b) Especificaciones sísmicas para transformadores de poder;

c) Especificaciones sísmicas para instalaciones eléctricas primarias en Centrales y Subestaciones ;

d) Especificaciones sísmicas para instalaciones eléctricas auxiliares y de control, en Centrales y Subestaciones.

1.- GENERAL.

1.1.- El equipo eléctrico de alta tensión deberá estar diseñado de manera que sea capaz de resistir, sin daños ni perturbaciones de servicio, los efectos de movimientos sísmicos de las siguientes amplitudes máximas de oscilación horizontal, correspondientes al terreno de fundación (1) :

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(1) Se supone un terreno firme de fundación. Si las condiciones de la obra obligan a fundar en terreno suelto o de relleno, estas características deberán ser revisadas.

Aceleración, menor o igual que 0,5 g.

Velocidad, menor o igual que 60 cm/s.

Desplazamiento, menor o igual que 46 cm.

en que "g" designa la aceleración de gravedad (980 cm/s^2).

En los casos en que se señale expresamente, se considerará también un movimiento oscilatorio vertical de aceleración máxima 0,2 g.

1.2.- Se aceptará que las respuestas horizontales máximas más desfavorables en cuanto a aceleración, velocidad o desplazamiento, de un oscilador simple sometido a movimientos sísmicos de duración media y con las características máximas indicadas en el artículo 1.1, quedan dadas por los espectros de respuesta de la fig. 1, en función de la frecuencia propia y de la amortiguación de dicho oscilador.

2.- DISPOSICION GENERAL Y MONTAJE DEL EQUIPO.

2.1.- El equipo puede ser clasificado en dos tipos de disposición estructural :

Tipo I, los equipos que, por tener sus partes principales de alta tensión encerradas en una envoltura metálica al potencial de tierra, presentan una disposición estructural cuya rigidez y características elásticas no están generalmente limitadas por razones de diseño.

Tipo II, los equipos que, por tener sus partes principales de alta tensión soportadas en aisladores, o por consistir esencialmente su estructura en un aislador, tienen una disposición estructural básicamente limitada por razones de diseño.

2.2.- Los equipos del tipo I deberán estar construídos de manera que, al ser fijados sobre una base rígida con los medios previstos al efecto en su diseño, y bajo una fuerza horizontal igual al 50 % de su peso, aplicada en su centro de gravedad, el desplazamiento de éste con respecto a la base, por efecto de las deformaciones elásticas en la estructura, no sea superior a 0,5 mm.

2.3.- Los equipos del tipo II deberán estar, preferentemente, construídos de manera que sus elementos de alta tensión consistan en simples columnas, estructuralmente aisladas y soportadas solamente en la base, con el objeto de facilitar su oscilación libre durante el temblor.

2.4.- El equipo estará, en general, destinado a ser montado apoyado sobre estructuras relativamente rígidas, que aseguren una frecuencia propia de oscilación, del centro de gravedad del equipo o de cada grupo de columnas soportadas en una base común, igual o mayor que 15 Hz.

2.5.- Los equipos del tipo II que consten de una sola columna, sobre una base de dimensiones pequeñas en relación con las de aquella, deberán poder ser montados sobre estructuras de rigidez inferior a la indicada en el artículo anterior, capaces solamente de asegurar una frecuencia entre 7,5 y 12 Hz en el centro de gravedad del equipo.

3.- ESTRUCTURAS DE SOPORTE.

3.1.- Cuando el diseño del equipo incluya estructuras de soporte destinadas a ser ancladas directamente a la fundación, o cuando la orden de compra incluya las estructuras de soporte, se interpretará que éstas cumplen con la condición del artículo 2.4, si se verifica que :

El desplazamiento horizontal del centro de gravedad del equipo, por efecto de desplazamiento horizontal de la estructura en los puntos de fijación del equipo y por efecto de la inclinación del plano de sujección del equipo con respecto a la horizontal, cuando actúan sobre el equipo y la estructura fuerzas horizontales iguales al 50 % de los pesos en los centros respectivos de gravedad, sea igual o menor que 0,5 mm.

3.2.- La verificación indicada en el artículo 3.1 se hará tomando en cuenta solamente las deformaciones elásticas de la estructura soporte, sin incluir desplazamientos de pernos en los nudos o sentamientos en la fundación. Naturalmente, la ejecución de los nudos deberá ser adecuada para prácticamente eliminar la posibilidad de dichos desplazamientos, durante el período de formación de la respuesta máxima de la estructura al movimiento oscilatorio del temblor.

3.3.- En el caso de que una misma estructura sirva para soportar varios grupos de columnas, cada grupo en una base separada, la verificación indicada en el artículo 3.1 deberá considerar la fuerza horizontal actuando en cada grupo, en el sentido más desfavorable al desplazamiento del centro de gravedad de cualquiera de ellos.

3.4.- La resistencia de las estructuras soporte que cumplan con el artículo 3.1 deberá ser verificada agregando a las cargas usuales en este tipo de estructuras, un esfuerzo sísmico horizontal igual al 50 % de los pesos de las estructuras y del equipo, actuando en los respectivos centros de gravedad, y en la dirección más desfavorable. La resistencia de los puntos de anclaje a la fundación deberá verificarse agregando, además un esfuerzo sísmico vertical igual al 20 % de los pesos, en la dirección más desfavorable al volcamiento de la estructura.

4.- VERIFICACION DEL EQUIPO.

4.1.- El equipo del tipo I que cumpla con el artículo 2.2, deberá verificarse como sigue :

4.1.1.- Todo elemento o grupo de elementos, cuya frecuencia propia de oscilación, determinada según el capítulo 7, sea igual o superior a 15 Hz, deberá resistir una aceleración horizontal igual a 0,5 g y una vertical igual a 0,2 g combinadas en la dirección más desfavorable, sin daños para el elemento o grupo de elementos ni perturbaciones para el servicio.

4.1.2.- Si algún elemento o grupo de elementos tiene una frecuencia propia de oscilación inferior a 15 Hz, dicho elemento o grupo de elementos deberá ser sometido a las mismas verificaciones que se especifican para elementos del equipo del tipo II, según las disposiciones del artículo 4.2.

4.2.- Los equipos del tipo II que cumplan con las condiciones de los artículos 2.3 y 2.4, serán verificados como sigue :

4.2.1.- Cada elemento, o porción de elemento, cuya columna sea de sección transversal uniforme o asimilable a ella, podrá ser considerado como un oscilador simple, con la masa total que soporta concentrada en su centro de gravedad, y con la frecuencia propia de oscilación y el factor de amortiguación que se deducen de su prueba según el capítulo 7.

4.2.2.- Para cada elemento definido como en 4.2.1, supuesto soportando el conjunto más desfavorable de masas a que está destinado, deberá verificarse :

a) que resiste, en las condiciones que se fijan en 4.4, una aceleración horizontal en la dirección más desfavorable, de valor igual al que se deduce para la frecuencia propia del elemento, en la curva de respuesta que corresponde a su factor de amortiguación, en el gráfico de fig. 1;

b) que es capaz de tomar, en la dirección más desfavorable, sin que ninguna de sus partes sobrepase las condiciones 4.4 ni altere las condiciones de trabajo previstas en el diseño, un movimiento oscilatorio de la amplitud que se deduce, para su centro de gravedad, según su frecuencia propia, en la curva mencionada.

c) que no dará lugar a perturbaciones en el servicio cuando sus respuestas de aceleración y de desplazamiento sean las determinadas según a) y b).

4.2.3.- Toda unión entre elementos diferentes del equipo deberá permitir, sin que aparezca un esfuerzo apreciable de reacción sobre ellos, un desplazamiento relativo entre sus extremos, igual a la combinación más desfavorable de movimientos determinados según 4.2.2.b) para los respectivos elementos. Si un elemento puede asimilarse a una columna con masa repartida uniformemente, deberá tomarse en cuenta, para este objeto, que el desplazamiento de su parte superior puede alcanzar a 3 veces el valor del desplazamiento en el centro de gravedad.

4.2.4.- Cuando un elemento de frecuencia propia de oscilación inferior a 15 Hz, tenga su base de oscilación fija sobre otro elemento de frecuencia propia inferior a 15 Hz, el elemento soportado deberá verificarse como en 4.2.2 y 4.2.3, pero con sus respuestas de aceleración y de desplazamiento amplificadas por el siguiente factor :

$$k = (m_2/m_1)^{1,8(f_1/f_2 - 0,5)} \qquad \text{para} \quad 0,5 \leq f_1/f_2 \leq 1$$

$$k = (m_2/m_1)^{1,2(f_2/f_1 - 0,25)} \qquad \text{para} \quad 1 \leq f_1/f_2 \leq 4$$

siendo "k" un factor de multiplicación, "m_1" la masa oscilante del elemento soportado y "m_2" la masa oscilante que corresponde al elemento soportante, "f_1" la frecuencia propia del elemento soportado y "f_2" la frecuencia propia que corresponde al elemento soportante., Cuando este último está representado por la base del elemento soportado, sujeta a una oscilación angular de su plano de fijación, la razón de masas deberá reemplazarse por la razón de momentos de inercia en torno al eje respectivo.

Es importante, en la determinación del factor de multiplicación "k", que en el valor de la masa "m_2" se considere el efecto de todas las masas (o momentos de inercia) que contribuyan simultáneamente al modo de oscilación de que forma parte el elemento soportante.

Para $f_1/f_2 < 0,5$ o para $f_1/f_2 > 4$, el factor de multiplicación podrá considerarse igual a 1, cualquiera que sea el valor de la razón "m_2/m_1".

4.3.- Los equipos del tipo II que se encuentran en las condiciones del artículo 2.5, deberán ser verificados como en el artículo 4.2, tomando en cuenta que la estructura soporte representa un elemento oscilante, que introducirá un factor de amplificación de las respuestas, como se indica en 4.2.4. Se supondrá como mínimo, en estos casos, un factor k = 1,5.

4.4.- La resistencia de los elementos de los equipos considerados en los artículos 4.1 a 4.3 será considerada aceptable, cuando las tensiones elásticas máximas que resulten en cada una de las piezas, para las condiciones más desfavorables de las aceleraciones de(*)verificación, sean inferiores a la carga unitaria mínima de ruptura dinámica de los materiales frágiles o al límite inferior de fluencia de los materiales dúctiles que las componen, para el tipo de trabajo que corresponda a la respuesta, tenida consideración de todas las concentraciones de forma que pueden provenir de la disposición final de montaje de la pieza. El trabajo de las piezas, en las condiciones de carga señaladas, no podrá implicar la alteración de ninguna de las condiciones adecuadas para el servicio en otras piezas.

4.5.- Las verificaciones detalladas previstas en los artículos 4.1 a 4.3 podrían ser dispensadas en aquellos elementos cuya resistencia mínima al tipo de trabajo proveniente de la respuesta

(*).....verificación sumadas [illegible] de servicio, [illegible].....

el temblor, en las condiciones prescritas en 4.4, sea igual o superior a 4 g, verificada según el capítulo 6.

4.6.- Los equipos del tipo II cuya disposición estructural no cumpla con las disposiciones del artículo 2.3, por presentar restricciones hiperestáticas en la estructura de sus elementos, no podrán ser sometidos a verificación bajo los métodos aproximados del presente capítulo. Sólo podrán ser verificados representando su estructura por un número suficiente de masas concentradas para poder deducir sus diferentes modos de oscilación, sometiéndolos a prueba como se indica en el artículo 7.14 para determinar las características oscilatorias de dichos modos, y calculando luego las solicitaciones más desfavorables de sus elementos, por la combinación de las respuestas de los diferentes modos, calculada según los espectros de la fig. 1. Dichas solicitaciones más desfavorables deberán cumplir con el artículo 4.4.

5.- <u>ADAPTACION DEL EQUIPO A LAS CONDICIONES SISMICAS</u>.

5.1.- En el caso de que algún elemento del equipo no satisfaga las condiciones que se indican en el capítulo 4, podrán aceptarse modificaciones en el diseño bajo las siguientes condiciones :

a) Antes de recurrirse al aumento de la resistencia de las piezas afectadas, deberán haberse agotado las posibilidades de reducir cualquier factor eventual de amplificación de la respuesta del elemento por motivos como los señalados en 4.2.4, y las de reducir las respuestas propias del elemento a través del aumento de su factor de amortiguación o de la modificación de su frecuencia propia de oscilación.

b) Todo efecto de amortiguación deberá introducirse, en lo posible, en el lugar y del modo en que incida más directamente en la reducción de la respuesta del elemento afectado.

c) La modificación deberá hacerse de manera que no se produzca una pérdida apreciable de ninguna de las cualidades de servicio o de mantención ya logradas para el equipo en su diseño normal.

d) La modificación, tanto en su diseño como en sus materiales, deberá estar sancionada por una clara experiencia anterior del fabricante en dispositivos similares, sometidos a condiciones de servicio comparables con las que deberá satisfacer en el equipo modificado.

5.2.- En igualdad de condiciones, se dará preferencia :

a) Al equipo que con su diseño normal, satisfaga las condiciones sísmicas de las presentes especificaciones, frente al equipo que requiere de modificaciones.

b) Al equipo cuyas modificaciones consisten sólo en el reemplazo de partes del diseño normal, por partes adecuadas de diseños normales para otras condiciones de servicio.

c) Al equipo que presente un factor de seguridad más adecuado en el cumplimiento de las presentes especificaciones.

d) Al equipo cuyas características de respuesta puedan adaptarse más favorablemente a posibles variaciones individuales en el acelerograma de un determinado movimiento sísmico, y a posibles variantes en la combinación más desfavorable de solicitaciones dentro de los máximos señalados en el artículo 1.1.

5.3.- Sólo podrán aceptarse disposiciones que alteren el montaje del equipo, con respecto a lo especificado en el artículo 2.4, cuando se demuestre :

a) que la solución recomendada satisface las condiciones del artículo 5.1 y represente ventajas en el sentido del artículo 5.2;

b) que se pueda realizar una prueba efectiva para las condiciones de respuesta más desfavorables a movimientos sísmicos de las características señaladas en el capítulo 1;

c) que existe una ventaja económica capaz de compensar los mayores costos que pueda representar el nuevo sistema de montaje;

d) que el sistema de montaje no interfiere con la disposición general del resto del equipo en la subestación.

6.- VERIFICACIONES Y PRUEBAS EXIGIBLES.

6.1.- El fabricante de cualquier equipo de alta tensión deberá proporcionar, junto con la propuesta, la frecuencia de oscilación de cualquier elemento o grupo de elementos susceptible de ser excitado en vibración por las oscilaciones de un temblor.

6.2.- Para aquellos elementos o grupos de elementos cuya frecuencia propia quede por debajo de 15 Hz, deberá proporcionarse además el factor de amortiguación de las oscilaciones, expresado en por ciento de la amortiguación crítica.

6.3.- En el equipo del tipo I, vale decir, interruptores, transformadores de medida o especiales en estanque a tierra y con aisladores de paso, equipos blindados con carcasa a tierra y similares, la estructura del equipo y sus soportes deberán ser verificados de acuerdo con lo dispuesto en el artículo 2.2 y el capítulo 3. Todo elemento o grupo de elementos susceptible de ser excitado en vibración deberá ser sometido a las verificaciones que correspondan según el capítulo 4, previa determinación de su frecuencia propia de oscilación y de su factor de amortiguación en conformidad con los artículos 7.1 a 7.3.

6.4.- Si en un equipo del tipo I pueden existir dudas respecto de la capacidad efectiva, del conjunto o de cualquiera de sus elementos, para cumplir sin reparo las condiciones del capítulo 4, dicho equipo deberá ser sometido a una prueba de respuesta máxima en mesa vibratoria, como se indica en el capítulo 7. Si la duda se restringe a la resistencia estructural de un elemento o grupo de elementos dados, la prueba podrá restringirse a una de resistencia mínima a la ruptura, según el artículo 7.4.

6.5.- El equipo del tipo II, es decir, interruptores de aire comprimido o pequeño volumen de aceite, con sus cámaras de ruptura sobre columnas aislantes, desconectadores, transformadores de medida de columna, pararrayos y otros elementos de alta tensión similares, en columnas flexibles aislantes, deberá cumplir con los artículos 2.3 a 2.5 y el capítulo 3 cuando corresponda, y cada uno de sus elementos deberá ser verificado de acuerdo con el capítulo 4. Para esto, deberán determinarse la frecuencia propia y la amortiguación de ellos, de acuerdo con los artículos 7.1 a 7.3.

6.6.- El equipo del tipo II que consista en grupos de columnas con relación funcional entre ellas, o que contenga en sus columnas otros elementos susceptibles por sus respuestas de aceleración o de desplazamiento, como son los interruptores de potencia, desconectadores, algunos transformadores de medida, pararrayos y otros, deberá ser sometido a pruebas tipo de aceptación en mesa vibratoria, como se indica en 7.5 y siguientes. Estas pruebas se harán además de las verificaciones 6.5, que necesariamente deberán incluir la de todo elemento susceptible interior. Como quiera que alguno de dichos elementos resulte crítico en el sentido de las condiciones del capítulo 4, deberá ser expresamente tomado en cuenta durante la prueba, considerando, en caso necesario, las disposiciones del artículo 7.12.

6.7.- Los equipos o elementos del tipo II que consistan en simples columnas y que no presenten partes que resulten tan críticas en sus respuestas como la misma columna, como es el caso de diversos transformadores de medida y pararrayos, y de los aisladores soportes y de paso, terminales para cable y similares en general, en lugar de la prueba especificada en 6.6, podrán ser sometidos a una prueba de mínima resistencia dinámica horizontal, como se indica en 7.4, para la respuesta de aceleración que les corresponda según el capítulo 4.

La determinación de la respuesta, con determinación previa de la frecuencia propia y del factor de amortiguación, podrá además ser evitada en aquellos elementos de disposición estructural homogénea, como aisladores soportes y de paso, terminales para cable, etc., siempre que el elemento sea capaz de resistir en esa prueba, según 7.4, una respuesta de valor igual o superior a 4 g.

7.- CONDICIONES MINIMAS PARA LAS PRUEBAS.

7.1.- Para la determinación de la frecuencia propia de los elementos, según se pide en los artículos 6.3 y 6.5, el equipo, en condiciones reales de servicio, será fijado, por los medios previstos al efecto en su diseño, sobre una base rígida. Sobre el centro de gravedad de cada uno de los elementos cuya frecuencia se requiere determinar se aplicará una tracción, en la dirección de la máxima amplitud de las oscilaciones, de valor no inferior a 1/3 del peso del elemento oscilante, y se registrarán las oscilaciones que el elemento efectúe por segundo, cuando se interrumpe bruscamente la tracción aplicada.

7.2.- Para la determinación del factor de amortiguación, se aplicará el mismo procedimiento que en el artículo anterior, pero en este caso, el registro de las oscilaciones deberá realizarse por medios que proporcionen sensibilidad y precisión suficientes para determinar el decremento de las oscilaciones en función del tiempo transcurrido desde la interrupción de la tracción. El factor de amortiguación equivalente se determinará de acuerdo con el gráfico de la figura 2, a través de la sucesión de máximos de las ondas en la zona del registro en que el decremento aparece con claridad y precisión suficientes.

7.3.- Cuando el equipo contenga diversos elementos susceptibles de vibración, las pruebas de los artículos 7.1 y 7.2 se efectuarán aplicando tracciones en los centros de gravedad de las diversas masas sujetas a oscilación, y registrando simultáneamente las oscilaciones de los puntos correspondientes a las mayores amplitudes, para tratar de detectar todos los modos de oscilación de la disposición. En estos casos, es posible que el decremento de las oscilaciones de un elemento se vea interferido por batimientos con las oscilaciones de otro elemento de frecuencia parecida, en cuyo caso deberá procederse como se indica en la fig. 2.

7.4.- La prueba de resistencia dinámica mínima a la ruptura horizontal prevista en el artículo 6.7, será efectuada en forma similar a la del artículo 7.1 pero aplicando una tracción igual a unas ~~dos~~ 1,3 veces la respuesta de aceleración que corresponde al elemento según el capítulo 4. La prueba deberá efectuarse sucesivamente a cada elemento definido según el artículo 4.2.1, y será necesario registrar las aceleraciones instantáneas en el centro de gravedad del elemento, en función del tiempo transcurrido desde que se interrumpe bruscamente la tracción. El valor cuadrático medio de las dos primeras semiondas, deberá alcanzar para cada elemento, un valor igual o mayor que $a/\sqrt{2}$ (siendo "a" la respuesta de aceleración). El equipo sometido a la prueba no deberá presentar, después de ésta, ningún daño, deformación o filtración y deberá estar plenamente apto para resistir cualquiera de las pruebas especificadas de recepción.

7.5.- Las pruebas en mesa vibratoria que se prescriben en el artículo 6.6 deberán ser realizadas en un laboratorio autorizado que cuente con el equipo y la experiencia necesarios para la prueba.

Las pruebas consistirán en la aplicación, en régimen forzado, de oscilaciones sinusoidales horizontales, a la base del equipo en condiciones reales de servicio para :

a) Verificar las frecuencias de resonancia de sus distintos modos o elementos (artículos 7.9 y 7.10) ;

b) ~~Reproducir en los distintos elementos las respuestas máximas que les corresponden según las especificaciones del capítulo 4, para comprobar si se cumplen las principales condiciones establecidas para ellas (artículo 7.11).~~

Se reemplaza artículo 7.5 b. Ver página 1[illegible]

7.6.- La prueba deberá efectuarse sobre un conjunto completo del equipo en condiciones de servicio. Este conjunto deberá comprender todos aquellos elementos montados en columnas sobre una base común, la que deberá ser fijada por los medios previstos en su diseño, y teniendo especial cuidado de no alterar sus condiciones naturales de rigidez, a una mesa vibratoria de capacidad suficiente para la masa y dimensiones del conjunto, según las disposiciones del artículo 7.13.

7.7.- En el caso de conjuntos con diferentes planos verticales de simetría, la prueba deberá efectuarse aplicando oscilaciones en la dirección de cada uno de estos planos. Si el conjunto puede alterar sus condiciones estructurales cuando se encuentra en distintas condiciones de servicio, como sería el caso de un polo de desconectador en posiciones "abierto" o "cerrado", la prueba deberá efectuarse en cada una de dichas condiciones de servicio.

7.8.- Durante la prueba deberán registrarse las variaciones instantáneas en función del tiempo de los siguientes valores cuando menos :

a) respuesta de aceleración horizontal en el centro de gravedad de cada uno de los elementos sujetos a verificación;

b) tensiones elásticas máximas en no menos de dos puntos de la pieza más solicitada por la respuesta de cada elemento;

c) desplazamiento relativo entre aquellas piezas cuya respuesta de desplazamiento pueda ser de importancia para el resultado de la prueba (contactos de un desconectador por ej.);

d) desplazamientos de la base del conjunto con respecto a la mesa vibratoria, en un número suficiente de puntos como para caracterizar los modos de oscilación propios de dicha base;

e) aceleración o desplazamiento de la mesa vibratoria.

El registro de las respuestas de aceleración indicado en a) deberá indicar valores medios cuadráticos. Las lecturas de los detectores de tensiones del punto b) deberán ser calibradas aplicando en cada elemento una fuerza horizontal variable entre 0 y 50 % de su peso oscilante, para registrar las lecturas correspondientes.

7.9.- Para una amplitud constante de oscilación de la mesa, tal que la respuesta de ninguno de los elementos sobrepase un 80 % de la máxima respuesta sísmica que le corresponde según el capítulo 4, se efectuará una prueba de frecuencia variable, haciendo variar la frecuencia por escalones, y manteniéndola constante en cada escalón durante un tiempo suficiente para que se establezca la máxima respuesta de los elementos a esa frecuencia de excitación. La prueba se repetirá para distintas amplitudes de la mesa, hasta lograr registros de respuestas máximas en los distintos elementos del equipo con lecturas no inferiores a 10 veces la mínima sensibilidad de la medida, para frecuencias de excitación entre 0,5 y 20 Hz.

7.10.- Los resultados de la prueba 7.9 serán representados en curvas de respuesta de cada elemento en función de la frecuencia de excitación, para diferentes amplitudes constantes de excitación. En estas curvas aparecerán máximos correspondientes a diferentes resonancias secundarias, que podrán ser identificados por interpretación de los registros en 7.8. En el caso de efectos no lineales o semiplásticos de amortiguación, la frecuencia de resonancia de cada elemento variará apreciablemente en función de la amplitud de excitación; por lo que deberá ser estimada por extrapolación de dicha variación.

7.11.- A cada una de las frecuencias correspondientes a una resonancia en las curvas indicadas en 7.10, se efectuará una prueba de excitación a frecuencia constante, con una amplitud tal que la respuesta media cuadrática del elemento o medio correspondiente sea igual a la máxima que le corresponde según el capítulo 4. La amplitud de excitación necesaria en la mesa, se determinará aproximadamente de la razón respuesta-excitación obtenida para esa frecuencia durante la prueba 7.9.

7.12.- En caso de que la excitación de un elemento a la respuesta que le corresponde según capítulo 4, no sea posible por el procedimiento establecido en el artículo 7.11, debido a que se sobrepase la respuesta correspondiente a un elemento de apoyo del elemento en cuestión, o cuando el elemento no sea accesible para medir su respuesta, deberá realizarse una prueba separada sobre dicho elemento, montado sobre una base rígida, fijándolo por medios idénticos a aquellos por los que va montado en el equipo en condiciones reales.

7.13.- Las pruebas que se especifican en el artículo 7.5 deberán ser realizadas en una instalación de las siguientes característi-cas mínimas :

a) La mesa vibratoria deberá ser de dimensiones y masa suficientes para que se pueda lograr una oscilación predominantemente sinusoidal con el equipo montado sobre ella en condiciones de prueba. Se interpretará cumplida esta condición cuando la amplitud de la suma de las armónicas en la onda de desplazamiento de la mesa no sobrepase un 15 % de la amplitud de la fundamental.

b) La frecuencia de la mesa deberá ser ajustable entre 0,5 y 20 Hz con una precisión mejor que el 1 % del valor de la frecuencia ajustada, con el objeto de lograr estabilidad de excitación en los puntos de resonancia.

7.14.- Los equipos del tipo II, cuya disposición estructural no queda dentro de las disposiciones del artículo 2.3, deberán ser sometidos a una investigación detallada de sus modos de oscilación, aplicando en lo posible métodos similares a los de los artículos ..1 a 7.3. No se puede asegurar que las respuestas máximas de sus elementos sean posibles de obtener durante una prueba como la del artículo 7.12, por lo que será necesario verificarla, en todo caso, de la combinación por cálculo de las respuestas de sus distintos modos de oscilación, como se indicó en el artículo 4.6.

8.- APLICABILIDAD DE LAS PRESENTES ESPECIFICACIONES:

8.1.- Dado que las disposiciones de las presentes especificaciones se basan en muchos casos en una representación aproximada de las condiciones reales, los casos de dudas solo podrán ser resueltos mediante la discusión teórica de un modelo más detallado de la estructura del equipo, en forma similar a la indicada en los artículos 4.6 y 7.14.

Modificación del artículo 7.5.

b) Reproducir en los distintos elementos las respuestas máximas que le correspondan según las especificaciones del capítulo 4 (véase 7.11). Las tensiones elásticas máximas durante esta prueba, combinadas con las que se calculen para los esfuerzos de servicio especificados, deberán quedar dentro de los límites del artículo 4.4. El equipo no deberá presentar, después de la prueba, ningún daño, desplazamiento ni alteración de sus elementos, y deberá encontrarse plenamente apto para pasar cualquiera de las otras pruebas de recepción especificadas.

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Contents

1. Introduction

Japan is well known for the most frequent occurrence of earthquakes in the world.

It is absolutely necessary for the electric utility companies to make complete provisions for the safety and efficient restoration of electric facilities after an earthquake because the utility has a responsibility for the continuous supply of electricity to the community.

We wish to explain here the many examples of seismic damage to electric utilities which have occurred in Japan in the past and the measures which the electric utilities have adopted to counteract seismic effects.

2. The Earthquake Hazard

Magnitude 8 earthquakes have occurred about once every 10 years; and M.7 or less occur almost yearly. Following is a brief description of the damage caused by major seisms in modern times.

a. The Kanto Earthquake, M7.9 (1923) extensively damaged electrical facilities in the vicinity of Tokyo and Yokohama. Service restoration took several months. In later earthquakes the electrical system was more resistant, damage was considerably less severe and service outages lasted only a day or two.

b. The Fukui Earthquake, M7.3 (1948) led to adoption of an earthquake strength provision in the Japanese Building Law.

c. The Niigata Earthquake, M7.5 (1964) was the first occasion on which the infrastructure of a modern Japanese city was subjected to a damaging seism. Many buildings and industrial structures were damaged by soil failures. The utilities (water, gas and electricity) were all simultaneously disrupted, so the city's functions were virtually paralyzed.

d. The Matsushiro earthquake swarms (1965 to 1968) were only moderately damaging, but provided valuable opportunities for observation of seismic response of full-scale structures and to develop seismic prediction techniques.

e. The Tokachi-offshore Earthquake, M7.5 (1968) disrupted various telecommunication facilities. In its aftermath, the Seismological Prediction Liaison Council was established.

f. The Miyagiken-oki Earthquake M7.4 (1978) caused much damage to the utility systems of Sendai City.

In all of these earthquakes, we have never seen any major damage to generating equipment, due to seismic shaking, either in thermal or nuclear power plants. In each of the Niigata, Tokachi, and Miyagi earthquakes, apparently due to ground liquefaction or subsidence, there was typically one instance of minor damage to a thermal power plant, affecting the fuel-feed system or the buried distribution piping.

Table 1 shows the number of cases of seismic damage in overhead transmission and distribution facilities in the various earthquakes. There was no damage due to ground shaking, but we perceived secondary damage due to ground settlement or ground cracking and liquefaction.

There was no damage to structures on firm ground. Some damage occurred to pole-transformers due to support failure but there were very few cases in which the transformers actually dropped off the poles.

We did not perceive any damage to underground facilities due to ground shaking. Secondary damage, owing to soil liquefaction, was slight in tunnels and direct burial lines, but comparatively great in duct lines. (See Table 2).

Table 3 shows many examples of damage to substation transformers, circuit breakers, and lightning arresters, disconnecting switches and potential transformers.

3. Experiences and Lessons Derived From the Miyagiken-oki Earthquake

On 12 June, 1978, the Miyagiken-oki earthquake, M7.4, shook the Tohoku district. Damage was especially great, with 27 persons dead, more than 10,000 persons injured, and at least 172,000 houses collapsed. Public utilities were severely damaged.

Electric generating capacity of 1130 MW went out of service and about 680,000 houses (20% of all the customers in the district) suffered power failure.

Both the Sendai and Miyagi 275 KV substations, as well as 7 units of one 154 KV station, and 9 units of one 66 KV station, were heavily damaged, mostly in bushings, disconnecting switches, arresters, and circuit breakers. (See Figure 1).

Four thermal electric generating units tripped out of service. There were no outages resulting from damage to a boiler, turbine or generator. We did find some damage to auxiliary machines and, in one case, in the piping system.

There was no damage which caused malfunction of transmission lines. On the other hand, in the distribution lines there was breaking and tilting of supports. Also, we found damage by tilting of transformers situated in poor soil areas.

Most of the damage to power equipment comprised breakage of large porcelain tubes at high voltage substations.

3.1 Damage to Substations

In the aftermath of the earthquake, soil conditions were investigated at the 275 KV Sendai Substation, which had experienced the most extreme damage among all of the substations in the area. Reasons for the unusually severe damage are described below.

It is clear that there were significant differences of ground acceleration and predominant vibration frequency on different types of soil.

Ground acceleration was increasingly amplified in moving from cut ground to native ground to filled ground, as shown in Table 4. (However, damage to transformers and porcelain type equipment was almost the same for those installed on filled ground and on native ground.)

At the site of damaged 275 KV equipment, we conducted vibration tests and a response analysis. Almost all of the devices lacked strength under dynamic design conditions (resonant frequency, sinusoidal 3-wave with 0.3G). As a consequence, three principal causes of damage appeared, as shown in Table 5.

Judging from observed damage of equipment like condensers and arresters, it appears that these devices interact with each other by way of the conductors which connect them.

Seismic strength of current transformers is considerable. Their vulnerability lies in their susceptibility to porcelain tube flange-cracks. Also, equipment adjacent to these is apt to break due to interaction with these very rigid devices.

Table 6 lists bushing damage of the 275 KV transformers at Sendai Substation. The damage initiated with breakage at the lower inside parts of the porcelain tubes. See Figure 3.

In order to verify this, we have done field vibration tests and also a response analysis of the combined system of foundation and ground. The results are shown in Figs. 4 and 5. From the results of these tests, we could confirm that when the natural frequency of the transformer bushing, natural frequency of the foundation-and-ground system, and predominant frequency of ground motion are all approximately equal, the bushing response becomes several times larger than the ground acceleration.

This is because the use of rubber pads for noise suppression results in strong rocking of transformer and this induces resonant amplification of the seismic input to the bushings.

3.2 Damage to Transmission and Distribution Facilities

As regards transmission facilities, there occurred a ground crack around the base of a steel tower, and partial wall breakage. Also, there were a few minor damages such as tilting of poles and loosening of guy wires in poor ground.

The underground duct-lines, due to lessons derived from the Niigata earthquake, were in steel pipes; also, location of the lines was carefully selected. Therefore, there was no damage to duct-lines due to soil movements.

The most notable feature in the distribution facilities was extensive breakage or tilting of poles and supports in Sendai City and its environs, especially on poor ground. The lessons derived from the Niigata and Tokachi seisms had been adopted to prevent seismic damage; so, in this earthquake there was no falling down of transformers, and also no damage to substation bus-bars.

Damage to the supports and wires caused by ground shaking was, in reality, very slight in distribution systems. By far the greatest part of the damage occurred for secondary reasons, such as ground shifting and liquefaction.

In the telecommunication system there occurred breakage of wave guides at micro-wave stations, but this did not interfere with their operation.

4. Countermeasures for Earthquakes by the Electric Utilities

Based on review of the damage caused by the Miyagiken-oki earthquake, aseismic design concepts for power equipment have been developed.

4.1 Substation Facilities

In case of substation facilities, especially large equipment such as transformers and circuit breakers formerly were designed with seismic coefficient of 0.5.

However, the damage experienced in recent seisms has demonstrated that, if the predominant frequency of the ground and a natural frequency of the device match each other, a resonance [illegible]menon occurs and this results in a larger stress than that d[illegible] a 0.5g static equivalent load.

Therefore, a dynamic design standard input (resonance, sinusoidal 3-wave with 0.3G maximum) has been adopted for the large, critical equipment, especially at 500 KV substations. (See Table 7).

The substation facilities consist of the foundation (including the ground), the supports and the devices. Strength is decided by dynamic design rather than from a static equivalent.

Taking into consideration future seisms and also occurrence of large earthquakes over the last 50 years in Japan, peak ground acceleration of 0.3g has been adopted as a standard seismic loading.

As shown in Fig. 6, natural frequencies of the 275 KV and 500 KV class equipment exist over a wide frequency range, in general. Therefore, the possibility is high to develop resonance under seismic loading.

As a conservative dynamic loading, it is necessary to consider sinusoidal wave input which has a natural-frequency equal to that of the equipment.

Fig. 7 shows that when the equipment, idealized as a one-dimensional lumped-mass system, has resonance against a ground motion input, response amplification will occur. This amplification of acceleration is compared with those of resonant sinusoidal 2-wave & 3-wave inputs.

Here, the amplification due to resonant sinusoidal 2-wave input covers almost the full range of acceleration response magnification due to an actual seismic wave, and no earthquake response exceeds that induced by the resonant sinusoidal 3-wave. Accordingly, we have adopted the resonant sinusoidal 2-wave as the input waveform for designing substation facilities. Tables 8, 9 and 10 show seismic design criteria for substation facilities.

Improvement of dynamic performance of transformer bushings may be achieved by eliminating rubber pads between transformer and foundation inasmuch as sound isolation devices can cause adverse reaction to seismic inputs. Figures 4 and 5 indicate how ground accelerations at Sendai Substation were amplified from foundation to tank to bushings.

Usually, special investigation of soil conditions is performed only if fill is to be placed. If the existing ground is known to be weak, improvements can be made, e.g., densification or consolidation, pile placing; or bearing pressure can be reduced by using larger foundations.

See Figures 9 and 10 for suggestions for improvements of seismic strength of porcelain type equipment.

4.2 Nuclear Power Facilities

We have never yet experienced damage to a nuclear power plant due to an earthquake. However, to guarantee that no release of radioactivity could result from a seism, we practice stricter seismic-proof measures for nuclear equipment than for other power equipment.

At nuclear power facilities, the building, structure, equipment and piping systems are all classified, by seismic categories related to the safety of the facilities, and their seismic designs are conducted in accordance with their classification.

Two kinds of seismic motions (on the basis of the strongest seism and the marginal seism) are stipulated based on the seismic history, active faults, and seismic geological structure at the generating power sites; and seismic design incorporates dynamic analytical methods.

Building foundations are usually installed directly on bedrock. The piping systems are designed to prevent resonance with the building, by being made as stiff as possible.

In order to obtain maximum safety, a seismic detecting device is installed to initiate shut-down of the reactor in case of an earthtremor greater than anticipated.

When a reactor suddenly is "scrammed" during its operation owing to earthquake shocks, thorough inspection of the nuclear power facility must be carried out immediately to determine if the plant has been damaged.

4.3 Hydro-power Dam Installation

Aseismic design is required, based on a pseudodynamic process. However, for especially important structures, dynamic response analysis is included in the design process. Also, for principal hydraulic dams, continuous seismic monitoring instrumentation has been set up. Thereby, the actual dynamic properties of the major dams have been determined.

4.4 Thermal Power Generating Facilities

For thermal power facilities Table 11 shows seismic design criteria for each equipment.

4.5 Transmission Facilities

In the design of overhead transmission lines, the stress due to wind loads is always greater than seismic.

In the case of transmission route selection, we avoid bad ground as much as possible.

Although underground transmission cables are not directly affected by the ground shaking itself, the following design precautions are necessary:

(a) When the routes are selected, liquefiable sand subgrades or filled ground are studiously avoided.

(b) We use flexible joints in buried lines, so that the ground strains do not transfer to the buried conduits.

(c) Potheads and oil supply devices are designed for shaking loads.

4.6 Distribution Facilities

Distribution facilities have usually exhibited substantial resistance to earthtremors. By selecting the routes and duplexing the circuits, sufficient seismic protection is obtained, as long as we avoid poor ground.

4.7 Telecommunication Equipment and Computers

Seismic design for telecommunication equipment and strengthened techniques of installation of flexible micro-wave guides have all been carried out. The important facilities are provided with permanent emergency power sources. Therefore, by the use of duplex circuits, the telecommunication functions will be maintained.

Computer systems have been provided with sufficient protective devices to insure that operations will not be interrupted by seisms.

4.8 Service Restoration Policies

All electric power companies in Japan have established their own emergency damage recovery systems.

At a time of unexpected accidents power is sent to another company if a power supply shortage occurs. This is done under a mutual aid agreement. If necessary, personnel and materials will also be dispatched to assist in emergency recovery.

First priority for restoration of power is assigned to those facilities vital for prevention of further damage and for speedy recovery of community functions. This includes hospitals and subway shopping streets, which are important places for sheltering and conserving lives. Also, recovery facilities such as damge-relief headquarters, governmental agencies and gas, water, and traffic systems are assigned high priority.

Also, high priority is assigned to news media as they are essential to allay anxiety in the community.

4.9 Research Needs

a. It is desirable for us to conduct a seismic study, with an evaluation of center-clamp-type bushing failure mechanisms, by examining transformer behavior in actual seisms.

b. We wish to evaluate the effect of ground improvement, by conducting tests on various improving methods, such as using soil-cement.

c. At the Nuclear Industrial Center, a large seismic shaking table is under construction. This apparatus has a 15m x 15m vibrating platform weighing 1,000 tons. It is a biaxial machine, which will be able to develop maximum inertia force of 3000 tons. We plan to conduct full-scale seismic reliability tests on large nuclear power components to determine seismic design margins.

5. Conclusion

We have described in this paper the seismic resistance measures adopted by the Japanese electric utility industry.

Early restoration of power facilities at the time of seismic disaster will contribute greatly to the recovery of both civic function and industrial-economic activity and will be reassuring to the public.

We of the electric utility industry recognize that we must take all reasonable countermeasures to protect against effects of earthquakes in Japan.

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Table 1. Damage to overhead facilities

(1) Transmission

Installation	Unit	Tokachi	Niigata	Ebino	Shizuoka	Matsushiro	Miyagi
support (tower)	unit	56	61	1	0	0	33
support (wood)	unit	537	37	0	0	0	16
electric wire	wire	36	14	0	1	2	0
underground	circuit	3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

(2) Distribution

Installation	Unit	Tokachi	Niigata	Ebino	Shizuoka	Matsushiro	Miyagi
support	unit	7,555	1,068	550	0	30	2,317
high voltage electric wire	wire	9,076	4,451	311	0	73	944
transformer	wire	3,019	1,193	31	0	0	4,085

(CRPN, Nov. 1970)

Table 2. Damage to underground transmission facilities

Name of seism	Damage state: Duct: Break	Duct: Man's tunnel	Duct: Others	Cable: Bad insulation, oil leaks	Cable: Joint slip off	Cable: Joint failure	Cable: Other	Transmission Fault (circuit)	Remarks
Niigata	(span) 19	2	11	(span) 15	2	8	2	7	[illegible] circuits in Niigata City (underground transmission lines)
Tokachi	-	-	[illegible]	(span) 7	-	-	-	-	Cables-seams of insulation joints, and oil leaks of plumbing
Miyagi	-	-	7	-	-	-	-	-	Slight, like man's tunnel cracks

(DKH, Dec. 1979)

Table 3. Damage to substation equipment

Area	Date occurred	Epicentre	Intensity	Magnitude	Distance from epic. (km)	Damaged substation facilities: Air-blast breaker	Arrester	Others	Remarks
HOKKAIDO	[illegible]	Tokachi-oki	[illegible]	[illegible]	[illegible]			[illegible]	
	[illegible]	[illegible] off-shore	[illegible]	[illegible]	[illegible]			[illegible]	
	[illegible]	[illegible] off-shore	[illegible]	[illegible]	[illegible]		[illegible]	[illegible]	Tokachi
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		[illegible]	[illegible]	
	[illegible]	[illegible] peninsula off.	[illegible]	[illegible]	-		[illegible]	[illegible]	
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		[illegible]		
TOHOKU	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		[illegible]	[illegible]	
	[illegible]	[illegible] Miyagi-Ken	5	[illegible]	[illegible]			[illegible]	[illegible]
	[illegible]	Niigata City	[illegible]	[illegible]	[illegible]			[illegible]	Niigata
	[illegible]	[illegible] Niigata-Ken	[illegible]	[illegible]	[illegible]		[illegible]		
	[illegible]	Miyagi-Ken [illegible]	[illegible]	7.4	[illegible]	[illegible]	[illegible]	[illegible]	Miyagiken-oki
KANTO	[illegible]	[illegible] of Ibaraki-Ken	[illegible]	[illegible]	[illegible]	5		[illegible]	
	[illegible]	[illegible]-Ken [illegible]	[illegible]	[illegible]	[illegible]	3			
	[illegible]	Saitama-Ken [illegible]	4	6.1	[illegible]	1	[illegible]		
	[illegible]	[illegible] of Ibaraki-Ken	[illegible]	[illegible]	[illegible]	1			
	[illegible]	[illegible] of [illegible]-Ken	[illegible]	[illegible]	[illegible]	1			
CHUBU	[illegible]	[illegible] City	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	
	[illegible]	[illegible]	[illegible]	[illegible]		2			Matsushiro
	[illegible]	ditto	[illegible]	[illegible]				[illegible]	
	[illegible]	Central [illegible]-Ken	[illegible]	[illegible]	[illegible]	[illegible]		[illegible]	
	[illegible]	[illegible] off-shore of [illegible]-Ken	4	[illegible]	[illegible]		[illegible]		
	[illegible]	[illegible] peninsula offshore	[illegible]	[illegible]	[illegible]				
	[illegible]	Boundary of [illegible]	[illegible]	[illegible]	[illegible]		[illegible]	[illegible]	
	[illegible]	East of [illegible]-Ken	[illegible]	[illegible]	[illegible]	1		[illegible]	
HOKURIKU	[illegible]	North of Fukui City	[illegible]	[illegible]	[illegible]		[illegible]	[illegible]	Fukui
[illegible]	[illegible]	[illegible] of [illegible]-Ken	[illegible]	[illegible]	[illegible]			[illegible]	
[illegible]	[illegible]	[illegible] of [illegible]-Ken	[illegible]	[illegible]	[illegible]			[illegible]	
Total	-	-	-	-	[illegible]	50 units	[illegible] units	[illegible]	

[illegible]

Table 4. *Ground surface acceleration on various types of soil*

Soil type	Maximum ground acceleration (gal), calculated*	Dominant frequency (Hz), calculated*
Fill	290 to 550	1.9 to 8
Undisturbed	270 to 380	2.5 to 8
Cut	210 to 230	4 to 10

*Based on multiple reflection theory, assuming 200 gal in firm ground.

Table 5. Damage to porcelain type equipment (275 KV)

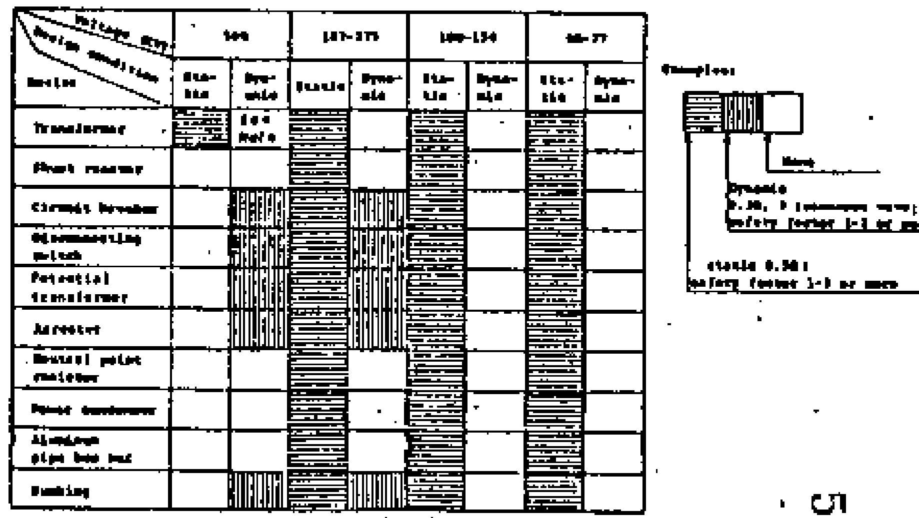
Classification	Study Design	Safety factor*	Number installed	Number damaged	Cause of damage: Lack of strength	Cause of damage: Mutual interference by leads between devices	Cause of damage: Secondary damage by other device breakage
275 kV ABB Breaker (air)	Static	0.6	3 units	3 units	1		
275 kV GCB Breaker (gas)	Dynamic	0.9 (Double load)	6 "	4 " (1 unit under construction)	1		
275 kV LA Arrester	Static	0.7	12 " (part)	11 " (part)	1		
" "	"	1.2	6 " (")	6 " (")		1	
275 kV CT Condenser	"	1.6	12 " (")	6 " (")			1
" "	"	2.0	3 " (")	1 " (")			1
" "	"	2.8	3 " (")	3 " (")			1
275 kV DS	"	0.9	24 "	3 " (one under construction)	1		
275 kV LPD	"	1.3	6 "	3 "		1 (1 units)	1 (2 units)
" CC	"	1.3	12 "	10 "		1 (2 units)	1 (8 units)

*Safety factor determined from input of 0.3G, at resonance, sinusoidal 3-waves, at the base of the equipment's support.

Table 6. Damage to transformer bushings

Capacity	Extent of damage
275/154 KV 250 MVA	No. 1 Transformer 275 KV, No. 2 bushing A, B phases: Breakage of upper porcelain tube, C phase: Breakage of upper porcelain tube
	No. 2 Transformer 275 KV, No. 2 bushing (under installation); oil leakage B, C phases: Breakage of upper porcelain tube tip-out; oil leakage B phase: Breakage of upper porcelain tube cracks; oil leakage
	No. 3 Transformer 275 KV, No. 2 bushing 3 phase (all the same) upper porcelain tube tilting, oil leakage, (No pressure cut-off in O ring)

Table 7. Seismic design condition, classified by the voltage and the type of device



Note: In the aftermath of the damage caused by the Miyagiken-oki earthquake, the dynamic design method has been extended to the transformers, also.

Table 8. General seismic design criteria for substation facilities

Items	Porcelain-type devices	Transformer bushing
Seismic design force, which is the basic input into ground surface: A	Indicial application of resonance sine 2-wave with 0.3G maximum	Same as at left
Amplification factor, due to the foundation: B	1.2	2.0 (inclusive of amplification of transformer proper)
Uncertainty factor, based on vertical acceleration, effect of connecting conductors, etc.: C	1.1	1.1
Correction factor (B x C): D	1.2 x 1.1 = 1.3	2.0 x 1.1 = 2.2
Seismic design force calculations.	A x D = resonance sine 2-wave with a 0.39G; but, in terms of traditional usage: 3-wave acceleration $= 0.39G \times \frac{1}{1.3} = 0.3G$ (See Note)	A x D = resonance sine 2-wave with a 0.66G; but, in terms of traditional usage: 3-wave acceleration $= 0.66G \times \frac{1}{1.3} = 0.5G$ (See Note)
Seismic force to be used in design of devices	Resonance sine 3-wave with 0.3G maximum applied at the lowest parts of the device	Resonance sine 3-wave with 0.5G maximum, applied at the lowest part of bushing pocket

Note: This is a conversion of the desired (newer) 2-wave input to the equivalent "traditional" 3-wave input. Amplification is greater for 3-wave input, as shown in Fig. 7.

Table 9. Seismic design of porcelain devices

1. Ground	The equipment should be built on ground which has more than 150 m/s speed of S-wave, or Standard Penetration Test value, N>5.
2. Design approach	Quasi-resonance method.
3. Seismic design input	Resonant sine 3-wave with 0.3G maximum (indicial application)
4. Dynamic load point	The lowest parts of the device.*
5. Judgement criteria	The stress which is generated in the porcelain tube is not to overrun the destructive stress value.
6. Range of Frequency	As for the device whose natural frequency is in the range of 0.5 - 10 Hz, use the natural frequency; as to those which are under or above this band: in case of less than 0.5 Hz, assume 0.5 Hz is its natural frequency; in case of 10 Hz or more, assume 10 Hz is its natural frequency.

Table 10. Seismic design criteria for transformer bushings

	Primary check	Secondary check*
1. Ground	The standard is to put it on ground of more than 150 m/s speed of S-wave (or N > 5) Amplification of ground motion by combination of soil foundation-and-transformer should be less than 2.	
2. Design Approach	Quasi-resonance	Flange system: Center clamp system Quasi-resonance: Actual seismic process
3. Seismic design input	Indicial application of resonant sine 3-wave with 0.5G maximum (From Table 8)	Chiefly, indicial application of resonant sine 2-wave with 0.3G maximum. Use several kinds of seismic waves of various predominant frequencies so that they may be well distributed.
4. Dynamic load point	The lowest parts of bushing pocket	Ground-surface (response at the lowest part of bushing pocket is obtained from calculation and experiment).
5. Critical of Judgement	Flange system. The same as porcelain devices. The mouth should not open. Even if it is adjudged "no good", it is acceptable if passed with a secondary check.	1. It is necessary to maintain operability. 2. The stress which is generated in the porcelain is not to overrun the destructive stress value.
6. Frequency range	If natural-frequency is in the 0.5-10 Hz range in bushing pocket, use the natural-frequency itself. In those beyond this range: 0.5 Hz or less, assume 0.5 Hz; 10 Hz or more, assume 10 Hz as natural-frequency.	Same as at left.

*To be applied if (a) bushing fails to satisfy the "primary" check, or (b) ground motion amplification by combined transformer-foundation-and-soil is greater than 2.

Table 11. Seismic design criteria for thermal power facilities

Equipment	Seismic coefficient	Remarks
Main building	0.2G	Based on Code*
Boiler	0.2G	Based on Code. Horizontal seismic load is transferred to building via bucksteys.
Boiler structure	0.2G	Based on Code and Steel Structure Criteria of Academy of Architecture. For seismic design, assume boiler, tanks, heat exchangers and piping are full of water.
Pipings	0.2G	Pipe supports are designed for seismic load. Vibration of piping is prevented by using snubbers. Principal piping gets dynamic analysis.
Turbine generator	(0.2 to 0.3G)	Seismic loads do not govern the design. Actual seismic resistance capacity is about 0.5G.
Main auxilary machines	(0.2 to 0.3G)	Seismic loads do not govern the design. Based on experience, there is a seismic resistance capacity of more than 0.2G. Static strength of the casings are checked for lateral forces.
Phase bus-bars	0.3G	Wind load (40 m/s) may govern.
Protective relays	0.22G	This design is based on JEC 174. For components of low seismic resistivity, shake tests are conducted.
Fuel oil tanks	0.3G	Based on Fire Defense Law. Flexible joints are used. Sloshing requirements determine freeboard height.
LNG tank	0.3G	Based on Electric Industrial Law, and Seismic Intensity Code. In some cases, a dynamic analysis is conducted.

*Code = "Building Standard Law"

(OHM, June 1978)

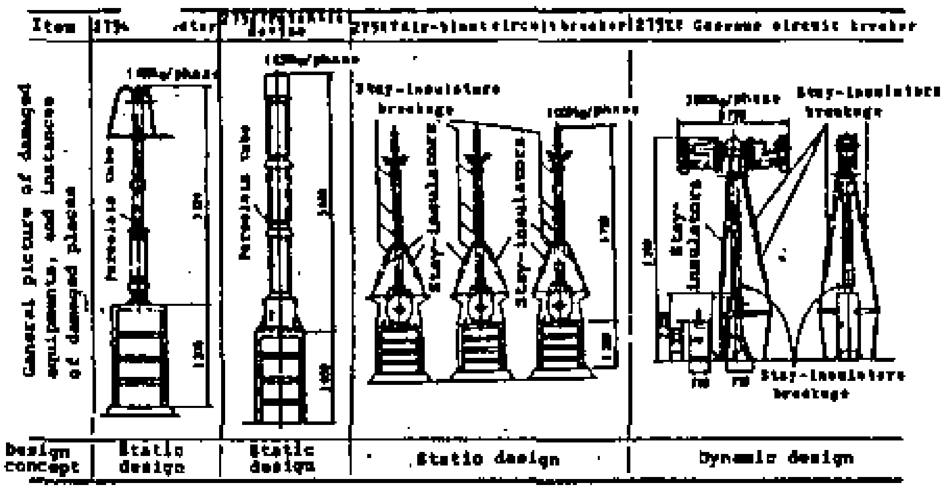


Fig. 1. Damage to the equipment

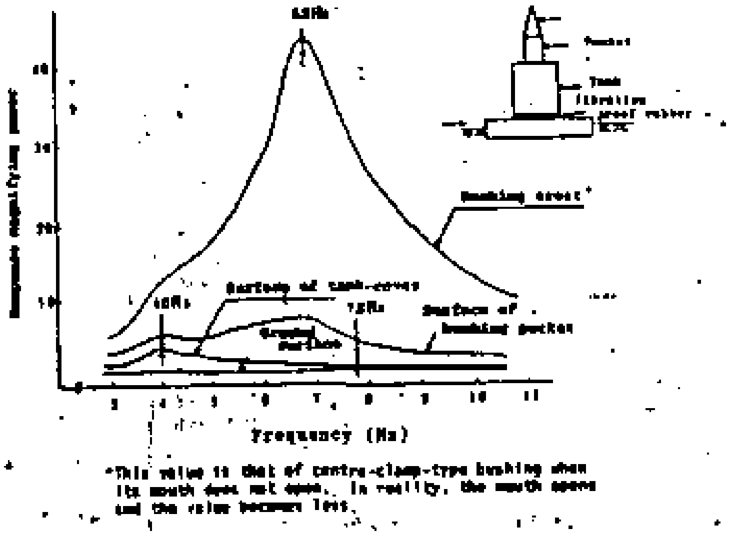


Fig. 4. 275 KV transformer's frequency charcteristic at Sendai Substation (sine-wave acceleration time's calculation)

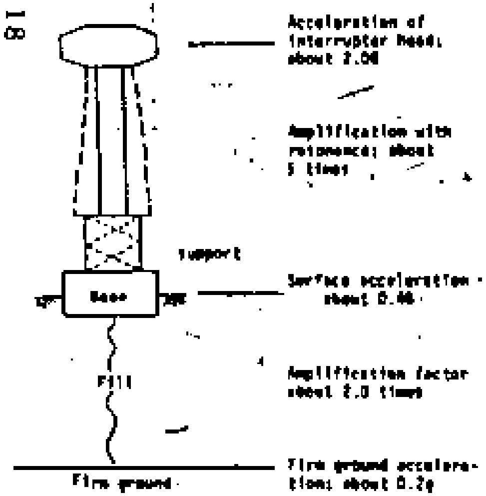


Fig. 2. Response of damaged circuit breakers on filled ground at Sendai Substation

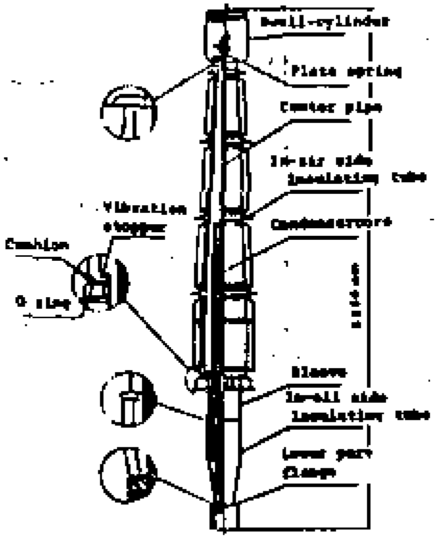


Fig. 3. Structure of bushing

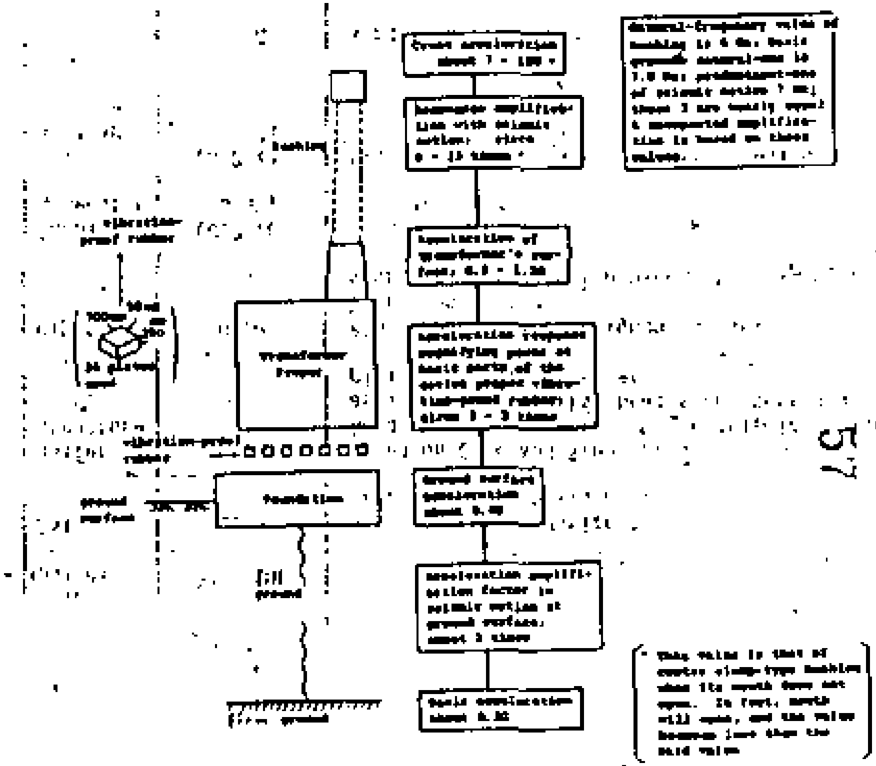
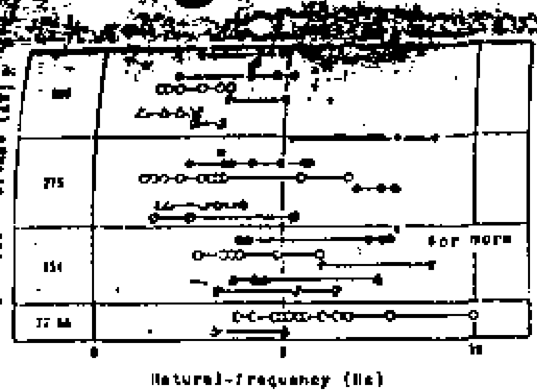


Fig. 5. Chart of response feature of transformers damaged at Sendai Substation.



(Denki Kyodo Kenkyu | Vol 34,No.5)

- bushing for transformer
- instrument transformer
- current transformer
- ○ insulator-type circuit breaker
- ● tank-type circuit breaker
- △ arrester
- disconnecting switch

Fig. 6. Range of natural-frequency of substation equipment

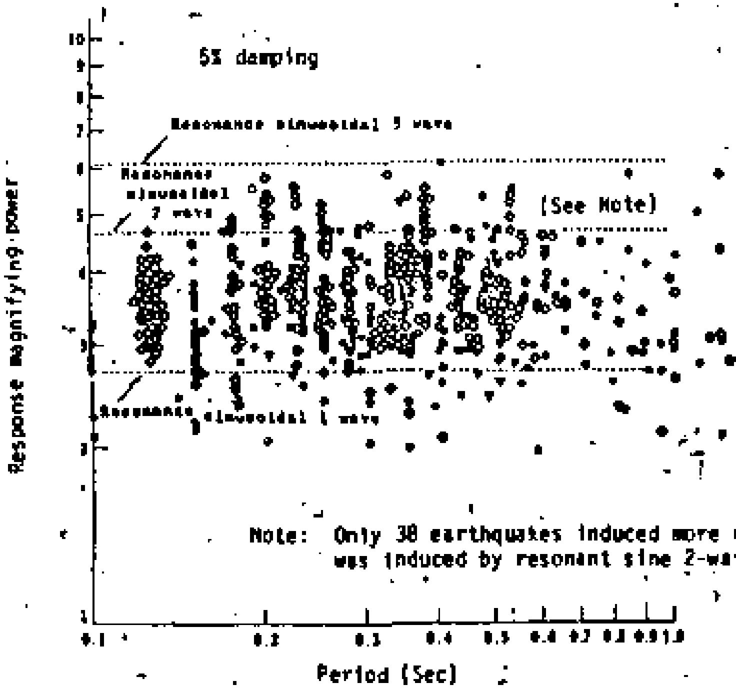
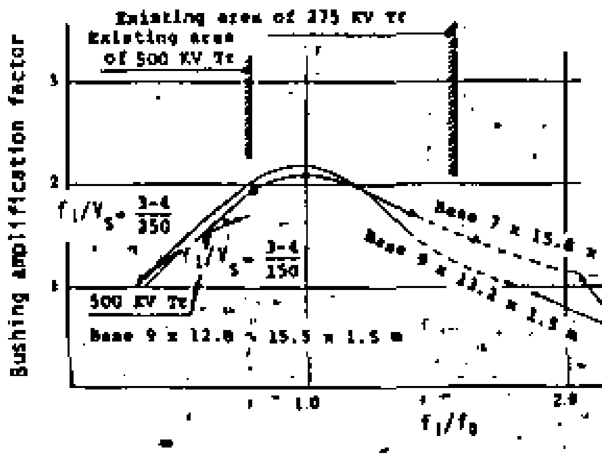


Fig. 7. Response to resonant sinusoidal input compared to response to actual seismic accelerograms.



f_0: natural frequency of system comprising transformer-foundation-ground

f_1: natural frequency of bushing, alone

V_s: shear wave velocity

Note: Bushing, mounted without rubber blocks, was subjected to 3-wave sinusoidal input at different values of f_0.

Fig. 8. Bushing response as influenced by dynamic characteristics of transformer-foundation-ground.

(a) before strengthening (one-step stay; two directions)

(b) after strengthening (two-step stay; 3 directions; and support truss addition)

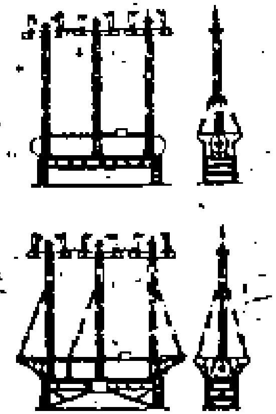


Fig. 9
Aseismic strengthening of 275 KV air-blast circuit breaker

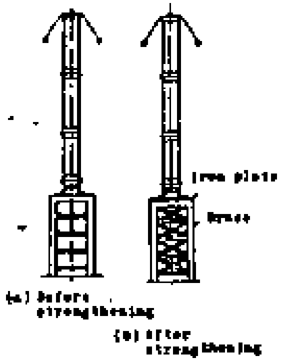


Fig. 10
Aseismic strengthening of 275 KV lightning arrester

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(10) Gupta, I. N. and Nuttli, W. O., "Spacial Alternation of Intensities for Central United States Earthquakes", Bulletin of Seismological Society of America, Volume 66, No. 3, June 1976, pp. 743-751.

(11) Oppenheim, I. J., "Vulnerability of Transportation and Water Systems to Seismic Hazard Methodology for Hazard Cost Evaluation", A Paper submitted to TCLEE Specialty Conference in Los Angeles, California, September 1977.

(12) "Seismic Analysis: Beaver Valley Power Station of the Duquesne Light Company, Shippingport, Pennsylvania", by Weston Geophysical Research, Inc., Weston, Massachusetts, Westinghouse Electric Corporation, "Draft of LWBR Safety Analysis Report", Material Document No. WAPD-LD(TS)-8, 1974.

(13) Buchanan, G. G. et al., "Facing the Cost of Water Quality", Journal AWWA, Volume 69, No. 4, January 1977, pp. 46-51.

NO

TABLE 1. - Distribution System Damage
Number of Failures per Pipe Group

Average Pipe Group Size in Inches	Percent of Total Length	Modified Mercalli Intensity*					
		6.8	7.2	7.5	8.3	8.7	9.0
4	2.9	0	2	6	154	1,105	4,221
6	51.5	4	25	97	2,678	19,470	74,339
8	19.0	2	9	35	961	6,989	26,686
10-12	10.5	1	5	19	525	3,815	14,568
14-16	4.2	0	2	7	191	1,391	5,310
18-20	3.6	0	1	6	157	1,141	4,357
24-30	4.3	0	1	6	157	1,141	4,357
36	1.8	0	1	1	54	392	1,498
42	0.5	0	0	0	10	71	272
48-50	0.7	0	0	1	15	107	408
50-84	0.7	0	0	0	5	36	136
TOTAL BREAKS		7	46	179	4,907	35,658	136,152

*MMI corresponding to a given ground acceleration based on Trifunac and Brady (6) correlation.

ELECTRICAL POWER AND COMMUNICATION LIFELINES

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The current state of knowledge
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ENGINEERING

ASCE, 1977

COMMUNICATIONS LIFELINES IN EARTHQUAKES

By J. W. Foss,[1] M. ASCE

ABSTRACT: State-of-the-art information is presented on protection of communications lifelines against earthquake damage. Seismic effects, design objectives, and strengthening techniques are discussed for cable and radio systems and for equipment in buildings.

INTRODUCTION

Communications lifelines, always necessary, become immensely vital in the period following an earthquake. Radio, television, telephone, telegraph, and other communications media must close the information gaps in normal life that are caused by earthquake damage. Obviously, communications are needed to summon emergency aid, to coordinate possible rescue and medical efforts, to collect damage data on utilities and other facilities, to aid speedy and safe reconstruction efforts, and to keep people informed. And, when there is danger of aftershock damage--for example, a weakened dam about to fail--communications can save thousands of lives by providing ample warning time for evacuation to take place.

Although there are many communications avenues, the strain during any reconstruction period will necessarily fall on those facilities which are still usable. This is where overloads or malfunctions can occur if system checks or controls are not incorporated into designs and engineering plans. For example, the telephone switching machines in the Los Angeles area were able to operate successfully after the 1971 San Fernando earthquake because the bulk of nonemergency incoming calls were selectively blocked. This prevented overloading the network and allowed the more important outgoing and local calls to be processed.

Perhaps the single most important deterrent to complete breakdown of communications is the redundancy in the many systems available. These systems are spread in a web-like matrix over regions significantly larger than destroyed

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areas. Path rerouting will divert major communications around the destroyed area while communications within that area will be partially restored with available mobile or transportable facilities prior to repair or replacement of the initial system. However, the enormous need for post-earthquake communications demands that measures be taken before an earthquake occurs to assure survival of as much of the total lifeline plant as is economically feasible.

NETWORK STRUCTURE

The lifeline referred to consists of communications networks made up of a series of nodes or end points interconnected by distribution systems as shown in Figure 1. Communications main centers or nodes most often consist of buildings, or structures housing equipment. The distribution system is generally configured of pole lines or buried cable as well as radio links that interconnect each node point. In most instances, the high diversity in these systems allows for loss or failure of certain elements without complete loss of communication in the specific areas where failures occur. The effects of system redundancy vary from one region to another. This makes it logical to consider seismic requirements on a local basis rather than codifying criteria independent of regional considerations. Using this approach is a sensible way to assure optimum and balanced seismic resistance of lifelines.

Working with the knowledge of likely earthquake epicenters and motion-distance effects for a particular region, one can perform a network analysis to find the desirable balance design level of seismic resistance. The distribution paths in the network form very complex circuits. From the standpoint of survivability studies, these may be simpler to analyze by using decomposition techniques to reduce them to a number of series or parallel systems rather than by studying the circuits themselves. An analyst requires not only the network and seismic data but also information on the fragility level of each network element. Generally, equipment in structures at the nodes will be significantly more vulnerable to ground excitations than the cable or radio distribution plant. And, because earthquake-induced motion will spread over large areas, much emphasis must be placed upon improving the fragility levels of equipment. This will be discussed in subsequent sections.

SEISMIC EFFECTS ON COMMUNICATIONS LIFELINES

Earthquake effects on communications will vary in magnitude and frequency of occurrence throughout the country as dictated by the complex action of global tectonics. Zoning maps portraying information on local seismic risk are being developed to aid system designers in choosing design levels appropriate to achieve optimum performance. Through the years, the sophistication used in developing these maps has

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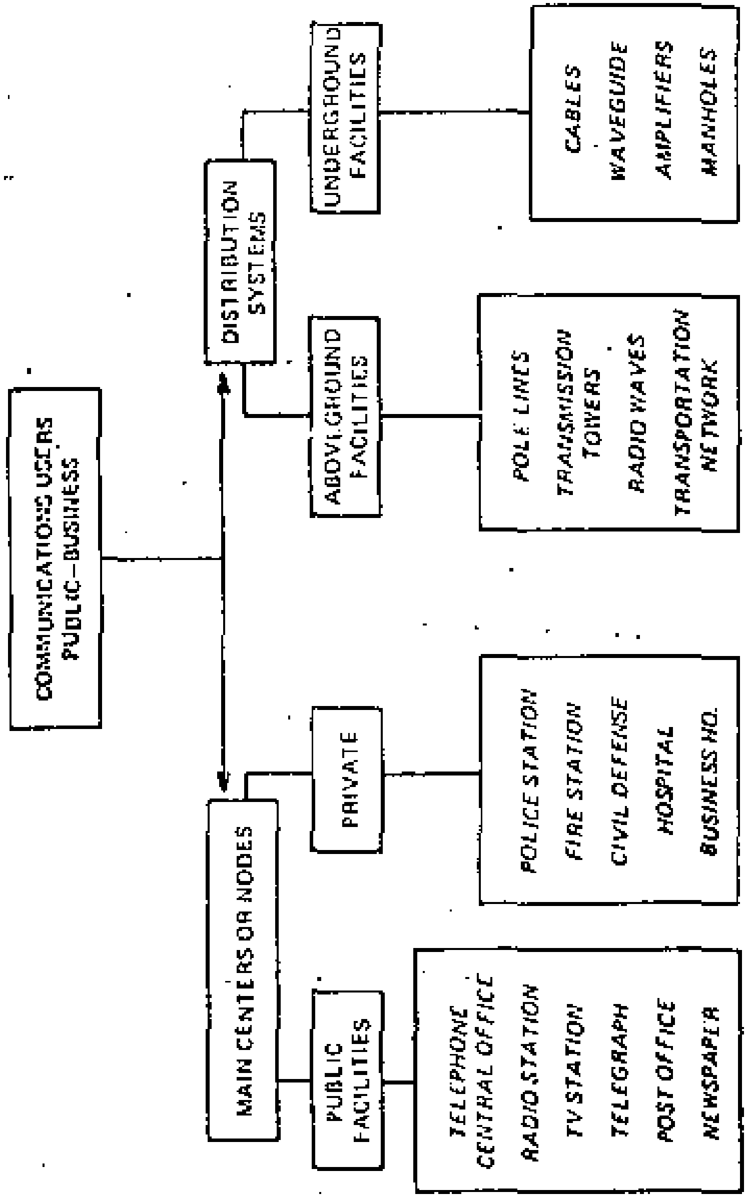

FIG. 1.–Communication network elements

been increasing, with later versions based upon microzonation techniques. Such techniques combine historical earthquake data and seismological and geological information with statistical models to predict likely earthquake occurrence and local effects.

The most complete recent report on earthquake effects and zoning appears in Reference 1, "Recommended Comprehensive Seismic Design Provision for Buildings," prepared by the Applied Technology Council, January 1977, and soon to be released. This reference contains ground motion regionalization maps of the United States showing effective peak acceleration and velocity levels not likely to be exceeded over a 50-year period. It also goes into considerable detail in defining how to obtain the recommended design levels for buildings and appurtenances. Reference 2 gives the building floor accelerations shown in Table I for the different seismic zones mapped in Figure 2. These criteria have been prepared for use in the design of new installations of central office equipment in Bell System communications facilities.

Table 1

Floor Accelerations for Different Seismic Zones and Locations in Buildings

Location	Floor Accelerations (g) in Seismic Zones 1	2	3	4
Ground Level 1st Floor	0.05 → 0.1	0.1 → 0.2	0.2 → 0.4	0.4 → 0.8
Upper Floors	0.2 → 0.3	0.3 → 0.4	0.4 → 0.6	0.6 → 1.0

The sensitivity to earthquake effects of communications buildings and structure-housed equipment is different than cable or radio distribution systems. The former are very sensitive to ground accelerations or repetitive shock-type loadings, while the latter are more sensitive to relative ground displacements and generally less sensitive to ground acceleration effects. Relative ground displacements include those occurring at the fault slip plane and areas of overthrusting and fissuring as well as the oscillations caused by compressional and shear waves propagating from the earthquake source.

While considerable work has been done to develop criteria on earthquake acceleration effects, only limited research has been done on the development of criteria covering ground displacements and relative motion. Displacement studies

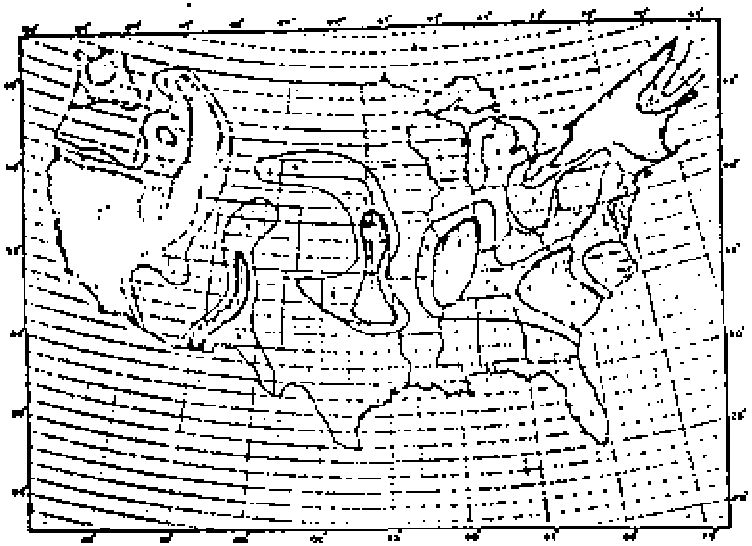

FIG. 2.-Earthquake zoning map (Reference 2)

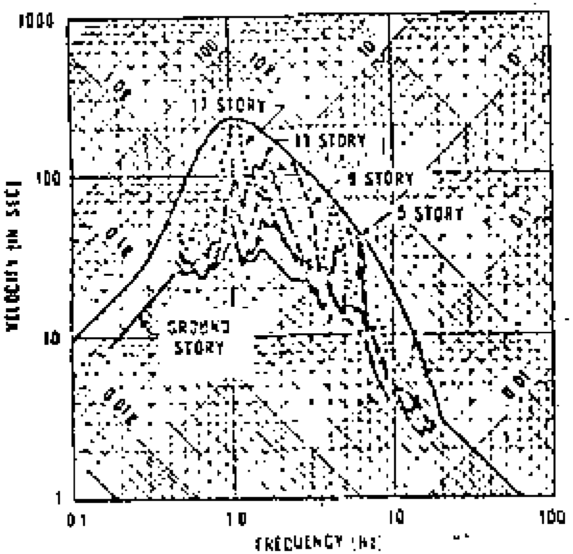

FIG. 3.-Typical Zone 4 multistory building upper floor response spectra (Reference 2)

have been made for use in the design of the Alaskan pipeline (Ref. 3). Reference 4 presents a brief review of some formulas used to estimate relative ground motions. Much additional work is required to develop usable criteria for different regions of the country, different soil site conditions and for different types of seismic excitations.

DISTRIBUTION NETWORK PROTECTION

- Cable in the Ground

Earthquakes may damage cables buried in shallow trenches where such cables cross the region of fault slipping, rupturing or fissuring and where relative motions induced by seismic waves in the ground "pulse" the cable along its length. Compression of the cable by local hard objects such as stones or pavement can also cause damage--especially to the coaxial variety. Most cable has a great deal of flexibility and can be significantly bent and twisted with minimal effects upon transmission. Also, the shock resistance, or the ability of the cable to withstand ground velocity changes, is sufficiently great that this type of loading is of little concern during earthquakes.

Axial elongations of the cable pose the greatest threat to its ability to carry signals. Elongations can occur on cables crossing faults as the ground ruptures and shears at these locations. They can also occur as seismic compression waves move through the ground along the cable length. Possible effects include: (1) a failure in tension by necking down at a particular location, (2) a capacitance change sufficient to cause signal impairment, or (3) the separation of connections at manhole and building splices.

A variety of strengthening procedures are used to improve the resistance of important cables which cross known active faults. One such procedure designed to protect major high-capacity telephone trunk lines crossing the San Andreas fault in California involves using steel double-armored cable laid in "S" loops along a 1,000 foot trench that straddles the fault. The trench is 4 foot deep and 10 foot wide at the top with 45-degree angle side slopes. Noncohesive gravel backfill is used in the trench to allow the cable to undergo large motions without damage. Further, the cable trench is placed at an angle to the fault crossing so that fault shifting in the expected direction tends to normalize the cable position and minimize stress.

Protection of buried cable from elongation failures caused by seismic stress waves traversing the ground may best be accomplished by designing displacement tolerances into the cable run. This may be done by laying cable loosely in trenches; by providing loose, porous, noncohesive backfill; and most important, either by leaving some slack or by providing full cable strength at connections in manholes, at splice points, and at the building attachment.

Also at the building attachment, the cable must be protected against shearing. Shearing is caused by (a) seismic-induced settling of backfill soil around the building causing cable motion relative to the building, or (b) seismic-induced building settling causing cable motion relative to the soil. At locations where soil conditions are poor or settling is likely, the cable may be protected by placing it in a length of steel pipe anchored at one end to the building foundation.

• Cable on Pole Lines

Cable supported by poles extending from the ground will be exposed to earthquake effects similar to those described for buried cable. In addition, a whipping motion of the cable-pole system may cause further damage. Heavy splice cases and amplifier equipment on suspended cable spans are likely to experience severe cable oscillations during major earthquakes. Such oscillations are caused by response of the pole-line system to motions normal to the run as well as to seismic "pulse" motions in the ground in a direction along the run. In the latter case, one concern is with motions caused by a slack cable with a heavy mass attached suddenly becoming taut. The principal strength member of most aerial cable is a continuous steel strand to which the cable is attached. The strand will carry all major stresses along the line and prevent axial loadings from straining splices or other line-supported apparatus. Methods of protecting pole-line systems against earthquake effects include securely attaching the cable and other supported apparatus to the steel strand and the strand to the supporting poles. Pole-mounted equipment, such as amplifier cases, must be securely attached to prevent separation which can cause line failure and produce dangerous falling objects.

• Radio and Television Towers

Radio and television towers may be structured as free-standing cantilevers on the ground; guyed to the ground; or free-standing and/or guyed on top of other structures such as buildings. Most towers are of relatively light metal lattice or pole-type construction. Because of the relatively light weight of the tower structure, wind loadings generally dictate the design strength even in areas where earthquake codes stipulate seismic loadings.

There may be, however, seismic considerations important in the design of certain types of towers that have not in the past been accounted for.

1. Towers supported upon multistory buildings will experience earthquake motions that may be amplified by the supporting building structure. Multistory buildings may whip the tower horizontally and also propagate amplified vertical motions to the tower. When vibrational frequency-matching is right, these combined horizontal and vertical motions induced by the earthquake may produce tower stresses significantly higher than those caused by wind loadings. Computer solutions considering the dynamic response to seismic forces are generally required to analyze the building-tower entity. The point of maximum stress, like that which occurs on flagpoles similarly supported and exposed to vibration inputs, is likely to occur between the tower base and mid-tower height.

2. Tall radio and television towers that are guyed and supported on the ground are often constructed with a substantial distance between the concrete blocks which provide anchorage to the guy cables. Thus, in some cases, earthquake dilatational waves in the ground may possess half wavelengths that are as short or shorter than the distance between the anchor blocks. Under such conditions, the seismic motions may slacken or tense the guy cables unequally and thus induce damaging relative motions between guy point anchorages. Vertical motions acting at the same time may further lead to excessive stresses in the guy system or tower mast.

3. Certainly, communications equipment attached to towers must be well supported to be capable of sustaining the seismic vibration response of the mast. Antennas, feedhorns, waveguides, couplers, etc., should be rigidly attached to prevent resonance from occurring with the tower structural system. However, sufficient flexibility should be allowed in waveguide runs between towers and equipment buildings to avoid displacement-induced damage to the waveguide. Elements such as waveguides that might be damaged by falling objects should be protected with overhead metal plates. 63

NODE POINT PROTECTION

Communications facilities at node or terminal points include building/structure-equipment assemblies used to produce, disseminate, amplify, or switch voice or data signals. Seismic analysis and design techniques for buildings are now reasonably well understood and documented after several decades of research and test work conducted by universities, research organizations, architect engineering firms and industry. Reference 1 presents the latest and most comprehensive information on this subject.

Broad scale progress on equipment protection research and development has lagged behind that applied to buildings and structures. Data gathered from the great Alaskan earthquake of 1964, the San Fernando earthquake of 1971, and Japanese and other recent earthquakes throughout the world have shown that equipment failures form a large part of the destruction loss. Further, equipment failures have greatly delayed efforts to restore an area after an earthquake.

In communications facilities, the net worth of the equipment is often many times that of the building or structure in which it is housed. Most buildings, even if not specially designed and constructed to withstand earthquake effects, possess a measure of resistance by the very way they are constructed to withstand other loadings, such as wind. Except for special earthquake-resistant designs, equipment is generally designed only for gravity and handling loadings and in many cases is not capable of withstanding lateral seismic forces of any significance. A result of this may be collapse of equipment situated in buildings which themselves sustain only minimum seismic damage.

Preventing equipment damage during an earthquake is mainly a job of holding it in place during the shaking to keep it from sliding, overturning, or bouncing around and striking other equipment or building elements. When equipment frameworks are properly secured or restrained, the components within are in most cases capable of withstanding low-frequency seismic vibrations. There are, however, some exceptions, e.g., the air-supported disc memories used in electronic data processing systems.

Equipment situated on upper stories of multistory buildings will usually experience a more severe shaking than that situated on the ground floor. This is due to building amplifications that occur because the natural frequencies of tall buildings lie close to those of ground excitations (0.5 to 10 Hz), and resonant buildup occurs. Further resonant buildup may occur when the fundamental frequencies of the equipment also lie in this band. Table 1 indicates the difference between the ground and upper floor acceleration levels for the different seismic zones shown in the map in Figure 2. Notice that the amplification decreases as the zone earthquake level increases. This is because of the increased elastoplasticity and damping action of the building in severe environments.

Figure 3 shows floor response spectra prepared for use as design criteria for new Bell System central office communications facilities (Refs 2 and 5). The particular envelope spectrum shown in Figure 3 is used in designing lightly damped equipment situated on upper floors of buildings in Zone 4 and includes all necessary safety factors. Notice that the maximum 5g acceleration response limit indicated in Figure 3 will be experienced by equipment with fundamental frequencies between approximately 1 and 7 Hz.

Design Objectives

Design objectives to improve the mechanical resistance of communications equipment are noted below:

1. Strengthening at Building Connections. Many equipment failures that occur during actual earthquakes as well as those simulated in the laboratory are caused by inadequate strength of the connection used to attach the equipment to the building. Weldments, bolts, and inserts to concrete fail most frequently because of inadequate strength. Equipment supported by vibration isolators may shake loose and topple if snubbers or limit stops are not strong enough to resist the motions amplified through resonance with the isolator.

2. Lowering Center of Gravity. Lowering the center of gravity by placing heavy components closer to the bottom (or to the support points) reduces the bending moment induced by inertial response to the seismic motion.

3. Increasing Frequency. Increasing the natural frequency of the equipment by stiffening it or reducing weight generally decreases displacement and acceleration response through decoupling from a near-resonance condition with building and earthquake motions. Where possible, the resonant frequency of equipment components should exceed 10 to 20 Hz to avoid amplifications due to resonance. This is often difficult to achieve for tall, slender equipment bays (frameworks) used in communications systems because of size, weight, and space constraints. However, it is desirable to keep the fundamental frequencies of frameworks above 3 Hz to hold displacements to reasonable limits. In this respect, equipment design differs from building design. Designers of tall, slender buildings achieve a beneficial effect by designing for structural frequencies below those of earthquakes. Doing so minimizes acceleration forces which may act upon structures because they will behave as low-pass filters. For equipment, in contrast, the displacements associated with low-frequency design become intolerable. For example, a 1-foot relative displacement at the top of a 300-foot building may be acceptable, but similar motions at the top of a 7-foot equipment frame with a comparable natural frequency would be destructive.

4. Increasing Damping. Even small increases in damping diminish the amplification of motions caused by closely coupled equipment-building systems. An increase in damping from 2 to 5 percent may reduce equipment response by 30 percent in the critical resonance band between 1 and 7 Hz.

5. Reserve Ductility. One of the most desirable characteristics in any seismic-resistant component is its capability of absorbing energy after being stressed to the design limit. To yield and still carry load rather than suddenly fail through buckling or fracture provides a significant safety factor, allowing conservative and less costly designs.

6. Vertical Strength. Because equipment is designed to withstand gravity and handling forces in the vertical direction, it usually can also withstand seismic forces under 1g in this direction without exceeding the safety factor for conventional design. However, when equipment is braced between the floor and ceiling, vertical motions must be considered to accommodate the relative displacements resulting from out-of-phase response of the building floors. Also, when designing against horizontal motions, one cannot count on the stabilizing effect of gravity loads of the equipment because of unweighting due to vertical seismic forces. Likewise, added stresses caused by vertical earthquake motions combining with gravity-induced stresses must be considered because these stresses may act vertically upon horizontally deflected equipment (P · Δ effect).

7. Achieving Symmetry and Rigidity. Components with eccentrically loaded or irregularly framed structures experience unnecessary and sometimes excessive seismic stress caused by torsion or unevenly distributed response load. Symmetrical structures well balanced around the mass provide better support and may be less costly in the long run. Slender tube or channel-type structural elements such as those used on framework uprights may roll and buckle if not stabilized properly. Increased material thickness, stiffeners, and rigid end constraints minimize this problem.

Equipment Strengthening

Equipment strengthening to meet the above design objectives will be achieved through careful attention to even the smallest detail. Below are some typical protection concepts for communications hardware:

1. Batteries. Batteries for equipment operation or for standby power generally constitute the heaviest loading to be considered in communications centers. Batteries situated on metal frames or on shelves in metal cabinets must be secured to prevent them from pounding together or sliding and falling to the floor. The stands or cabinets must be securely braced and firmly attached to the building to prevent collapse of the entire assembly.

2. Cabinets. Equipment such as computers, rectifiers, and radio gear is packaged in cabinets constructed of sheetmetal sides with top and bottom held together by structural angle corner pieces. Much commercial cabinetry of this type does not incorporate diagonal braces to carry earthquake-induced shear loadings from the upper parts to the floor supports. When diagonal members are not present, the shearing forces must be carried by the metal skin of the cabinet, which in this case behaves as a diaphragm. This arrangement requires a good structural connection between the skin and the angle corner members. Some commercial cabinetry incorporates little or no connection of this type and, for this reason, should not be used in earthquake-prone areas unless specifically strengthened. Metal cabinets equipped with large swing open doors in one or more sides must have the remaining sides and top constructed to carry out not only shear but the resulting torsion and bending moments as well. The bottom of the cabinet must be sufficiently strong to transfer the shears and moments to the anchor bolts in the building floor.

3. Frame-Mounted Components. Equipment frames subject to the most severe earthquake excitations may respond with accelerations of up to about 5 g's with single-amplitude displacements of 6 inches or more at the frame top relative to the floor. The absolute displacement of the frame is usually much higher when the motion of the floor is added. Absolute horizontal single-amplitude motion may total 2 feet or more in major earthquakes depending upon the construction of the building. Further, vertical motions up to ±3/4g above gravity may act at the same time as the horizontal motions. Components supported by the equipment frames must resist such motions unless separations or other types of malfunctions are considered acceptable in that they may be repaired after the quake.

Care should be taken to provide solid support for heavy items such as transformers. Eccentricity in the connections should be minimized to prevent overstress of materials or fatigue failure of the fastener. Wire wraps, where used, should be secure. In the maximum seismic risk areas, heavy circuit packs should be equipped with restraining latches to prevent them from being dislodged from their connectors and ejected from the framework. Tests have shown that the static withdrawal force of typical current circuit packs ranges from about 2 to 15 pounds depending upon the type of connector and the lubricant used on the contacts. An average value of about 8 pounds is typical. The withdrawal forces under dynamic conditions are very close to the static withdrawal forces in the frequency response band of most frameworks. Therefore, without latches, a frame acceleration of 4 g's may dislodge circuit packs weighing more than 2 pounds if the static withdrawal force is 8 pounds.

4. Attachment Hardware Details. Strength designed into equipment is of no avail unless all of the critical components conform as intended to the earthquake loading standards. For example, a strong well-designed framework will not survive in an earthquake if the anchor bolts supporting it easily withdraw from the building. Bolts to the floor or other parts of a building require meticulous control in design and in installation. Frequently, expansion-type anchors are used to secure the bolts to the building masonry. Care must be taken in drilling holes for such devices to keep the holes from being oversized. Oversize holes result from using worn masonry drill bits and greatly reduce the carrying capacities of the anchor. Self-drilling anchor systems are usually more reliable in concrete because each device is equipped with an integral drill bit. Anchors placed in the building concrete during construction should be made from a material of toughness and some ductility--not cast iron, which may easily be snapped.

Since building floors may not be perfectly level, leveling shims are sometimes used under the metal frameworks or cabinets. Shims must extend fully to the vertical or side face of the equipment frame. When they do not, the motion may force the frame or cabinet base to bend around the shim, giving rise to a large sideway response and gross reductions in frame stiffness and response frequency.

Raised floors supporting equipment should be provided with sufficient lateral support and removable floor panels should be secured to the understructure when attached equipment loadings may be large enough to cause tipping.

Attaching equipment to steel bracing elements of the building is sometimes accomplished by using forgings or cold-bent steel fasteners such as J bolts. It is important that material for these devices be ductile, for example, mild steel, so that brittle failures are avoided during an earthquake.

Seismic Testing

Determining the true earthquake resistance of equipment through calculation alone frequently is relatively difficult because of the mechanical complexity, as well as because of the nonlinear and inelastic response of some individual components. On the other hand, testing on a hydraulic shaker

programmed to match the seismic motions at the equipment support points evaluates the design and allows modifications before field use.

A qualification test should be used to prove-in equipment. Synthesized time history waveforms can be prepared that closely resemble actual earthquake motions in a building and match the response spectrum of the various earthquake zones. Figure 4 shows a waveform (Ref 6) used to match the response spectrum of Figure 3. The amplitude of the waveform is scaled downward to match the floor accelerations shown in Table 1 for use in testing gear for installations in less severe seismic zones. Sine sweep vibration tests may also be used but should be limited to equipment that does not respond in a nonlinear or heavily damped manner.

When testing is performed it is preferable that the equipment or component and its supporting medium simulate as closely as possible that actually used in practice.

COST ASPECTS

Protecting communications networks from earthquake damage is desirable to minimize the loss of vital services and expensive plant. The costs incurred to implement the protective features may be estimated and related to costs associated with the potential loss of service and plant.

Earthquake protection is cost-effective when the total amount spent in protecting all installations is less than the dollar value of the singular or discrete service and plant losses. Figure 5 illustrates this. Shown are discrete losses incurred during each earthquake damage event related to the communications system. The amount of each discrete loss will increase with time because of inflation and because of greater plant exposure as a result of growth of the system. Likewise, the time spacing of discrete losses diminishes with time because there is a greater probability of damage by a given earthquake in a system with many elements than there is in one with fewer components.

The discrete losses may be averaged over time as shown in the figure and compared with the average cost of earthquake protection plus the reduced average losses of the protected system. The net cost advantage of the protected system is, of course, the difference between the loss and cost curves summed over the period of interest.

Earthquake protection in areas of significant earthquake occurrence is generally cost-effective in new construction since the price of such protection amounts to only a very small percentage of the value of the installation. However, retrofit strengthening of existing installations is very costly. Also, installations in marginal earthquake areas

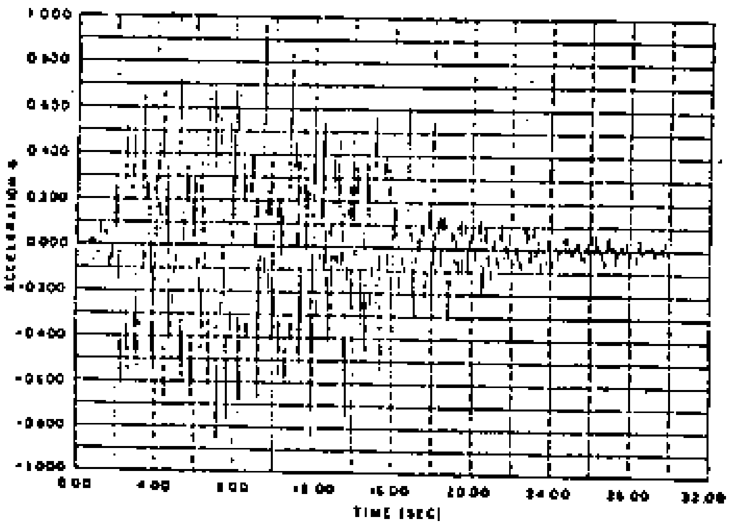

FIG. 4.-Analytically Developed Accelerogram

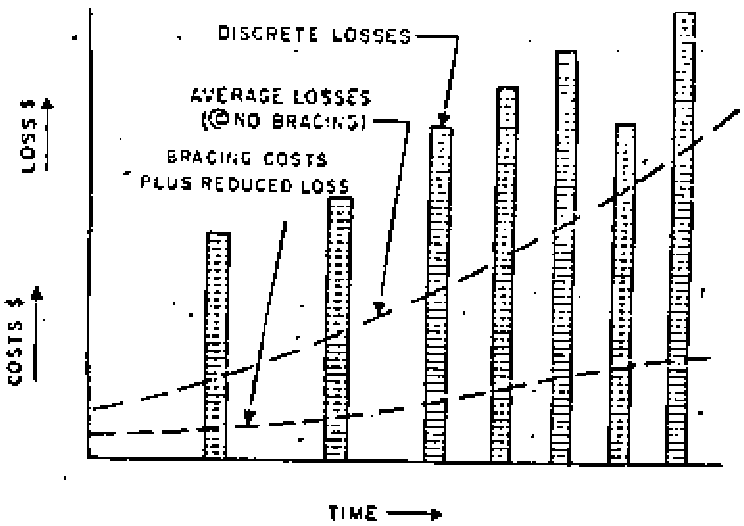

FIG. 5.-Bracing Cost and Loss Relationship

need to be evaluated from the cost standpoint to determine the degree to which such strengthening should be employed. In the latter case, the importance of the installation relative to that of the entire spectrum of communications must be considered.

SUMMARY

Communications distribution systems such as buried cable and pole lines are most sensitive to axial elongations caused by the ground motions of an earthquake. Proper siting and control of backfill together with sufficient cable slack and strength at splices, manholes, and buildings will help minimize damage. Survivability is further improved through avoidance of other structures or utilities which may fail and damage communications, for example, a gas line exploding in an adjacent trench or a bridge collapsing and destroying any cables it carries.

Building-housed equipment, if not properly braced during an earthquake, forms one of the most vulnerable segments of a communications system. The building floors that support equipment will undergo seismic motions with accelerations up to 1g in major earthquake zones. Responding to these motions, equipment whose vibration frequencies lie below 10 Hz may sustain accelerations up to about 5g's. Thus, the strength of the equipment and that of its fastening or bracing to the building must be equal to the forces induced by such accelerations.

Resistance of equipment to seismic shaking is enhanced by designing for high stiffness, frequency, damping, and ductility, and low center of gravity in a symmetric, torsion-resistant structural framework. Subcomponents must be securely attached or latched into place to prevent being dislodged from their supports. Damage to cabling, ducts, or piping caused by unequal or out-of-phase motions between adjacent pieces of equipment is prevented by minimizing or eliminating the relative response--as for example, through the use of tie-struts.

Earthquake protection of communications equipment is generally cost effective in major seismic risk areas. The feasibility of similar protection in low-risk or marginal earthquake areas may be determined from cost/loss analyses which consider the importance of the facility as well as plant expenditures. Applique bracing that is graded in strength to match the zonal and/or importance classifications may provide a low-cost option for such areas.

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PROTECTING A POWER LIFELINE AGAINST EARTHQUAKES[1]

by

Otto W. Steinhardt[2], M.ASCE

ABSTRACT

An electric power system is described, its relevance to the life of the community is examined, and its seismic withstand capability is evaluated. Recommendations are made to assure that this lifeline will be able to keep the vital functions of the community going in the face of seismic disaster.

INTRODUCTION

An electric power system consists of many diverse elements and covers much area. To evaluate its ability to withstand an earthquake requires that the system be analyzed as a whole. Failure of one element may constitute failure of the system, but perhaps the system can function well enough, at least temporarily, even if some of its parts have been knocked out of service.

When an earthquake strikes, the lights may dim, but in most parts of the city they won't go out -- at least not in a city where the power company has built and maintained its system with a reasonable standard of seismic resistance.

What is a "reasonable" standard? To answer that question, we first need to consider why electric power is important to a community in the aftermath of an earthquake, what a power system is made up of and which parts are necessary for providing uninterrupted power. Also we need to ask what earthquakes actually have done to power systems, what is the earthquake withstandability of a large power system, what additional capability can be provided in the future through replacements and new construction, and what recommendations should be implemented so that the response to the great earthquake when it happens will be all that anyone could reasonably hope for.

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[1]Prepared for presentation at ASCE TCLEE Specialty Conference, Los Angeles, California, August 30-31, 1977

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LIFELINE FUNCTIONS

Hospitals, pumping stations, telephone exchanges and certain other important facilities are equipped with emergency power generators, but those generators may not start when most needed. Also, there are many important facilities which are not so equipped. In some cases a system may be so spread out that it would not be practical to provide emergency power at every place where power would be needed. Cold-storage warehouses, walk-in refrigerators in supermarkets, gasoline pumps, and transit systems are just a few of the power-dependent facilities which are quickly felt by their absence when they stop functioning. Television and radio stations and newspaper printing plants are needed in disaster-stricken communities to keep up morale and help counteract the panic-inducing rumors which quickly start circulating. Home freezers contain much food which should be saved from spoiling if possible. Even in places which have emergency generators, the need for power is usually much greater than can be provided by the standby units in order for the facilities to function at anywhere near full capability. Clearly, it is important to the community that there should be no widespread power outage resulting from an earthquake.

DESCRIPTION OF POWER SYSTEM

An electric power system consists of generating stations, transmission lines and distribution lines. Besides these strictly electrical components, there are the backup systems: communications and power control, repair and maintenance service, warehouses, shops and fuel-handling facilities.

The generators are driven by turbines which are, in turn, driven by water, steam or hot gases. The steam may originate in a boiler heated by fossil fuels or in a nuclear reactor, or may come from a geothermal well. Gas turbines, which are actually modified jet aircraft engines, are used to drive generators, also.

Because bulk power is most efficiently transmitted at voltages much higher than practical levels at which it can be generated step-up transformers are provided at all generating stations. Then, when the power enters the distribution system, it must be stepped-down to levels which are efficiently usable by the customers. To maintain the predetermined voltage, autotransformers and voltage regulators are used. Capacitors are used to improve the power factor by keeping the three phases of a transmission circuit in good relationship to each other. To protect transformers against power surges resulting from short circuits, circuit breakers of various types are provided, and, for long term interruption of a circuit, isolating switches are used. To protect the system against voltage spikes resulting from lightning, lightning arresters are used; they resist the passage of current at normal voltages but allow transient peaks of overvoltage to go into the ground. Wave traps are used to enable transmission of supervisory signals through the power lines. Busses provide low-loss transmission linkage of the many and varied components within a switchyard or substation.

The minimum essentials to keep power flowing are the generating stations (although sometimes bulk power is available from outside sources), transmission lines, transformers, circuit breakers, and switches (although bypasses can sometimes be rigged). Bus structures must not collapse. The communications system is vital for controlling the flow of power and enabling quick restoration of service. The many other components of the system, valuable as they are in daily service, can be dispensed with, bypassed, or substituted for in an emergency. Efficiency will suffer and system capacity will be lowered, but power still will flow to those customers able to use it. Of course, in a great earthquake a large number of customers will be knocked out and so, temporarily at least, the demand for power will be down. It is hoped that the repair rate will exceed the rate of recovery of the demand for power.

EFFECTS OF PAST EARTHQUAKES

Subjecting a system to an earthquake is a good way to test its ability to withstand other earthquakes. So, let us take a look at what earthquakes have done to power systems.

1. When the great earthquake occurred in San Francisco and along the coastal zone north and south of there in 1906 (M=8.3), there wasn't a great deal of damage to electric systems because not much of a system existed. Power system damage came from a falling roof, falling chunks of brickwork and a broken smoke stack. Pole lines and underground ducts were not heavily damaged, except in locations where the soil failed. (1)

2. In the great Kanto (Japan) earthquake of 1923 (M=8.2), 23 out of 91 hydroelectric plants and all 11 steam-electric stations in and near Tokyo were damaged. There was much destruction of wood pole transmission lines and substations, mostly owing to the fire which followed the quake. Unreinforced masonry failures were responsible for damage of reservoirs, canals and penstock intakes. This, in turn, led to washouts and landsliding which damaged the penstocks and powerhouses. Except where the foundations settled, ground shaking caused no damage to generators. Transformers survived where they had been anchored. Circuit breakers were undamaged if they had flexible connections to the busses. Bus structures survived if the buildings to which they were attached did not collapse. Of 2400 transmission towers about 10% were damaged. Wherever a tower was a complete failure the cause was landsliding. Some were damaged because of foundation failure; the few structural failures which occurred were in lines which were "at right angles to the seismic waves." (2)

3. In Long Beach, California, in 1933 (M=6.3), transformers shifted, bushings broke, oil spilled from transformers and switches. Transmission towers were undamaged. All circuits were back in service in 5 minutes. In Compton, the substation was back in service in 47 minutes; however, several

distribution lines were down so full restoration of service took longer. (3)

4. The strongest earthquake to strike California since 1906 was the 1952 Kern County (Arvin-Techachapi) earthquake (M=7.7). It did enough damage to electrical equipment to cause the California electric power companies to take notice. Substation transformers rolled off their foundation rails unless they were strongly anchored. Pole transformers fell off of platforms except where they were bolted in place or otherwise securely fastened. Standard procedures for installing transformers were changed as a result of the type of damage observed in this earthquake. There were other kinds of damage: fan blades bent, pump bearings burned out, oil sloshed over the top of storage tank walls and in some cases wrecked the roof. Some transmission towers were disabled because of landslides or foundation failures. In many spans of transmission and distribution lines the conductors slapped against each other because of swinging, and some burned down as a consequence. A hydroelectric plant in Kern Canyon was damaged by falling rocks and dirt, but it continued operating at reduced capacity. (4), (5)

5. In 1964, Niigata, Japan, was hit by a strong (M=7.5) earthquake. Seven out of 8 substations were heavily damaged. A steam power station suffered damage to the condenser and cooling lines. Eleven hydroelectric stations out of 230 in the district were put out of service; 47% of the city's power was out for nearly five days. Portable transformers were helpful in getting service restored. Underground lines were damaged, probably because of soil liquefaction.

6. Alaska was struck by an even greater earthquake (M=8.4) in that same year (1964). At the Eklutna hydroelectric project near Anchorage, the operator on duty thought an atom bomb had exploded and had triggered a huge landslide. The plant was able to go back on line within 20 minutes, but the power had no place to go until temporary jumpers could be installed around wrecked circuit breakers. Intermittent rock and snow slides and reservoir cave-ins made operation of the plant a touch-and-go matter for the next six weeks. In Anchorage, an oil storage tank ruptured and gasoline supply lines broke, so the city's gas turbines and diesel generators couldn't operate. Heroic efforts by all hands, made possible a 90% restoration of power within 3 days. (7), (8)

7. In Chile in 1960 (M=8.5) and again in 1965 (M=6.4) circuit breakers, transformers, and capacitor banks were damaged in considerable numbers. This was the first time that high voltage substation equipment, other than unanchored transformers, had been seriously damaged in an earthquake. Until that time it seemed that there was no particular need to provide seismic resistance when designing such equipment. (9)

8. The San Fernando Valley earthquake of 1971 (M=6.6) dramatically demonstrated that seismic design of major equipment needed more attention than it had been given in the past. The quake tripped out 4 major steam units, although they were not damaged. A landslide wrecked a small, 50-year old hydroelectric plant. Ground shaking seriously damaged two switching stations, resulting in electrical isolation of 10 bulk power substations and many outages on the distribution system. Most customers who could receive power were reenergized within a few hours and nearly all within 5 days. Nevertheless, some severe damage occurred at locations near the epicenter of this "moderate" earthquake. At Sylmar Switching Station, all of the 230 KV air-blast circuit breakers were damaged badly; all 26 disconnect switches were damaged, especially in the bearings of the rotating columns; most of the 12 potential transformers were damaged; all of the wave traps broke off of their columns; several lightning arresters fell; some of the 230 KV rigid bus broke loose from supports; some transformer anchor welds broke.

Olive Switching Station also was heavily damaged, and the Sylmar high voltage DC Converter Station was 40% destroyed and required over 18 months to be restored to service. There was significant damage at several receiving stations (transmission terminal substations) and at five distribution substations, and less damage at about two dozen distribution substations and three receiving stations. About a mile of 34.5 KV underground cable, and a half mile of 5 KV, had to be replaced, along with six manholes and some potheads and trifurcators. (10), (11)

9. In the December 1972 Managua, Nicaragua, earthquake (M=6.5), the Managua Power Plant, 70MW, was less than a mile from one of the major fault traces. This plant was designed for 0.1g by static equivalent load method; it had to shut down for repair. Switchgear and batteries were damaged. A 40 MW and two 15 MW turbine-generator units were damaged because of hammering of turbine pedestals against the surrounding concrete floor. The smaller machines were repaired within 4 weeks but the 40 MW unit required replacement parts and technicians from Europe so repairs took about 4 months.

The plant had two 16,000 barrel oil tanks. The floor plate of one tank ruptured where a roof support column was welded to the floor. The other tank was undamaged. In the plant switchyard, rail-mounted transformers derailed and some bushings broke. Substations suffered transformer and insulator damage. Wood pole distribution lines were not damaged by ground shaking. By December 23, six days after the earthquake, electric service was almost fully restored to Managua. (12), (13)

10. Guatemala was struck by an earthquake, M=7.5, in February 1976. An electric power plant, 42.6MW, in Guatemala

was designed and built in 1958 by EBASCO, a US firm. There are 4 gas turbines and one oil-fired boiler unit. The plant was probably designed according to typical US design practice for seismic areas like California, i.e., using 0.2W (or less) static equivalent force. The power plant is 100 miles from the epicenter, but only 25 miles from the nearest part of the fault trace. The plant was tripped out by the earthquake and was out of service for 40 minutes but suffered no damage. The oil tanks apparently were undamaged as they are not mentioned in the report. (14)

ABILITY TO WITHSTAND EARTHQUAKES

Now that we know something about what earthquakes have done to power systems, what guesses can we make about what a future earthquake will do to a particular power system?

The particular system I have in mind is the Pacific Gas and Electric Company which serves 47 counties in northern and central California (Figure 1). It can generate over 14,000 megawatts (MW), of which 60% is in steam plants and 40% is hydroelectric capacity.

Because PG&E is in California, the threat posed by earthquake is accepted as a fact of life. There have been at least 10 strong earthquakes (Richter Magnitude over 6.0) in the service area in the last 60 years, and there have been at least three great earthquakes (RM over 8.0) in the last 200 years (San Francisco 1838 and 1906; Fort Tejon 1857). There are about a dozen active faults (Figure 2), of which the most threatening are the San Andreas, the Hayward, and the Calaveras Faults.

By far the greatest part of the PG&E system is of modern vintage. The four largest steam plants, with combined capacity equal to 45% of that of the total system are at Pittsburg, Contra Costa, Moss Landing and Morro Bay. Two-thirds of their capacity is in units of 300 MW or larger, housed in braced steel frameworks designed for 0.2W static equivalent lateral load. In their design close attention has been given to the details which often make the difference between what survives an earthquake and what doesn't.

Hydroelectric development on the McCloud, Pit, Kings, and Feather rivers, added 10% of system capacity, between 1952 and 1965. Also, nearly 4% of system capacity has been installed in the Geysers geothermal steam field since 1959, and a 63 MW nuclear unit was built at Humboldt Bay only 15 years ago.

In the next few years the system will be further enlarged by the addition of a large pumped storage hydroelectric plant and by the two large nuclear units at Diablo Canyon which will add 15% to system capacity. The Diablo Canyon plant is nearly complete but is not yet licensed to operate. Another nuclear plant, a coal-burning plant, and several gas-turbine units are planned for the future.

The transmission lines, mostly of modern construction, are widely

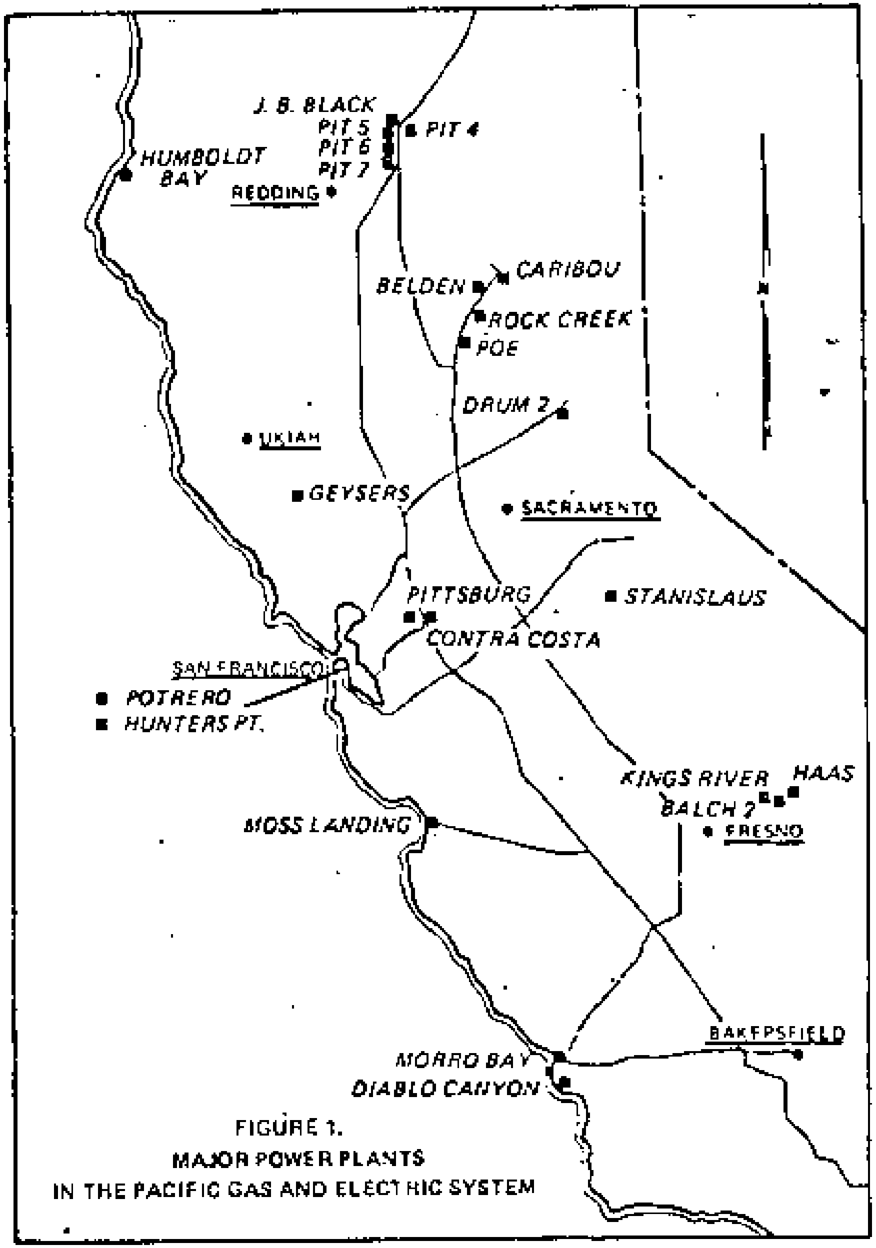

FIGURE 1.
MAJOR POWER PLANTS
IN THE PACIFIC GAS AND ELECTRIC SYSTEM

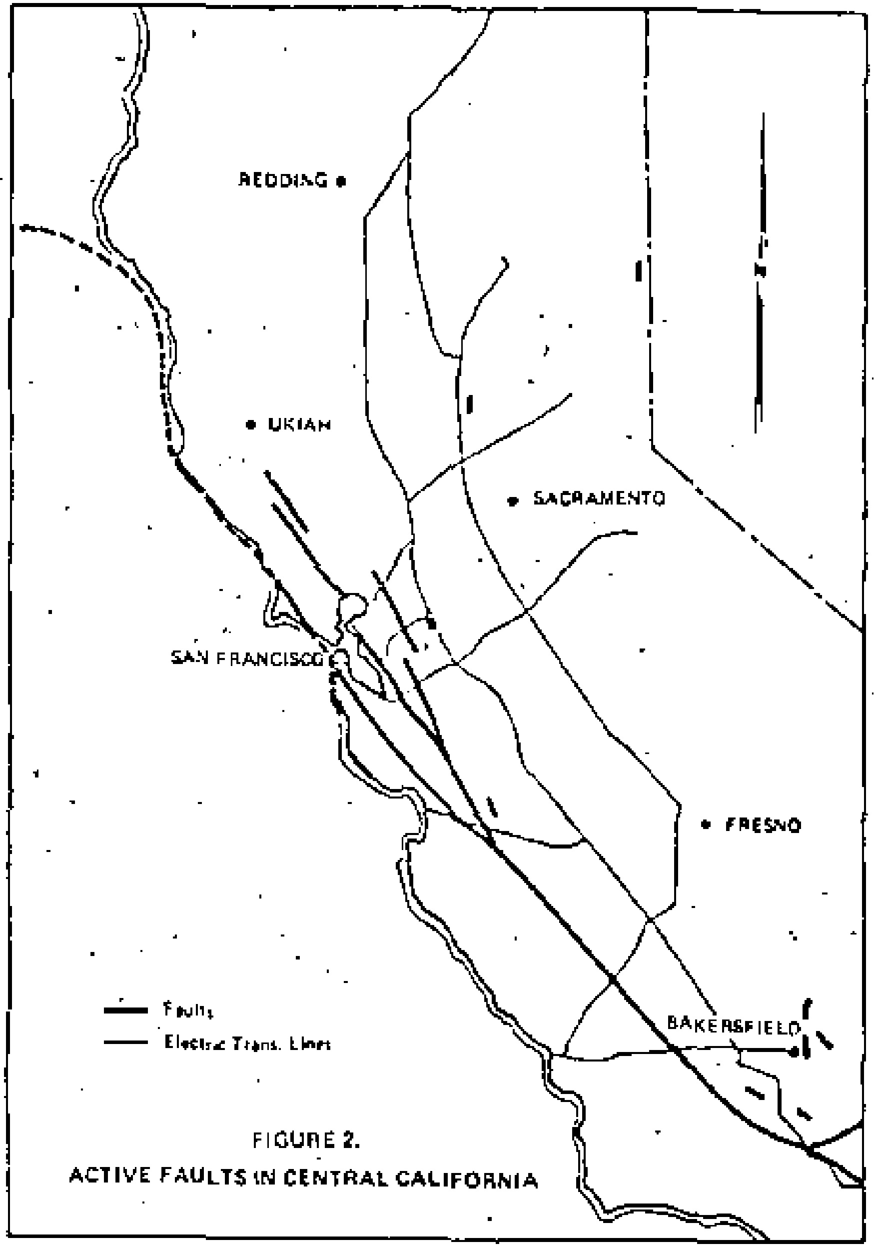

FIGURE 2.
ACTIVE FAULTS IN CENTRAL CALIFORNIA

dispersed. This factor lessens the danger of earthquake damage. Also, transmission lines have never suffered much from seismic shaking, because the towers are designed to survive heavy loads resulting from ice, wind and broken wires, so the seismic load doesn't have much influence on the design. However, a tower may be damaged by landslides, rolling boulders, or liquefaction of the supporting soil. Also, power lines sometimes swing so wildly when shaken that they slap against each other and burn down, or at least trip out the circuit. The hazards of landslides, etc., can be minimized by careful site selection and good foundation design. As for the problem of conductor slapping, wider spacing of conductors would be of some benefit but considerably more expensive. Inasmuch as this hazard seldom occurs, it's not justifiable to spend money to prevent it from happening.

Some transmission lines are underground. Fault slippage is not likely to damage such underground lines because of their thick-wall welded steel pipe jacket. Shaking has never been a source of damage to such lines except where it has led to failure of the earth masses in which the line is buried. Displacement of manholes from the conduits in which the cables are laid sometimes can cause the line to fail. In case an underground line is knocked out, repair usually takes a long time and an overhead line would probably be rigged as a temporary bypass.

Substations and switchyards are vulnerable to earthquake damage. Substation equipment installed before the San Fernando Valley earthquake (1971) was purchased under specifications which required that the equipment be able to withstand 0.2 W lateral load applied statically. After 1971, specifications changed to a higher level to meet the demonstrated need. Now, substation equipment which is necessary for continuity of power supply (the minimum essential system) is required to withstand maximum horizontal ground acceleration of at least 0.4g horizontal acceleration and 2/3 as much vertical using designated response spectra. Equipment which is not part of the minimum essential system is required to withstand at least 0.2 W lateral load, as before.

If critical equipment is to be supported on a structure, dynamic modeling of equipment and structure is included in the design. In case the equipment is unable to take the seismic loads under these conditions, modifications are made.

Many types of substation equipment have been dynamically evaluated in the last 5 or 6 years, either in-house or by the manufacturers. Some modifications of design have resulted, and in a few cases installed equipment has been back-fitted or replaced. In many cases the analysis has shown that all that's needed is to provide sufficient anchorage to the supports.

Seismic design criteria for distribution nets are not as conservative as for transmission facilities. It is expected that after a seismic event line repair crews, reinforced by electrical contractors, using on-hand materials from Company and other warehouses, will be able to restore service to damaged areas at least as quickly as the customers' demand for power recovers from the disruptive effects of the quake.

In San Francisco, an energy control center is housed in a low-rise building which was designed to withstand the effects of debris falling from the nearby high-rise buildings. The nonstructural partitions, the suspended ceiling, the operational map boards and other installed equipment have been subjected to a post-construction review to identify seismic vulnerability and a modification program is currently underway to correct the deficiencies which were found.

Besides keeping generating stations and transmission facilities from being knocked out we need to be sure that the fuel supply won't be disrupted.

The fossil fuel steam plants burn oil or natural gas. A two month supply of oil is stored at each such plant. In the last six years, twenty-four oil storage tanks, mostly the half million barrel size, have been installed. They were designed for 0.2 W lateral load and have been checked by dynamic analysis and found capable of sustaining 0.67g maximum ground acceleration without buckling or overtopping by sloshing.

The supply of natural gas comes from California (17%), from Texas (38%), and from Alberta (45%). The 78 billion cubic foot reserve is stored in what was a nearly-depleted gas field in central California.

The oil and gas pipelines cross active faults at several locations. These crossings have been plotted on maps of the system so that if a fault breaks the lines crossing it, the damaged places can quickly be found and inspected. In many instances there is little to worry about because the amount of fault slippage would probably be only a few inches. A welded steel pipeline properly installed in a sand filled trench can take many inches of fault offset. This was dramatically demonstrated after the Kern County earthquake (1952) when a 34-inch diameter high-pressure gas main was offset more than two feet with no significant damage. The pipe was later cut to relieve the locked-in stresses and the cut ends sprang out of alinement by about four inches.

The hydroelectric system depends on dams and penstocks for its "fuel." There are over 150 dams of various types and sizes in the PG&E system. Some are at least 100 years old, having been built by other groups, for other purposes, like gold-mining. About a dozen major dams were built after World War II and were designed for 0.1 W lateral loads. Some of the other major dams need attention. Because of the potential hazard to downstream populations, an on-going dam safety evaluation program is being conducted. It utilizes both pseudostatic and dynamic analysis methods to check seismic safety. The program has revealed some deficiencies which the Company has corrected. Thus far, a major dam has been replaced, a reservoir has been lowered, and a half-dozen major strengthening projects have been completed.

ABILITY TO RECOVER

It would be too much to expect a power system to come through an

earthquake unscathed. Economic considerations require that the defenses against seismic hazards be prudently high but when prevention of such possible damage becomes significantly expensive, some damage must be accepted. Therefore, it is essential to be ready to spring into action quickly and efficiently as soon as the shaking has stopped. An assessment of damage must be made, damaged circuits must be isolated and undamaged areas restored to service immediately. Available crews must be dispatched, reserve forces must be mobilized, material needs must be identified and supplies located. The rapidly changing situation must be kept under continuous observation so that personnel and materials can be redeployed if necessary. To accomplish all of this, a communications and control system is essential.

PG&E has a mobile radio system, with 250 base stations at service centers throughout the area controlling 3400 maintenance and supervisory vehicles. Power control signals can be transmitted directly over the power lines. Also available are microwave transmission stations and intra-company telephone lines.

Each of the 13 operating divisions of the Company has one or more switching centers, each of which is continuously staffed. The operators monitor the status of all transmission circuits and take immediate action on emergency situations as they develop. At the energy control center, the dispatcher coordinates actions which are outside the control of the switching center operators and reports to management any situation which cannot readily be taken care of.

The communications system is capable of operating on batteries and emergency generators for 72 hours. Communication racks and battery racks have been surveyed for resistance to earthquake loads and have been provided with bracing and anchorage sufficient for 0.5g horizontal and 0.30g vertical acceleration in all critical relay stations.

Substation and switching center storage battery racks are currently being surveyed to identify and strengthen those which need improvements to withstand possible seismic loads.

Line crews would not be able to accomplish much if their trucks were wrecked by the earthquake or if the gas tanks were empty. Most trucks are parked outdoors, so there is little danger of their being trapped by collapsed buildings. In several division areas, resupply of gasoline is carried out by tanker trucks with on-board pumps. The tanker goes to where the trucks are parked and fills the gas tanks. There is little danger, therefore, that the crews would be crippled by running out of gasoline just when they need it most. In all divisions, either the gasoline tanks are above ground or an emergency generator is available to run the pumps in case of power failure.

SUMMARY AND RECOMMENDATIONS

PG&E is not unique in the way in which it has addressed the earthquake problem. Probably all of the West Coast utility systems are similarly ready and able to cope with seismic events. However, there should be no relaxation of efforts to search out ways to improve the seismic withstandability of critical components of the system. Also, redundancy should be developed in all parts of the system so that alternate paths are available for routing of power in an earthquake disrupted system.

Utilities should exchange among themselves detailed reports of earthquake damage. Development of seismically resistant critical electrical equipment for substations and switchyards should be undertaken cooperatively.

Rapid restoration of electric service in an earthquake damaged area is achievable if foresight is applied.

ACKNOWLEDGMENTS

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ADVANCES IN MITIGATING SEISMIC EFFECTS ON POWER SYSTEMS

by

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ABSTRACT

The history of earthquake damage and facility specifications for electrical power systems is reviewed. A bibliography of power system damage reports is given. Current practices for mitigating earthquake effects in different parts of the country with high risk are discussed. Research to mitigate earthquake effects is reviewed and problem areas are discussed.

INTRODUCTION

This paper is one of a series written in conjunction with the Technical Council on Lifeline Earthquake Engineering to review the state of the art of lifeline earthquake engineering -- in this case, the electric power industry. The paper will review the history of earthquake damage suffered by power facilities, and concurrently, it will follow the evolution of seismic specifications for power equipment and facilities to address the lessons gained from damage experiences. Current engineering practice for mitigating earthquake effects in high and moderate seismic risk areas with both high and low awareness of the earthquake problem will be reviewed. Current research to mitigate earthquake effects is reviewed and problem areas are discussed. Finally, the paper gives conclusions and recommendations.

HISTORY OF EARTHQUAKE DAMAGE AND SEISMIC SPECIFICATIONS

This section will review reported earthquake damage to power systems and the attendant changes in design practice adopted by utilities to improve earthquake resistance of power system facilities.

Japan experienced its most damaging earthquake and post-earthquake fire in 1923. The earthquake and fire took 150,000 lives, destroying 500,000 dwellings. The earthquake magnitude is estimated at 8.2 and affected Tokyo and Yokohama. Seven different systems serviced the damage area, although two of the systems suffered most of the damage, which was extensive. The description of damage reported here will be rather detailed, since many of the failures observed in this earthquake will be reported in subsequent earthquakes. The material has been taken from a

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report [12] which probably represents the most complete effort to date to document earthquake damage to power systems. It is unfortunate that it did not serve as a model for future efforts. The paper which is published in English is "an outline from the original (published in Japanese) which was very extensive." This "outline" has some 51 pages of text and 66 pages of photographs. The coverage is broad, detailed, and concise. The paper was prepared by The Japanese Electrical Committee, consisting of 60 members representing the Institute of Electrical Engineers of Japan, the Joint Electro-Technical Committee, and the Electrical Association of Japan. The original paper was published on December 24, 1923. The English version was published in April, 1925, and at least one U.S. utility had a file copy by March 1927.

The material described below will, of necessity, be brief and fragmentary, as the source material is itself a concise summary. To give one an appreciation of the effort which went into the preparation of the original, one section dwelt on the possibility of the power system as a source of fire. In this study, some 36,000 houses (in unburned portions of the city) were inspected for fire damage and sources.

From an overall perspective, there was a total disruption of service with very limited service restoration after two days. It was felt that the disruption of electric power "greatly added to the chaos and turmoil at the time." Table 1 shows the growth of power demand after the earthquake. The practice of not providing system redundancy, particularly for transmission facilities, greatly affected the extent and duration of disruption.

TABLE 1. POST-EARTHQUAKE POWER DEMAND

Date	Power Demand (KW)
9/1/23	0
9/3/23	5
9/10/23	35,000
9/20/23	70,000
10/1/23	90,000
11/1/23	140,000
12/1/23	170,000
12/8/23	203,000
2/1/24	193,000
2/12/24	213,000

Maximum power consumption prior to the earthquake was 203,000 kilowatts

GENERATION

Steam and gas units were, for the most part, on reserve at the time of the earthquake, power being supplied by hydrogeneration. Steam and gas units required from one to several months to resume operation. Most units were on poor soils. Boilers had no serious problems except for leaks which developed at joints of tubes and drums from external structural failures. Thirteen boilers (21%) had severe damage, 26 (41%) had the sidewall bricks of the combustion chamber damaged, and 24 (38%) had no damage.

Steel pressure piping was undamaged, but cast iron water return pipes had cracked flanges. Prime movers were undamaged, although they were typically on good foundations.

Some hydroelectric generation systems required a month or more to recover; while others were totally destroyed in the most severely shaken area. Earth-filled dams experienced severe cracking, although there were no failures. None, however, were in the severely shaken area.

The transmission system experienced damage primarily from landslides and foundation failures. In some cases, cylindrical footings pulled up, legs failed if diagonal bracing did not start close to the footing, and poor workmanship, such as loose bolts, was observed. Concrete towers responded relatively poorly because their weight aggravated foundation problems. Wood towers experienced broken stays and also failed due to overload. Pin-type insulators faired worse than suspension-type insulators.

Poles on the distribution system -- primarily wood -- behaved well. Many conductors short-circuited because of inadequate separation. Unanchored pole transformers, which were common, fell. Service wires failed due to building damage.

Direct and conduit-enclosed buried cables (370 miles of transmission and 1025 miles of distribution) had twenty junction box failures and two shorts due to moisture seepage through failed casings. Brick and steel manholes and vaults were damaged. The damage to steel enclosures was caused by the fire. Reinforced concrete enclosures behaved well. Great difficulty was experienced in locating and repairing cable damage due to debris. Extensive damage was experienced at bridge crossings due to fire on wood, steel, and stone bridges. Outside of the fire area, there were no failures.

Substations and similar facilities associated with generating stations were most heavily damaged by falling buildings. Transformers, which were generally on wheels or unattached to pads, moved. Thirty-one units moved and developed leaks, broken bushings and broken control lines. Some foundations, which were often of poor quality, tilted, also causing units to shift. Oil circuit-breakers, disconnect switches, and control boards behaved well if they were in good structures and well-secured. Lightning arresters were extensively damaged. For some utilities, all lightning arresters of a given type failed, emphasizing the importance of design practice. Batteries, in general, (78%) were turned over, short-circuited, or completely destroyed. One station had them tied to the floor and experienced no damage.

Communication lines used to control the transmission system were carried on tra ssion towers or on adjacent, but separate, poles.

Tower-supported cables behaved better, but still experienced wrapping, and communications were, in general, disrupted. Public telephones which were operational and messengers provided communication to control the transmission system.

Within buildings, switchgear and accessories failed due to building collapse or improper mounting. Transformers broke power, oil, and water connections due to poor mounting. Batteries overturned and had shorted plates. Lights were heavily damaged (83% in one structure). Motors associated with various building systems failed due to poor mounting. Loss of power stopped all elevators and lack of emergency exits trapped passengers.

An extensive survey (36,000) of the electrical systems within houses indicated that existing interior wiring standards were adequate to avoid fires.

A summary of the various recommendations and conclusions of the report follows:

1. Siting with particular attention to soil conditions and foundations is vital to the earthquake resistance of facilities.

2. The improvement of the earthquake resistance of structures is vital.

3. There should be redundancy in transmission and communication lines between major facilities.

4. Earth-filled dams require further study.

5. Equipment must be adequately secured.

6. Adequate slack of electrical connections between equipment is required.

While present day facilities are physically much larger and the state-of-the-art for analysis and design has made significant advances since 1923, it is interesting to note that most of the recommendations voiced in this, the first comprehensive report of power system earthquake damage, have by and large, been reiterated, in part, after each damaging earthquake. The author does not know to what extent the recommendations in the report were effectively acted upon in Japan. It would appear that U.S. utilities did not respond to the report.

The Long Beach Earthquake of 1933 had a magnitude of 6.3. The reported power system damage was not too significant, consisting of the loss of two 220KV buses (460 broken pillar-type insulators, broken connections to transformers which shifted on their rails, shifted potential transformers, and overturned batteries). While the first building code with seismic requirements was initiated as a result of the Santa Barbara Earthquake of 1925, the Long Beach Earthquake stimulated their wider use in California. Prior to 1933 a .2g static lateral force was used in the design of some utility facilities. After 1933 the .2g s   fication was used more widely. Also, flexible bus was adopted for new high voltage facilities and then transformers were secured to their foundations. It was not until after 1945 that the .2g lateral force requirement was formally incorporated into equipment specifications. It should be noted that the decision to adopt seismic specifications rested with individual utilities.[3]

The Olympia Earthquake of 1949, with its epicentor located 15 miles from Olympia and 60 miles from Seattle, was of 7.1 magnitude. Damage was not significant, consisting of a 230KV transformer rolling and breaking insulators and control conduits; wrapping of wires on long spans caused delays in restoration in outlying areas; soil problems caused transformer faults; an insulation string broke and many units tripped; and many service lines were pulled from buildings. A brick stack on a standby unit was also damaged. The Bonneville Power Administration (BPA) experienced some damage and instituted a .2g seismic specification. Also, for new facilities transformers were secured to their foundations.[2]

The Arvin-Tehachapi earthquake of 1952 with a magnitude of 7.7 caused damage to generation, transmission and distribution facilities. A generator thrust bearing was damaged and a shaft bent but was corrected with a heat soak. Fifty-six cooling fans, four fuel tank roofs, and a boiler feed water pump were also damaged. Transmission facilities experienced two burnt down lines and three towers were lost due to slides. Several transformers shifted but there were no failures. Over 800 pole transformers fell to the ground although none which were secured to their platforms fell. It was felt that relaying of transmission lines prevented burn downs on distribution lines which were wrapped. The lines were difficult to clear. As a result of this earthquake, some California utilities reviewed their facilities and secured older equipment not covered by the practices adopted in 1933.[3,14]

The Niigata, Japan, earthquake of 1964 was of 7.5 magnitude and caused extensive power system damage and disruption of customer service. Eleven of 232 hydroelectric facilities were damaged as were condenser and cooling ducts at a thermoelectric plant. Seven of eight substations were severely damaged, requiring 5 days to restore, during which time approximately 50% of electrical service was disrupted. Portable transformers helped restore service. Underground facilities were extensively damaged and were difficult to repair.[4,13]

The 1964 Alaska earthquake with a magnitude of 8.4 caused damage to the inlet of a hydroelectric facility which maintained service but with numerous problems. Valdez lost the fuel supply and start up power for its diesel generators. Damaged circuit breakers were bypassed enabling the hydroelectric facility to provide service. Extensive damage, which was not reported in detail, was repaired so that service was 90% restored within three days.[7,15]

Power systems were damaged to different degrees by four South America earthquakes in Chile, 1960 and 1965, Nicaragua 1972 and Guatemala 1976. The now common damage of unsecured transformers and toppled battery racks was again observed but other types of damage were also observed. Three turbine-generating units were damaged. In one case damage was aggravated when the oil lubrication pump stopped due to the failure of

emergency batteries. Also of note was the failure of high voltage substation equipment from inherent inadequate strength.

The San Fernando earthquake of 1971 of magnitude 6.6 severely damaged power systems and other lifelines. While the magnitude of this earthquake would put it in a moderate class, there was very severe shaking in the area close to the epicenter. As this earthquake was extensively reported [5, 6, 11] the extensive damage will not be reviewed in detail. Because of the extensive damage to high voltage substation facilities, this earthquake triggered the most extensive evaluation of power system equipment done in the U.S. Some of the activities in the post San Fernando period are discussed below.

Caution must be exercised in attempting to summarize and identify patterns and trends from damage reports from earthquakes which span a forty-three year period, occurred in different parts of the world and for equipment and facilities designed in many different countries. The six recommendations from the 1923 Tokyo earthquake are not only applicable to the U.S. today but remain unused outside of California. Recent earthquakes have also indicated that high voltage transmission facilities are inherently more vulnerable to earthquake damage. This could also be said for large fuel storage tanks. Large fossil fuel generating facilities in the 1000+ megawatt class, particularly coal fired units, have not been subjected to strong earthquake motions to date. While the vulnerability of underground transmission lines may be no greater than overhead lines, which have proved to be quite earthquake resistant, when they are damaged the entire line is put out of service for an extended period.

POST-SAN FERNANDO DEVELOPMENTS

While the Santa Barbara and Long Beach earthquakes provided the motivation for the development of seismic requirements of building codes, the San Fernando earthquake has stimulated more governmental, professional and industrial activity to mitigate earthquake effects than any other U.S. earthquake. A significant part of this activity has been associated with lifelines. It would be impossible to summarize all of the developments since 1971 related to power systems in this paper, but a few of the developments will be traced and the present situation relative to earthquake mitigation will be reviewed.

Each of the utilities which experienced damage developed damage reports. Also the federal government funded through NSF a study of unprecedented magnitude to study the effects of the earthquake which included a modest effort related to power systems [11]. The utilities in the damaged area also started extensive analysis and test programs. Directed primarily at transmission equipment this work was done by utility personnel in conjunction with consultants or manufacturers at considerable expense to the utilities. The Bonneville Power Administration (BPA) also funded [ ] detailed studies of the DC converter station. While the Bonneville work has been published, many of the utility studies have not been released. There are several reasons for this situation. In some cases manufacturers have participated in the work or have provided detailed information about their equipment and consider the results as proprietary. In other cases the work was not done with the participation of the manufacturer but the utilities are reluctant to release the information, as to do so may adversely affect their accessibility to information from the manufacturer in the future.

Most of the major California utilities have had seismic risk maps made of their service area which are used to establish excitation levels in seismic specifications for equipment and facilities. The utilities have also developed relatively sophisticated seismic specifications for their equipment and facilities. While specifications differ for the different utilities, important equipment in high risk areas would require dynamic analysis or testing with inputs as high as .5g horizontal and .3g vertical. One difficulty encountered by the utilities is getting manufacturers to meet the specifications. In some cases smaller manufacturers do not have the technical resources to understand and perform the required tests or analysis. Even larger manufacturers who have the resources are unwilling to bid on equipment if all specifications must be satisfied. Also, for some equipment the meeting the specifications is technically or economically impractical. A complicating factor is that some equipment must be mounted on structures which may dominate the dynamic response of the equipment. Some manufacturers are not interested in designing the support structures and will not be responsible for the work of others. As the time from the San Francisco earthquake increases it would appear that some utilities are reluctant to provide the resources for the detailed dynamic analysis for new facilities.

An important aspect of the earthquake resistance of facilities is associated with installation details. The securing of battery racks and transformers to their foundations has now been uniformly adopted, although some utilities have done this since 1933. The connection of buses to equipment is treated in several different ways and somewhat unevenly among the utilities. This factor is becoming increasingly important since the newer tubular structures tend to be more flexible and have lower damping than the older lattice type structures.

California State Government has become more actively involved in earthquake related problems. The Seismic Safety Commission and The Energy Resources Conservation and Development Commission have been established. The latter is now reviewing the need for expanded earthquake specifications for fossil fuel power generating facilities. The California Water and Power Earthquake Engineer Forum has been established to improve communications on earthquake related developments between water and power utilities within the state. The forum meets a few times a year and consists primarily of top civil engineering representatives from each utility.

The practice to mitigate earthquake effects outside of California is markedly different. While BPA has done extensive studies of their DC converter station - a mirror image of which was severely damaged in

the San Fernando earthquake - and they do have seismic specifications requiring dynamic analysis for their important facilities, other utilities in the Northwest have not implemented the simple low cost measures such as securing transformers to their foundations or providing adequate slack in bus-equipment connections.

In other high seismic risk areas of the country (as defined by the Uniform Building Code) generating facilities have in some cases satisfied the .2g horizontal static equivalent load. Not surprisingly, transmission and distribution facilities in these areas have no seismic specifications.

CURRENT RESEARCH ACTIVITIES

The following discussion does not include work related to nuclear power generation facilities. As noted earlier, there was extensive research related to power systems being funded by numerous sources. Following the San Fernando earthquake of 1971 most of this activity dealt with the analysis of various power system components such as circuit breakers, switches, etc. At the present time the character of the research effort has shifted to looking at the system as a whole, major facilities such as fossil fuel power plants and underground facilities. The majority of current research is funded by the National Science Foundation (NSF), RANN, Earthquake Engineering Program. Appendix A contains a list of NSF projects related to electrical utilities giving the title, the principal investigator, and a brief abstract of research objectives. While there is extensive research activity on structures which might relate to power systems, this material is not included. Recently the California Energy Resources Conservation and Development Commission has initiated a modest effort related to power generation facilities.

Initial results from one of the above studies are now available. The major objective of the study was to develop a methodology for evaluating the response of an electrical power system in a metropolitan area to a major earthquake [8, 9, 10]. The methodology uses digital computer simulation in which the power system is represented in detail. That is, power sources, transmission lines, transformers, circuit breakers, switches, busses, etc. in the study area are included. For each of a series of hypothesized earthquakes the probability of failure for each piece of equipment in the study area is evaluated taking into account the location of the site relative to the earthquake fault, the soil conditions at the site, the dynamic properties of the equipment and its support structure where appropriate. The operating status of each piece of equipment is then determined so as to be consistent with its probability of failure. The system is then reconfigured to take advantage of its redundancy. Load flows are performed to determine the existence of overloaded equipment. The importance of the various restoration tasks is determined, and available crews are dispatched to restore the system. In this manner system performance, as measured by the extent and duration of service disruption, can be evaluated. In addition, effects on system performance can be evaluated for changes in repair strategies or changes in seismic specifications of equipment at specific sites or of a given class.

For the "typical" power system modeled, it was found that service could be restored from earthquakes of magnitude 7.0 in about two days while a magnitude 8.3 earthquake would require about a week. For a given magnitude, earthquake position alone of that part of the fault which released energy, can have significantly different effects on power system response. Results also indicate that not all equipment is equally important to maintaining system operation. For example, transformers may be more important than circuit breakers used with them. Secondly, the high degree of redundancy in power systems means that extensive damage can be sustained without system disruptions. For example, only about 60% of damaged equipment need be repaired to fully restore service after a major earthquake.

In developing the simulation it was necessary to get fragility data for the various pieces of equipment which constitute the power system. Not only is this information difficult to obtain, but attempts by utilities to determine the cost of increased earthquake resistance of equipment have been frustrated. For example, a request for bid for equipment wanted quotations for specifications of .2, .3, .4, and .5g. All bids had the same price.

CONCLUSIONS AND RECOMMENDATIONS

At the present time, major California electric utilities have institutionalized seismic specifications for equipment and facilities. While regional seismic risk maps are used and site conditions are sometimes considered, this should become standard practice for all facilities down through important substations. Dynamic analysis should also be performed on all high voltage equipment and structures so as to minimize deleterious interaction between equipment and support structures. Since most utilities use "standardized" structures which evolve slowly, in time the cost of dynamic analysis per structure would be relatively low.

One of the major impediments to improve seismic resistance of utilities outside of California is that no effective infrastructure exists for the utility industry to insure that minimum earthquake mitigation measures are adopted. There are no parallels for seismic resistance in the power industry such as the boiler code or elevator code which have been formulated primarily by the affected industries and have served the cause of public safety effectively. Trade organizations such as Edison Electric Institute or Electric Power Research Institute consider the earthquake problem as regional in character (exclusive of nuclear safety) and give it so low a priority as to exclude it from consideration. Likewise the Federal Power Commission, the National Reliability Councils, and the Energy Research and Development Administration consider the earthquake problem outside of their charters or of low priority. Thus there is a need for some organization at the national level to insure that at least minimal, cost-effective earthquake mitigation measures are adopted on a national basis.

Professional organizations such as the Institute of Electrical and Electronics Engineering have regional committees addressing some earthquake related problems through the development of seismic design guidelines.

The development of design guidelines for use in high seismic risk areas outside of California may be of limited value unless they are very specific as to things that are to be done. It has only been in the post-San Fernando Earthquake period that the California utilities have developed significant expertise on structure and equipment dynamic behavior. It is highly unlikely that Mid-Western and Eastern utilities will develop the expertise required to effecively utilize seismic design guidelines which only give general procedures. Numerous reasons can be cited for the utilities outside of the West Coast not adopting earthquake mitigation measures. These include the fact that there is no record of earthquake damage to power facilities in these areas; most utilities' experience with seismic requirements is related to nuclear generating facilities and this often has associated with it excessive bureaucratic red tape and high cost; and the uninitiated usually overestimate the cost of implementing earthquake mitigation measures. Guides should also provide information on the cost-effectiveness of recommended actions. With this information, engineering staffs will be in a better position to get the support from management required for implementation.

Where seismic specifications for equipment and facilities have been implemented, a uniform design excitation level, usually expressed as a fraction of the acceleration of gravity, is given for all equipment associated with a given function. Simulation studies show that the importance of different pieces of equipment is different. Thus, the level of seismic specifications should more closely correspond to the importance of equipment function. Closely related to this is the need for incremental costs associated with improved resistance.

Since the cost of many of the measures to reduce earthquake hazards on new installations is very low, it would appear that the failure to do so is due to a lack of awareness of the problem or a tendency to resist change in a large organization. The low initial cost and the difficulty in retrofitting facilities speak strongly for ensuring that new facilities have at least minimum earthquake resistance. Also, since the Northwest is aware of the seismic hazard and still has not acted, it would appear that code requirements will be necessary to achieve implementation outside of California.

Investigations since the San Fernando earthquake have confirmed that higher voltage transmission facilities are in general more vulnerable to earthquake damage. The importance of a growing 800KV transmission network and the introduction of 1100KV service in the Midwest suggests that the earthquake resistance of these facilities should be evaluated.

There is a need for better documentation and communication of earthquake induced damage. Substation damage is usually repaired quite rapidly because of the need to restore service. While damage statistics are valuable for the state-of-the-art to continue to progress, more details about the equipment and its installation must be known. Thus, information on damaged equipment including its age, manufacturer, method of mounting, details on interconnections, and type of failure should be documented. Procedures for obtaining this type of information are probably most important for California because it is the most seismically active area.

A potential problem associated with the communication of damage information is that it might be restricted because of litigation associated with equipment damage or disruption of service. Historically, this has not been a problem after disruptions due to natural disasters. However, the national attitude towards product liability and the general recourse to the courts in recent years hold the potential of denying access to information which is vital for improving the earthquake resistance of equipment and facilities. As the procedures for obtaining damage information are most important in California, the problem of litigation is also most critical there. Fortunately, there would appear to be an administrative way around the problem. California courts have held that safety studies conducted by the Department of Industrial Safety are confidential and thus cannot be subpoenaed for purposes of litigation. Thus, if damage reports can be so classified, access of vital information will be assured without jeopardizing anyone's legal position.

ACKNOWLEDGEMENTS

The support of the National Science Foundation is gratefully acknowledged.

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APPENDIX A: RESEARCH ACTIVITIES

National Science Foundation supported research projects relative to power systems as reported in NSF Summary of Awards.

1. Seismic Safety of Electronical Power Equipment; Anshel J. Schiff; Purdue University, School of Mechanical Engineering, West Lafayette, Indiana 47907; $193,800 for 24 months beginning June 15, 1973.

   A methodology will be developed to evaluate the response of power systems to major earthquakes. The methodology will enable the extent and duration of services disruption resulting from earthquakes to be determined. The effects on system performance of changes in equipment specifications, repair strategies and system configuration can be evaluated. Efficient methods for vibration testing of power equipment in the field will be developed.

2. Seismic Resistance of Fossil-Fuel Power Plants; John L. Bogdanoff, Purdue University, Lafayette, Indiana; $372,100 for 24 months beginning January 1, 1974.

   This project will concentrate upon the determination of the dynamic behavior of large fossil-fuel steam power generating plants when subjected to earthquake forces. The results of the research will be used to establish design guidelines and procedures for the principal components of a power plant for earthquake resistance. These guidelines and procedures will form the basis for the development of seismic code provisions and recommendations for the design and construction of fossil-fuel steam power generating plants.

   This research will study, as a start, the behavior of the principal components such as the furnace-boiler, steam and feedwater piping system, coal handling equipment and conveyor system, cooling towers, and stack.

3. Earthquake Response of Dams Including Hydrodynamic and Foundation Interaction; A.K. Chopra; University of California, Berkeley, California 94720; $89,400 for 24 months beginning August 30, 1974.

The research program will: (1) develop reliable and effective techniques for analysis of response of dams to earthquake motions including effects of hydrodynamic and foundation interaction; and (2) understanding the effects of interaction and the significance in the dynamic response of dams. The research will include studies on concrete gravity dams, arch dams, and earth dams.

4. Optimal Earthquake Design of Energy Production, Storage, and Distribution Systems; Alfred M. Freudenthal; The George Washington University, Washington, D.C. 20006; $140,400 for 24 months beginning February 1, 1976.

This project focuses on the elements of risk, cost, and loss associated with earthquakes as important design parameters in developing structural design methods. Specifically, the objectives of this project are to: 1) establish a new structural design concept on the basis of maintaining a proper balance between the cost of providing a protective measure and the expected cost of earthquake damage; 2) develop procedures of optimal design for ultimate seismic load carrying capacity and functional reliability of structures; 3) apply these procedures in the design of important industrial facilities; and 4) make the results of the study available to user groups, and prepare guidelines for making the optimal design and planning decision.

5. Analysis of the Seismic Stability of Earth Dams; H.B. Seed; University of California, Berkeley, California 94720; $173,500 for 24 months beginning February 15, 1976.

As part of this project, a study will be made of the significant differences between earth dams known to have performed well and dams known to have performed poorly during strong earthquake shaking to determine the factors responsible for the differences in behavior. Specific objectives of the project are to: 1) establish a data base concerning the field behavior of earth dams during earthquakes so that adequate and inadequate types of construction can be identified; 2) investigate the adequacy of dynamic analysis methods in predicting satisfactory and unsatisfactory performance of earth dams during earthquakes; and 3) develop simplified but rational procedures for evaluating the earthquake shaking. The results of this project will contribute significantly to both improved safety and economy in the design of these critical earth dam structures.

6. Vulnerability of Transportation and Water Systems to Seismic Hazards; Irving Oppenheim; Carnegie-Mellon University, Schenley Park, Pittsburgh, Pennsylvania 15213; $187,000 for 24 months beginning May 1, 1976.

This project focuses on the general transportation system and associated water systems in order to determine a lifeline model which can be used to measure the performance and the principal causes of failure or decreased performance in these systems. The model will recognize the effects of redundancies in the system, the geographical features of the total system, and will contain the principal seismic failure criteria.

7. Underground Lifelines in a Seismic Environment; M.L. Baron; Weidlinger Associates, 110 East 59th Street, New York, New York 10022; $407,430 for 24 months beginning June 1, 1976.

Research will concentrate on underground water distribution lifelines. The specific tasks include: 1) a survey of underground water lifelines; 2) the development of appropriate seismic input; 3) methodology development for modeling and analysis; 4) methodology application to real systems; and 5) risk and cost-benefit studies of lifeline systems. The results of the research will be presented in the form of design aids, guides, and specifications.

ASEISMIC DESIGN OF 500KV AIR CIRCUIT BREAKER WITH FRICTION DAMPERS

by Taro Shimogo[1] and Shigeru Fujimoto[2]

Abstract

Statistical properties of the response of a 500KV air circuit breaker under nonstationary seismic excitation are studied by using a simplified mathematical model of the top-heavy tall structure and it is shown that the analytical results are in fairly good agreement with the experimental results.

Notation

The following symbols are used in this paper:

a_i, b_i $(i=1,2,3)$ = x-, y-coordinates of the lower end position of each stay;
f = frequency;
g = acceleration of gravity;
K = stiffness of the support column (ratio of the restoring moment to the inclination angle);
k, k' = stiffnesses of the damper;
L = length of the support column;
L_0 = initial length of the stay;
$M_1 = m_1+m_2/3+m_3$ = mass related to the inertia;
$M_2 = m_1+m_2/2$ = mass related to the gravity;
m_1 = mass of the mounted body;
m_2 = mass of the support column;
m_3 = mass of the stay;
P_0 = frictional force of the damper;
R = radius of the circle, on which the lower ends of the stays are arranged;
t = time;
T_0 = initial tension of the stay;
$\ddot{u}$ = input acceleration in the horizontal direction;
x, y, z = coordinate system (the z-axis is vertical);
γ = angle of the input direction making with the x-axis;

1. Introduction

A 500KV air circuit breaker is constructed with a heavy interrupting chamber installed on the top of a long support column, the bottom of which is connected with a rigid foundation rack. Three stays are stretched from the interrupting chamber to the foundation rack to increase the aseismic performance and additionally friction dampers are provided for the stays to absorb the vibration energy as roughly illustrated in Fig.1. This kind of construction can be available not only for the circuit breaker but also for any top-heavy tall structures to increase the aseismic performance. The friction dampers such as ring springs are usually used in practice for these constructions, because the friction damper can be made compact and absorbs relatively big vibration energy. However, it is somewhat difficult to estimate the seismic response of these structures by an analytical method owing to the nonlinearity of the friction damper. Although the 500KV air circuit breaker is commonly designed to withstand an horizontal earthquake shock of 0.3G sinusoidal wave for three cycles at resonance frequency in Japanese electric power industry(1), it is an important matter for establishing an anti-earthquake design principle of the top-heavy tall structure equipped with the stays and friction dampers that the dynamic characteristics are examined under nonstationary random excitation corresponding to a horizontal earthquake input. In this paper the effects of structural parameters such as frictional force of the damper upon the statistical properties of the response are especially examined by using a simplified mathmatical model and these results are compared with the experimental results.

1) Professor of Mechanical Engineering, Keio University, Japan
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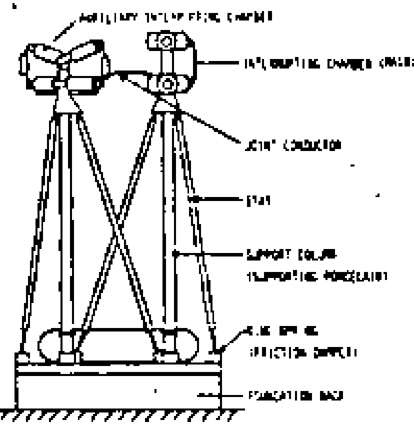

Fig. 1 The scheme of the 500KV ABM-type air circuit breaker (1/2 phase)

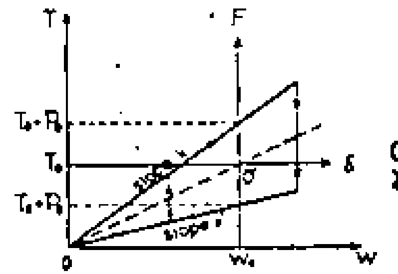

Fig. 2 The load-deflection diagram of the friction damper (the ring spring)

2. Mathematical Model of the Structure

A mathematical model of the structure is simplified under the following assumptions (see Fig.3):

(1) The coupling effects caused by the attachments such as cables, stays, rack and so on are not taken into account.

(2) The body mounted on the top of the support column such as the interrupting chamber is assumed to be a single mass point and the support column is assumed to be a uniform straight rigid bar, the bottom of which is replaced by a flexible joint. Accordingly the whole structure is expressed as a two-degrees-of-freedom system under the horizontal excitation.

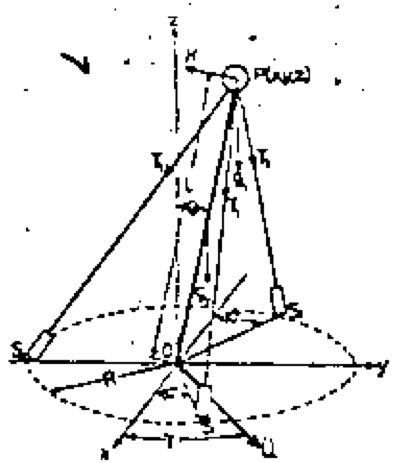

Fig. 3 The dynamic model of the circuit breaker structure

(3) The effects of the material and geometrical nonlinearities are neglected with the exception of the friction dampers. When the friction damper is constructed with a ring spring, the load-deflection characteristics of the damper is represented by a bilinear hysteresis as illustrated in Fig.2. In this case the frictional force P_0 at the equilibrium point is proportional to the initial tension T_0 of the stay ($P_0 = T_0(1-r)/(1+r)$, $r = k'/k < 1$).

From the above-mentioned assumptions the equations of motion are derived as follows(1):

$$\left.\begin{aligned} &d^2\xi/d\tau^2 + A\xi + C\eta + A_0 = -\ddot{u}_g \cos\gamma \\ &d^2\eta/d\tau^2 + C\xi + B\eta + B_0 = -\ddot{u}_g \sin\gamma \end{aligned}\right\} \quad (1)$$

where

$$\xi = x/L \ ; \quad \eta = y/L \ , \quad \tau = \omega_0 t \ \ (\omega_0 = \sqrt{g/L} \ ) \ , \quad \ddot{u}_g = \ddot{u}/g \qquad (2)$$

$$\left.\begin{aligned} &A = \kappa - \mu - \tau_0\lambda^3 \textstyle\sum_i \alpha_i^2 - \lambda^3 \sum_i \alpha_i^2 \pi_i + \lambda^2 \sum_i \alpha_i^2 \kappa_i \\ &B = \kappa - \mu - \tau_0\lambda^3 \textstyle\sum_i \beta_i^2 - \lambda^3 \sum_i \beta_i^2 \pi_i + \lambda^2 \sum_i \beta_i^2 \kappa_i \\ &C = -\tau_0\lambda^3 \textstyle\sum_i \alpha_i\beta_i - \lambda^3 \sum_i \alpha_i\beta_i\pi_i + \lambda^2 \sum_i \alpha_i\beta_i\kappa_i \\ &A_0 = -\tau_0 \lambda \textstyle\sum_i \alpha_i - \lambda \sum_i \alpha_i \pi_i \\ &B_0 = -\tau_0 \lambda \textstyle\sum_i \beta_i - \lambda \sum_i \beta_i \pi_i \end{aligned}\right\} \quad (3)$$

$$\left.\begin{aligned} &\kappa = K/M_1 g L \ , \quad \mu = M_2/M_1 \ , \quad \tau_0 = T_0/M_1 g \ , \quad \lambda = L/L_0 \\ &\alpha_i = a_i/L \ , \quad \beta_i = b_i/L \quad (i = 1, 2, 3) \end{aligned}\right\} \quad (4)$$

$$\left.\begin{aligned} \pi_i &= P_0/M_1 g \ , & \kappa_i &= kL/M_1 g & &\text{for } \alpha_i \tfrac{d\xi}{d\tau} + \beta_i \tfrac{d\eta}{d\tau} < 0 \\ &= 0 \ , & &= (k+k')L/(2M_1 g) & &= 0 \\ &= -P_0/M_1 g \ , & &= k'L/M_1 g & &> 0 \end{aligned}\right\} \quad (5)$$

When the input acceleration occurs in the y-direction ($\gamma = \pi/2$), the equation of motion is reduced to a simple form

$$d^2\eta/d\tau^2 + B\eta + B_0 = -\ddot{u}_g \qquad (6)$$

because the structure is symmetric about the y-z plane.

From the above equation the approximate value of the resonance circular frequency ω_n is estimated by using the mean values of frictional force and stiffness of the damper, that is, by putting $\pi_i = 0$, $\kappa_i = (k+k')L/(2M_1 g)$ as follows:

$$\begin{aligned} (\omega_n/\omega_0)^2 \fallingdotseq B &\fallingdotseq \kappa - \mu - \tau_0\lambda^3 \textstyle\sum_i \beta_i^2 + \lambda^2 \sum_i \beta_i^2 \kappa_m \\ &= \kappa - \mu - \tfrac{3}{2}\tau_0\lambda^3\rho^2 + \tfrac{3}{2}\lambda^2\rho^2\kappa_m = \Omega_n^2 \end{aligned} \qquad (7)$$

$$\text{where} \quad \rho = R/L \ , \quad \kappa_m = (k+k')L/(2M_1 g) \qquad (8)$$

3. Nonstationary Seismic Input

An approximate value of the mean squared response of a nonlinear system excited by a nonstationary random input is obtained by solving statistical moment equations which are introduced from the Fokker-Planck

equation. In this case the input is assumed to be a Gaussian white noise having nonstationary caracteristics, that is

$$\ddot{u}(t) = D(t)\zeta(t) \qquad (9)$$

where $D(t)$ expresses an envelope of the input acceleration and $\zeta(t)$ is a stationary random function with zero mean. Although $\zeta(t)$ has generally the dominant frequencies, $\zeta(t)$ is approximated to a Gaussian white noise in order to derive the Fokker-Planck equation. Then the power spectral density function of $\ddot{u}(t)$ is given by the expression

$$S_{\ddot{u}}(f,t) = S_\zeta(f)D^2(t), \quad -\infty \le f \le \infty \qquad (10)$$

where $S_\zeta(f) = 1$ [1/Hz] (power spectral density of $\zeta(t)$).

For example, the envelope of the El Centro seismic wave (N-S component, the maximum acceleration $\ddot{u}_{max} = 0.3g$), is obtained by a lag window and normalized so that the maximum value is unity as shown in Fig.4. The power spectral density of the stationary wave, which is obtained from the El Centro seismic wave devided by its normalized envelope function, is represented in Fig.5. On the other hand, the approximate value of the resonance frequency of the circuit breaker calculated by Eq.(7) is $\omega_n/\omega_0 \fallingdotseq \Omega_n = 7.54$ ($f_n/\omega_0 \fallingdotseq 1.20$), in the case when the dimensionless structural parameters are $\kappa=8.69$, $\mu=0.936$, $\tau_0=0.896$, $\lambda=0.96$, $\rho=0.287$, $\kappa_m=589$ as a typical numerical example. It may be assumed that the response of the circuit breaker mainly depends on the resonance frequency components of the input over the appropriate range, say, ±30% of the resonance frequency, if the damping ratio is approximated to 15%. This range is indicated with the shadowed portion in Fig.5. Therefore the input seismic wave may be replaced by an equivalent white noise whose power spectral

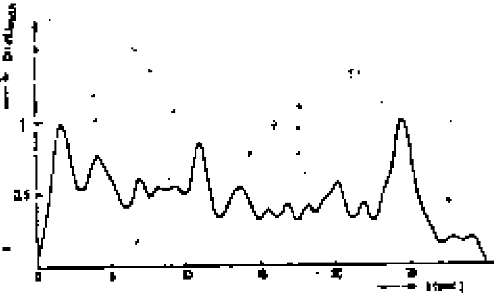

Fig. 4 The normalized envelope function of the El Centro seismic wave

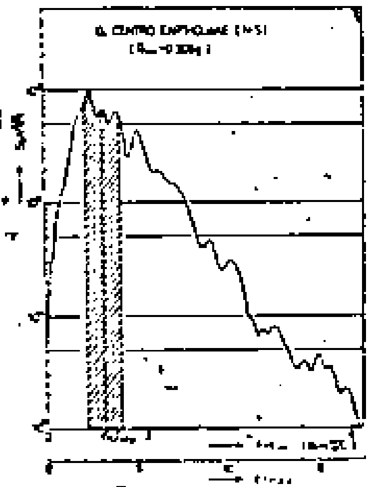

Fig. 5 The dimensionless power spectral density of the stationary wave corresponding to the El Centro seismic wave

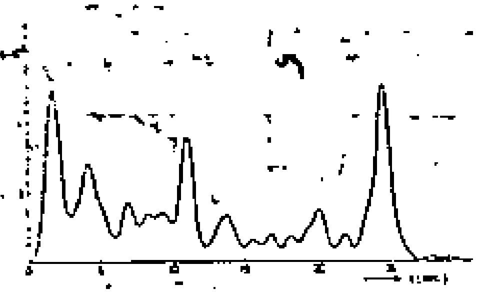

Fig. 6 The dimensionless power spectral density of the nonstationary white noise approximating to the El Centro seismic wave

density has the same value as the average level of the input power spectral density over the above-mentioned frequency range. In this example, the value of the dimensionless power spectral density of the equivalent white noise is about 5×10^{-3}. Since this value corresponds to the maximum value of the power spectral density of the nonstationary input, the time history of the dimensionless power spectral density $S_{\ddot{y}}(f,t)/\sqrt{g^3L} = S_{\ddot{y}}(t)/\sqrt{g^3L}$ is given as the squared value of the normalized envelope function multiplied by 5×10^{-3} (see Fig.6).

4. The r.m.s. value of the Acceleration Response

The Fokker-Planck equation governing the joint-probability density function of the responses is derived with the aid of the equation of motion, and further the equations relating to the second order moments of the responses are obtained from the Fokker-Planck equation on the assumption that the responses are Gaussian random processes with zero mean. When $\gamma = \pi/2$, the moment equations are reduced to [2]

$$dM_{11}/d\tau = 2M_{12}$$

$$dM_{12}/d\tau = M_{22} - \Omega_n^2 M_{22} - 2\sqrt{2/\pi}\,\rho\lambda\Pi_0 M_{12}/\sqrt{M_{22}} \qquad (11)$$

$$dM_{22}/d\tau = -2\Omega_n^2 M_{12} - 4\sqrt{2/\pi}\,\rho\lambda\Pi_0\sqrt{M_{22}} + S_{\ddot{y}}(t)/\sqrt{g^3L}$$

where $M_{ij} = E[\eta_i\eta_j]$ $(i,j = 1,2)$, $\eta_1 = \eta = y/L$, $\eta_2 = d\eta/d\tau$ (12)

$$\Pi_0 = P_0/M_1g = \tau_0(1-r)/(1+r), \quad r = k'/k < 1 \qquad (13)$$

The moment equations (11) are numerically solved by using the input power spectral density $S_{\ddot{y}}(t)/\sqrt{g^3L}$ indicated in Fig.6 and putting $\Pi_0 = 0.2125$ ($r = 0.616$), and the r.m.s. value of the acceleration response is obtained from the values of M_{ij} (see broken line in Fig.7).

The solid line in Fig.7 represents an experimental result. In the experimental study, a physical model of the circuit breaker was excited by a vibration testing machine under the El Centro seismic wave input and the r.m.s. response was investigated from the envelope of the squared value of the acceleration picked up at the top of the support column. A similarity of the dynamical properties between the physical model and the full-size structure of circuit breaker was taken into consideration by using the preceding mathematical model[1]. In this physical model, $L = 1.250$m ($\omega_0 = 2.8$rad/sec), $L_0 = 1.300$m, $B = 0.357$m, $K = 51.3$kgf-m, $M_1g = 1.07$kgf, $M_2g = 4.05$kgf, $T_0 = 4.23$ kgf, $P_0 = 1.00$kgf.

From Fig.7 it is seen that the approximate value of the response obtained from the moment equations are in good agreement with the experimental result with the exception of the peak values.

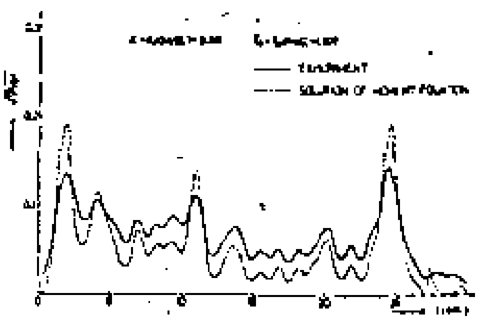

Fig. 7 The dimensionless r.m.s. values of the acceleration response (theoretical and experimental values)

The fact that the peak values of the theoretical result are overestimated may be originated in the variation of the dominant frequency of the El Centro seismic wave. The dominant frequency of the El Centro seismic wave becomes somewhat low and gets away from the resonance frequency of the structure at the time of the intense acceleration, thus the peak values of the response are relatively low in comparison with the theoretical result. However, if only the time history of the input power spectral density at the resonance frequency of the system can be estimated as the ensemble average of the actual seismic waves, then the analytical method described in this paper is available to estimate the r.m.s. value of the response.

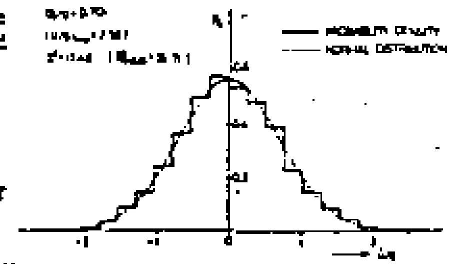

Fig. 8 The probability density of the input white noise for a digital simulation

5. Effect of the Nonlinearity of the Friction Dampers

The response of the structure was assumed to be a Gaussian process in preceding theoretical treatment notwithstanding a nonlinear problem. In order to examine how the probability density of the response deviates from the Gaussian distribution, a digital simulation is carried out by using a stationary Gaussian white noise as an input at the mathematical model. The level of this white noise is chosen to be equal to that of the equivalent white noise described in section 3, whose probability density is shown in Fig.8. The value of Chi-square for testing a fitness with the Gaussian distribution is also indicated in this figure. In the simulation, the characteristics of the friction damper is approximated to be parallel bilinear by putting $k = k' = (k+k')/2$ as a matter of convenience. The probability densities of the response obtained by the simulation are shown in Fig.9 and 10 for $\kappa = 6.69$, $\tau_0 = 0.896$ and $\tau_0 = 1.144$, respectively. From these figures it

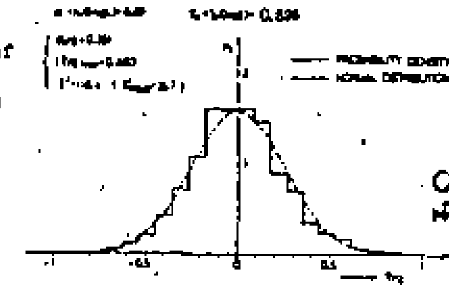

Fig. 9 The probability density of the response to the input white noise at the digital simulation (the case of lower initial tension at the stays or lower frictional force at the dampers)

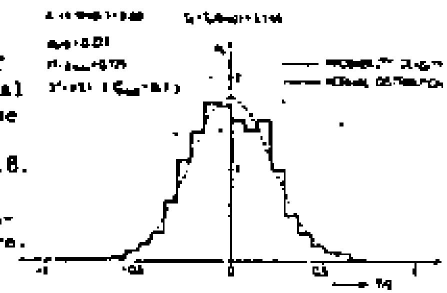

Fig. 10 The probability density of the response to the input white noise at the digital simulation (the case of higher initial tension at the stays or higher frictional force at the dampers)

is seen that the value of Chi-square increases and the fitness with the Gaussian distribution becomes poor, when the initial tension in the stays or the frictional force of the dampers becomes large. However, the shapes of the probability densities are not so largely different from the Gaussian distribution whithin the range of the initial tension or the frictional force at the circuit breaker and accordingly the error caused by the assumption of the Gaussian process in the preceding theory is not so big.

Although it is seen in these numerical examples that the r.m.s. value of the acceleration response decreases for the higher initial tension in the stays, the acceleration response may rather increase for the extremely high initial tension, because the relative displacement in the friction damper becomes small.

6. Conclusion

The optimal requirements of the parameters of the top-heavy tall structure, for example, the initial tension in the stays, the frictional force of the dampers, the stiffness of the support column, may be investigated by using the simplified mathematical model and the method of statistical approach described in this paper.

Acknowledgment

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UNDERGROUND PIPES

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IX CURSO INTERNACIONAL DE INGENIERIA SISMICA

DISEÑO SISMICO DE ESTRUCTURAS ESPECIALES

DISEÑO SISMICO DE PLANTAS INDUSTRIALES

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JULIO, 1983

CURSO INTERNACIONAL DE INGENIERIA SISMICA
DISEÑO SISMICO DE PLANTAS INDUSTRIALES

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Las presentes notas para diseño sísmico, pretenden establecer normas para definir el criterio de diseño sísmico a seguir, con el objeto de que las estructuras de plantas industriales tengan un comportamiento adecuado ante un sismo de mediana intensidad, después del cual la operación de la planta sea normal y que además para un sismo de gran intensidad no se produzca el colapso parcial o total de las estructuras y que los daños ocasionados puedan repararse en un periodo de tiempo relativamente corto.

Estas recomendaciones se aplican a la estructura completa y a todas sus partes, incluyendo la estructura en sí, pisos, muros y sistemas de techo, así como partes específicas de equipo y maquinaria de la planta.

NOTACION

Cada símbolo empleado en el presente capítulo se define donde se emplea por primera vez. Los más importantes son :

a	(adimensional)	=	coeficiente empleado en análisis dinámico modal.
B	(m)	=	base de un panel de vidrio.
C	(adimensional)	=	coeficiente basal (sin reducir por ductilidad).
c	(adimensional)	=	0.95 CD/Q
D	(adimensional)	=	Factor reductivo por flexibilidad.

H	(m)	=	altura de un panel de vidrio o espesor de estratos de suelo.
h	(m)	=	altura de un tablero de muro entre pisos consecutivos.
J	(adimensional)	=	factor de amortiguamiento.
L	(m)	=	longitud de un tablero de muro o mitad de la longitud de un tanque rectangular.
Q	(adimensional)	=	factor reductivo por ductilidad
T	(seg)	=	periodo natural de vibración.
V	(ton)	=	fuerza cortante horizontal en la base de la construcción.
W	(ton)	=	peso de la construcción.
y	(cm)	=	desplazamiento del centro de gravedad de la estructura descontando el que proviene de las deformaciones locales del terreno.
Ys	(cm)	=	desplazamiento del centro de gravedad de la estructura que se debe a deformaciones locales del terreno.

ZONAS

Para fines de diseño sísmico se considerará que las construcciones pueden desplantarse en las Zonas I ó II, atendiendo a la estratigrafía local del terreno.

Se consideran como pertenecientes a la zona I todos aquellos sitios donde exista evidencia de que no se encuentran a profundidades mayores de 20 m suelos con módulos de rigidez menores de 50,000 ton/m^2, o para los que el número de golpes por cada 30 cm. en la prueba de penetración estándar sea inferior a 50, y en que además se satisfaga la condición.

$$\sum H_i \sqrt{\gamma_i / G_i} < 0.45$$

En donde Hi es el espesor, en metros del i-ésimo estrato de suelo que se encuentra sobre el material con módulo de rigidez mayor o igual que - 50,000 ton/m^2. γ_i es su peso volumétrico en ton/m^3 y Gi es su módulo, de rigidez en ton/m^2. La suma deberá incluir los términos correspondientes a todas las capas que se encuentran sobre el material con módulo de rigidez mayor o igual que 50,000 ton/m^2.

Para fines de ésta clasificación, se tomarán en cuenta todos los suelos que se encuentren debajo del nivel en que las aceleraciones horizontales del terreno se transmiten a la construcción.

Se considerarán pertenecientes a la zona II aquellos sitios que no satisfagan los requisitos de los párrafos anteriores.

CLASIFICACION DE LAS CONSTRUCCIONES SEGUN SU IMPORTANCIA Y LAS CONSECUENCIAS QUE TENDRA SU FALLA

De acuerdo con este criterio las estructuras se clasifican en los siguientes grupos:

GRUPO A.- Estructuras muy importantes para el funcionamiento de la planta como son, la planta de fuerza, los soportes de tuberías, las chimeneas, los edificios de procesos, los soportes de reactores, etc.

GRUPO B.- Estructuras importantes para el funcionamiento de la planta que no queden comprendidas dentro del Grupo A, y todas aquellas estructuras cuya falla por movimientos sísmicos puedan poner en peligro otras construcciones de este grupo o del grupo A.

GRUPO C.- Estructuras que no intervienen en el proceso de la planta cuya falla no causa daños a construcciones de los dos primeros grupos.

GRUPO D.- Estructuras de poca importancia, cuya falla no causaría daños a construcciones de los tres primeros grupos. No necesitan diseñarse por sismo.

CLASIFICACION DE LAS CONSTRUCCIONES SEGUN SU ESTRUCTURACION

De acuerdo con su estructuración, las construcciones a que se refieren estas recomendaciones se clasifican en los siguientes TIPOS:

1. Edificios de cortante, incluyendo marcos de soporte de cubiertas para naves industriales.
2. Edificios de flexión.
3. Chimeneas y construcciones tipo torre.
4. Péndulos invertidos.
5. Tanques.
6. Muros de retención.
7. Otras estructuras.
8. Estructuras principales.

TIPO 1. Edificio de Cortante

Se considerarán como edificios de cortante, las construcciones cuyas deformaciones ante fuerzas laterales se deben esencialmente a las fuerzas cortantes entre pisos consecutivos. Incluye el presente tipo, por ejemplo, edificios cuya resistencia a fuerzas laterales es suministrada por muros cuando la relación de altura a base no pasa de 0.8, o por portales o marcos contraventeados o no cuya relación de altura a base no es mayor que 2.0 si las rigideces de sus vigas son del mismo orden que las de sus columnas.

TIPO 2. Edificios de Flexión

Se considerarán como edificios de flexión aquellos edificios cuyas deformaciones se deben en formar significativa a flexión de conjunto, como es el caso de las estructuras cuya resistencia a fuerzas laterales se debe a la acción combinada de marcos y muros esbeltos, o de marcos con crujías contraventeadas cuya acción sea semejante a la de muros esbeltos, de marcos con relación de altura a base mayor de 2 ó de marcos cuyas vigas son mucho menos rígidas que sus columnas.

TIPO 3. Chimeneas y Otras Construcciones Tipo Torre.

Se incluye en este tipo las construcciones cuya deformación ante fuerzas laterales sea esencialmente como la de una viga de flexión en voladizo.

TIPO 4. Péndulos Invertidos.

Se incluyen en este tipo las estructuras en que 50 por ciento o más de su masa se halla en el extremo superior y cuyo elemento de apoyo trabaja como una viga en voladizo.

TIPO 5. Tanques.

TIPO 6. Muros de Retención.

TIPO 7. Otras Estructuras.

TIPO 8. Estructuras Principales.

En este tipo se incluyen todas las construcciones del Grupo A. En general, tienen una estructuración especial que comprende uno o varios de los tipos ya mencionados y dado su trabajo específico y sus cargas de operación requieren de un estudio detallado.

METODOS DE ANALISIS SISMICO

El análisis sísmico podrá efectuarse empleando el método de análisis estático, o el método de análisis dinámico.

Se requerirá análisis dinámico en todas las estructuras en las que los efectos de modos superiores de vibración o la amplificación dinámica excesiva puedan afectar significativamente la respuesta de partes importantes de la construcción o de equipo costoso.

Deberán calcularse los efectos de las aceleraciones verticales y los de las aceleraciones horizontales para dos planos ortogonales, según los párrafos que siguen. Se revisará la seguridad de cada elemento estructural para la condición más desfavorable que resulte de considerar la acción de cada una de las componentes horizontal y vertical por separado o la combinación del efecto de cada componente horizontal con 0.7 veces el efecto de la componente vertical.

a) El efecto de las aceleraciones horizontales se tomará en cuenta suponiendo un sistema de fuerzas laterales obtenido de acuerdo con lo especificado más adelante.

b) El efecto de las aceleraciones verticales se considerará equiva lente a un sistema de fuerzas verticales (actuando hacia arriba o hacia abajo) obtenido multiplicando por 0.4 las cargas muertas y las vivas. Alternativamente, puede efectuarse un análisis dinámico que tome en cuenta los modos de vibración vertical de la estructura y que considere un espectro de diseño igual a 0.75 veces el correspondiente a aceleraciones horizontales.

COEFICIENTE BASAL

Se entiende por coeficiente basal, "C", el cociente de la fuerza cortante horizontal V, en la base de la estructura, sin reducir por ductilidad, y el peso W del mismo sobre dicho nivel.

El peso W, deberá incluir cargas muertas y cargas vivas. Los porcentajes de carga viva que deben incluirse en el análisis sísmico estan definidos adelante.

Para el análisis estático de construcciones clasificadas según las consecuencias de su falla en el Grupo "A" se tomará C igual a 0.78.

Tratándose de las construcciones clasificadas en el Grupo B, el valor de C se tomará igual a 0.60.

Para construcciones clasificadas dentro del Grupo "C" el valor de C, será igual a 0.48.

REDUCCION POR DUCTILIDAD

Para el cálculo de fuerzas internas en la estructura de producto CW se dividirá entre el factor Q que se especifica en los siguientes párrafos. El valor que adopta Q depende de la ductilidad de la estructura.

Para el cálculo de las deformaciones en la estructura no se hará reducción por ductilidad.

El factor Q podrá diferir en las dos direcciones ortogonales en que se analiza la estructura, según sea la clasificación y ductilidad de ésta en dichas direcciones.

A continuación, se presenta una relación de los valores del factor de ductilidad (Q) y los requisitos que debe llenar la estructura para poder adoptar éste valor en el diseño.

$Q = 5$

Este valor de Q se utilizará en estructuras del Tipo 1, cuya resistencia en todos los niveles sea suministrada exclusivamente por marcos continuos no contraventeados de concreto reforzado o de acero que tengan zona de fluencia definida y que cumplan con las siguientes condiciones.

a) Las vigas y columnas de acero, poseen secciones compactas según los requisitos del AISC.

Todas las juntas deberán admitir rotaciones importantes antes de fallar. Teniendo en cuenta lo anterior, el proporcionamiento y detalle de las juntas se hará de acuerdo con la parte 2 de la "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings" de la última edición del AISC. Estas especificaciones, deberán adaptarse para tomar en cuenta que se puede producir una inversión de momentos.

Además, todas las conexiones de los miembros que inciden a una junta, se diseñarán para tener una resistencia de 1.2 veces la capacidad del miembro incidente.

b) No se permitirá la formación de articulaciones plásticas en las zonas donde el área efectiva se reduce, (por ejemplo, por agujeros para remaches) a menos que la relación entre la resistencia última y la resistencia de diseño sea mayor que 1.5.

c) En la determinación de la longitud efectiva, que interviene en el cálculo de la relación de esbeltez de las columnas, se debe ignorar la ayuda proporcionada por contraventeos, ya que estos deberán ser diseñados para fallar ante la presencia de un sismo de gran magnitud.

d) En estructuras de concreto, el marco llenará los requisitos que para marcos dúctiles especiales fija el ACI 318 - 71 en su apéndice A y tendrá (por lo menos en la planta baja) columnas de concreto zunchadas.

e) Los factores de seguridad contra: falla en compresión por flexo compresión de columnas de concreto reforzado con estribos; fuerza cortante y torsión, en miembros de concreto reforzado, así como compresión axial y pandeo en todos los miembros, son cuando menos 1.3 veces los que resulten en flexión y en tensión para resistir fuerzas laterales.

f) En todo entrepiso, la estructura debe ser capaz de resistir 0.9 veces las acciones de diseño bajo la condición más desfavorable que resulte de considerar que la capacidad crítica de cualquiera de los miembros de dicho entrepiso se reduce a 0.5 de su resistencia de diseño.

g) El factor de seguridad para fuerza cortante de entrepisos, debe ser mayor en todos los niveles que 0.8 del promedio de dichos factores de seguridad.

h) La estructuración no sufrirá cambios repentinos en los distintos niveles y en las distintas crujías.

i) Se podrán usar contraventeos, con el fin de reducir las deflexiones siempre y cuando estos no se consideren como elementos resistentes y se diseñen para que fallen o se desconecten ante la acción de un sismo de gran magnitud.

<u>Q = 4.</u>

Se considerará un factor de ductilidad Q=4 para estructuras Tipo I, cuya resistencia en todos los niveles sea suministrada exclusivamente por marcos de concreto, madera o acero con o sin zonas de fluencia definidas, sean estos contraventeos o no, si se cumplen las siguientes condiciones:

a) Deberá cumplir con los incisos (a), (b), (e), (f) y (h) del art.

b) El factor de seguridad para fuerza cortante de entrepiso, es mayor en todos los niveles que 0.65 del promedio de dichos factores de seguridad.

c) La capacidad del marco para resistir fuerzas horizontales sin contar con la contribución de los contraventeos sea cuando menos 25% del total.

d) Los elementos de concreto deberán diseñarse, de acuerdo con los requisitos para estructuras en zona sísmica del ACI 318 - 71 (Apéndice A).

<u>Q = 3.</u>

Estructuras tipo I que reunan los requisitos del artículo 7.4 y cuya altura sea mayor que 3 veces la dimensión de la base. Para valores de h/b comprendidos entre 2 y 3 el valor de I, se obtendrá interpolando linealmente entre Q = 3 y Q = 4 donde h es la altura de la estructura y b la dimensión de la base.

Q = 2.

Se usará un coeficiente de ductilidad Q = 2 en estructuras de los Tipos 1, 2 y 3, cuya resistencia a fuerzas laterales sea suministrada exclusivamente por marcos o columnas de concreto reforzado, madera o acero o por muros de concreto o mampostería de piezas macizas.

Q = 1.50.

El valor del coeficiente de ductilidad Q será igual a 1.5 para estructuras del tipo 4 cuya resistencia a fuerzas laterales sea suministrada por una columna o hilera de columnas de concreto reforzado, madera o acero. Si la estructura se analiza dinámicamente puede considerarse Q = 2.

También se usará Q = 1.5 en estructuras de los tipos 1 a 4, cuya resistencia o fuerzas laterales en todos los niveles sea suministrada por elementos que se describen el el párrafo 7.6 y al menos en un nivel por muros de mampostería de piezas huecas.

Q = 1.

Para estructuras de cualquier tipo, cuya resistencia a fuerzas laterales sea suministrada al menos parcialmente por elementos hechos de materiales que no sean los arriba indicados, se usará un coeficiente de ductilidad Q = 1.

Para estructuras principales Tipo B, el valor de Q, se proporciona adelante.

Cuando las deformaciones locales del suelo contribuyan significativamente a los desplazamientos de la estructura los valores de Q que se especifican en los párrafos precedentes serán sustituidos por una nueva Q_m dada por: $Q_m = (Qy + 2y_s) / (y + y_s)$, donde y es el desplazamiento en cm. del centro de gravedad de la estructura, calculado sin tener en cuenta las deformaciones locales del terreno; y_s, en cm., es la parte del desplazamiento del centro de gravedad de la estructura que se debe a las deformaciones locales del terreno, y Q se especifica en los párrafos que anteceden. Para valores de Q iguales o menores que 2 el valor de Q_m se tomará igual a Q.

Las recomendaciones que anteceden, corresponden a estructuras con relación fuerza-deformación sensiblemente elastoplástica y para los cuales no se realice un estudio de ductilidades. En otras condiciones, se calculará Q según otros lineamientos.

Cuando la estructuración propuesta sea susceptible de tomar diferentes valores de Q se adoptará aquel valor de Q que proporcione la solución más económica.

CRITERIOS DE ANALISIS

En el análisis sísmico de toda estructura se supondrá que de manera independiente actúan los movimientos en dada una de dos direcciones horizontales ortogonales. Se verificará que la estructura es capaz de resistir cada una de estas condiciones por separado. Las estructuras de planta irregular o estructuras que son aproximadamente cuadradas en planta pueden requerir análisis en otra dirección. Además, en miembros que son más débiles en direcciones oblicuas que según los ejes de análisis, se revisará la resistencia en aquellas direcciones.

El análisis de los efectos debidos a cada componente del movimiento del terreno debe satisfacer los siguientes requisitos:

a) La influencia de fuerzas laterales se analizará tomando en cuenta los desplazamientos horizontales y verticales y los giros de todos los elementos integrantes de la estructura, así como la continuidad y rigidez de los mismos. En particular se considerarán los efectos de la inercia rotacional en los péndulos invertidos.

b) En cada elemento se tomarán en cuenta todas las deformaciones que afecten seriamente los desplazamientos y esfuerzos de diseño. También se tomarán en cuenta las deformaciones locales del terreno y las debidas a las fuerzas gravitacionales que actúan en la estructura deformada cuando estas tengan efectos significativos en la respuesta.

c) En estructuras metálicas revestidas de concreto reforzado, será factible considerar la acción combinada de estos materiales en el cálculo de esfuerzos de rigideces, debiéndose asegurar el trabajo combinado de las acciones compuestas.

d) Se supondrá que no obran tensiones entre la subestructura y el terreno, debiéndose satisfacer el equilibrio de las fuerzas y momentos totales calculados. Se revisará la seguridad contra los estados límite de la cimentación. Si existen elementos, tales como pilotes o pilas, capaces de tomar tensiones, se les prestará atención en el análisis.

e) El cortante en cualquier plano horizontal, deberá distribuirse entre los elementos resistentes proporcionalmente a su rigidez, considerando la rigidez del sistema de piso, diafragma o contraventeo horizontal.

Se verificará que las deformaciones de los sistemas estructurales, incluyendo las de las losas de piso, sean compatibles entre sí. Se revisará que todos los elementos estructurales, incluso las losas y los arriostramientos de los sistemas de piso o cubierta, sean capaces de resistir los esfuerzos inducidos por las fuerzas sísmicas.

Como simplificación en el diseño sísmico de construcciones de altura menor o igual que dos pisos o 6m., con sistemas de piso o cubierta arriostrados mediante sistemas cuya rigidez en su plano sea pequeña en comparación con la rigidez de los elementos que proporcionan la resistencia lateral, podrá considerarse que cada uno de estos elementos resistentes se ve sometido a la parte de fuerza sísmica que corresponde a su área tributaria por sismo en cada nivel.

f) En el diseño de marcos que contengan tableros de mampostería se supondrá que las fuerzas cortantes que obran en éstos están equilibrados por fuerzas axiales y cortantes en los miembros que constituyen el marco.

Asimismo, se revisará que las esquinas del marco sean capaces de resistir los esfuerzos causados por los empujes que sobre ellas ejercen los tableros.

ANALISIS ESTATICO

Para calcular lasfuerzas cortantes de diseño o diferentes niveles de una estructura, se supondrá los dos siguientes estados de carga actuando simultaneamente.

a) Un conjunto de fuerzas horizontales, actuando sobre cada uno de los puntos donde se supongan concentradas las masas de la estructura. Cada una de estas fuerzas se tomará igual al producto del peso de la masa correspondiente por un coeficiente que varia linealmente, desde cero en el desplante de la estructura (o desde el nivel a partir del cual sus deformaciones pueden considerarse despreciables) hasta un máximo en el extremo superior de la misma, de modo que la relación V/W en la base sea igual a 0.95 CDJ/Q, en donde C y Q son los coeficientes definidos, D es un factor reductivo que depende de la flexibilidad de la estructura y que vale 0.6/T para construcciones en la zona I, y 1.2/T para construcciones en la zona II, donde T es el período natural de vibración de la estructura en seg., calculado según se indica más adelante en este artículo. El valor de D no debe tomarse menor que 0.4, ni mayor que 1.0.

J es un factor reductivo que depende del amortiguamiento de la estructura, y que adquiere los siguientes valores:

J = 0.8; Para estructuras de acero remachadas o atornilladas, así como para estructuras de madera.

J = 0.9; Para estructuras de concreto reforzado o presforzado.

J = 1.0; Para estructuras de acero soldadas o con juntas a base de tornillos de alta resistencia trabajando a fricción.

La fuerza horizontal aplicada en el nivel i estará dada por la siguiente expresión:

$$F_i = 0.85\,\frac{CDJ}{Q}\,W \times \frac{W_i\ H_i}{\sum (W_1 H_1 + W_2 H_2 + W_3 H_3 + \ldots + W_n H_n)}$$

F_i = Fuerza horizontal en el centro de la masa de peso W_i y altura H_i sobre el nivel de la base de la estructura.

H_i = Altura sobre el nivel de la base del centro de la masa considerada.

$W = W_1 + W_2 + W_3 + \ldots + W_n$ = Peso total de la estructura.

W_i = Peso de la masa i

n = Número total de masas de la estructura

C y Q definidos en los artículos 6. y 7.

D y J definidos en este artículo.

El cálculo del periodo natural de vibración (T) de las estructuras de la planta que se utilizará en el cálculo del valor de D, se podrá efectuar utilizando la siguiente expresión:

$$T = 6.28\, d \left(\frac{1}{g} \quad \sum W_i y_i^2 \;\; / \sum P_i y_i \right) 1/2$$

donde

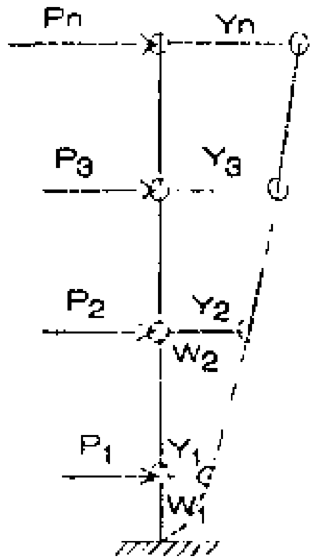
W_i = peso de nivel i

y_i = Desplazamiento horizontal en nivel i

P_i = Fuerza aplicada en el nivel i, proporcional a F_i

d = Coeficiente para tomar en cuenta las variaciones en el cálculo del periodo natural (d = 0.75)

En la figura siguiente se muestra esquemáticamente el significado de las variables que intervienen en el cálculo de T.



b) Una fuerza actuando horizontalmente concentrada en el extremo superior de la estructura, sin incluir tanques, apéndices u otros elementos cuya estructuración difiere radicalmente del resto de la construcción, igual a 0.05V.

La estabilidad de tanques que se hallen sobre las estructuras, así como la de todo otro elemento cuya estructuración difiera radicalmente de la del resto de la construcción no menor que el doble de la que resulte de aplicar la especificación anterior ni menor que la gravedad multiplicada por C/2. Se incluyen en este requisito los parapetos, pretiles, anuncios, ornamentos, ventanales, muros, revestimientos y cu anclaje y otros apéndices.

Se incluyen asimismo, los elementos sujetos a esfuerzos que dependen principalmente de su propia aceleración (no de la fuerza cortante ni del incremento de volteo), como las losas que transmiten fuerzas de inercia de las masas que soportan.

Para fines de diseño se tomará el momento de volteo calculado por cada marco o grupo de elementos resistentes, en el nivel que se analiza, igual al producto de la fuerza cortante que allí obra por su distancia al centro de las masas ubicadas arriba de dicho nivel.

La excentricidad torsional calculada en cada nivel se tomará como la distancia entre el centro de torsión del nivel correspondiente y la posición de la fuerza cortante en dicho nivel.

La excentricidad de diseño se tomará como se describe a continuación:

a) 1.5 veces el valor calculado más 0.05 veces la máxima dimensión del piso que se analiza (excentricidad accidental), medida en la dirección normal a la fuerza cortante ; para el diseño de miembros estructurales en que los efectos de torsión calculada sean aditivos a los de fuerza cortante directa.

b) El valor calculado de la excentricidad menos la excentricidad accidental, para el diseño de los miembros estructurales en que los efectos de torsión calculada y de cortante directo difieran en signo.

Además en ningún caso se tomará la excentricidad de diseño menor que la mitad de la máxima excentricidad de diseño de los niveles que se hallan abajo del que se analiza, ni se tomará la torsión de diseño de entrepiso menor que la mitad de la máxima torsión de diseño calculada para los entrepisos que se hallan arriba del que se analiza.

NOTA: Lo mencionado anteriormente se aplica cuando se garantiza la transmisión de la fuerza cortante sísmica entre marcos adyacentes por medio de sistemas de piso rígidos, contraventeos horizontales, u otros sistemas.

ANALISIS DINAMICO

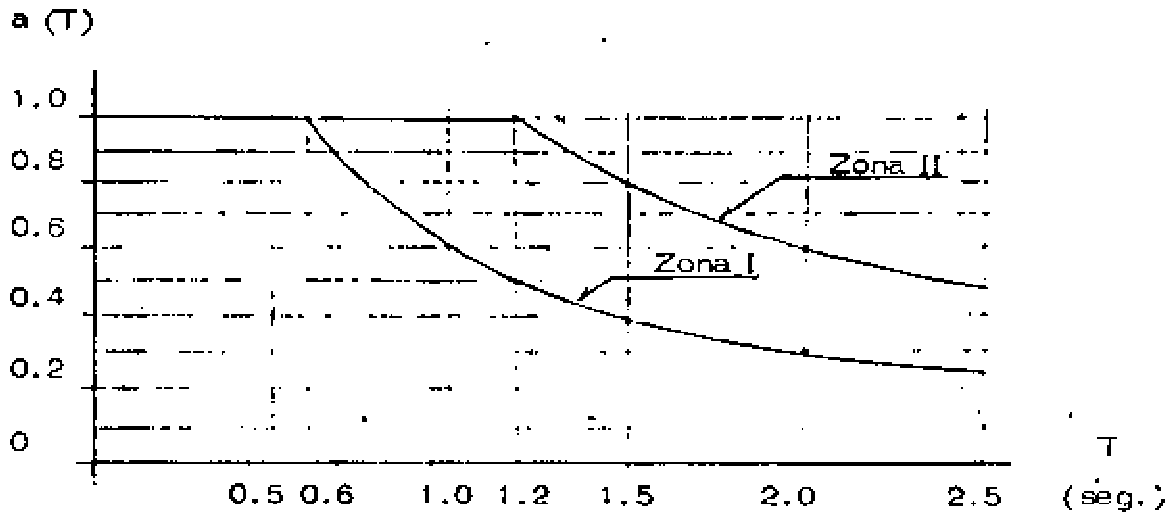
Son admisibles como métodos de análisis dinámico el análisis modal y el cálculo paso a paso de respuestas a temblores específicos.

Si usa el análisis modal, podrán despreciarse aquellos modos naturales de vibración cuyo efecto combinado no modifique los esfuerzos de diseño sísmico en más de 10 por ciento. Puede también despreciarse el efecto dinámico torsional que resulte de excentricidades, calculadas estáticamente, no mayores de 5 por ciento de la dimensión del piso, medida en la misma dirección que la excentricidad. El efecto de dichas excentricidades y de la excentricidad accidental se calculará como lo especifica el artículo correspondiente del análisis estático.

Cuando sea aplicable el análisis dinámico modal, este se llevará a cabo de acuerdo con las siguientes hipótesis:

a) La estructura se comporta elásticamente.

b) Tratándose de edificios ordinarios, el espectro de aceleraciones para diseño sísmico, expresado como fracción de la gravedad, es igual a a(T) C, donde C es el coeficiente basal T es el periodo natural de interés y a (T) está dada por las siguientes expresiones, en las que T está en segundos:



Zona I

a (T) = 1, si T < 0.6 seg.

a (T) = 0.6/T, si T ≥ 0.6 seg.

Zona II

a (T) = 1, si T < 1.2 seg.

$a(T) = 1.2/T$, si $T \geq 1.2$ seg.

En cualquiera de las zonas (I ó II) la aceleración espectral está dada por la expresión siguiente: $A(T) = a(T)\,Cg$ donde g es la aceleración de la gravedad.

Las fuerzas y esfuerzos calculados con los espectros citados arriba deberán dividirse entre el valor de Q aplicable.

Se supondrá que cada periodo natural de vibración puede ser inferior al calculado hasta en 25 por ciento y se adoptará el valor más desfavorable.

c) Las aceleraciones espectrales especificadas se deberán multiplicar por el coeficiente de amortiguamiento J, definidos anteriormente.

Si se emplea el método de cálculo paso a paso de respuestas a temblores específicos podrá acudirse a registros de temblores reales o de movimientos simulados o a combinaciones de éstos siempre que se usen no menos de cuatro movimientos representativos, independientes entre sí, cuyas intensidades sean compatibles con los demás criterios que consigna al presente reglamento y que se tengan en cuenta el comportamiento no lineal de la estructura y las incertidumbres que haya en cuanto a sus parámetros.

CALCULO Y LIMITACION DE DESPLAZAMIENTOS HORIZONTALES

Se deberán revisar los desplazamientos horizontales de la estructura y de partes y equipo que lo ameriten, debidos a las fuerzas producidas por un sismo de intensidad media.

Los desplazamientos se calcularán suponiendo que sobre la estructura obra una fuerza cortante total, V, igual a 0.3 D JW, si ésta es del grupo A, de 0.23 D JW si es del grupo B y de 0.18 D JW si es del grupo C. Esta fuerza se considerará actuando sobre la estructura con la distribución que se obtiene de aplicar los criterios de los artículos 9 ó 10.

Cuando haya peligro de colisión entre estructuras o partes de la misma, debidas a desplazamientos horizontales relativos, así como cuando se requiera revisar la estabilidad del conjunto ante un sismo de gran intensidad, las fuerzas cortantes totales, V, que se considerarán, serán iguales a 0.78 D JW, 0.60 D JW y 0.48 D JW para estructuras de los grupos A, B y C respectivamente. La fuerza se distribuye igual que en ..(1)

Las deformaciones laterales relativas entre entrepisos o entre niveles de sujeción de acabados o de piezas de equipo se limitarán de acuerdo con lo que se requiera para evitar daños en dichos elementos. Los límites impuestos a deformaciones en cuestión, deberán ser aprobados por el director del proyecto. La limitación puede omitirse cuando los elementos que no forman parte integrantes de la estructura estén ligados a ella en tal forma que no sufran daños por las deformaciones de ésta. En este caso no será necesario limitar los desplazamientos laterales sísmicos salvo para evitar choques entre estructuras contiguas.

En el cálculo de los desplazamientos se tomará en cuenta la rigidez de todo elemento que forme parte integrante de la estructura.

PRECAUCIONES EN VENTANAS

En fachadas tanto interiores como exteriores, los vidrios de ventanas se colocarán en los marcos de éstas dejando en todo el derredor de cada tablero una holgura por lo menos igual a la mitad del desplazamiento horizontal relativo entre sus extremos, calculado a partir de la deformación por cortante de entrepiso y dividido entre 1 + H/B, donde B es la base y H la altura del tablero de vidrio de que se trate. Podrá omitirse esta precaución cuando los marcos de las ventanas estén ligados a la estructura de tal manera que las deformaciones de ésta no les afecten.

PRECAUCIONES CONTRA CHOQUES ENTRE ESTRUCTURAS ADYACENTES

Las estructuras adyacentes deben separarse entre sí un mínimo de 5 cm., pero no menos que la suma de los valores absolutos de los desplazamientos máximos calculados para ambas construcciones, ni que 0.008 de la altura de la construcción más baja.

Estas separaciones pueden reducirse si se toman precauciones especiales para evitar daños por choques.

MUROS DE RETENCION

Los empujes que los rellenos ejercen sobre muros de retención debido a la acción de los sísmos se valuarán suponiendo que el muro y la cuña de falla crítica se encuentran en equilibrio límite bajo la acción de las fuerzas debidas a carga vertical, a una aceleración vertical igual a 0.3 Cg (hacia arriba o hacia abajo) y a una aceleración horizontal igual a 0.5 Cg, siendo C el coeficiente del Art. 6 y g la aceleración de la gravedad.

A partir de los empujes determinados mediante lo que se especifica arriba, deberán incluirse los siguientes conceptos en el diseño sísmico de todo muro de retención:

a) Diseño estructural del muro.

b) Seguridad contra volteo. Incluyendo los efectos de empujes estáticos y de sismo, el factor de seguridad contra volteo, calculado como el cociente de los momentos con respecto al centro potencial de volteo de las fuerzas que tienden a estabilizar el muro entre aquellos que tienden a voltearlo, debe ser cuando menos igual a 1.2.

c) Seguridad contra deslizamiento. Incluyendo los efectos de empujes estáticos y sísmo, el factor de seguridad contra deslizamiento, calculado como el cociente de la suma de aquellas fuerzas que tienden a impedir el deslizamiento sobre una superficie crítica entre aquellas que tienden a producirlo, debe ser cuando menos a 1.2.

OTRAS ESTRUCTURAS

El análisis y el diseño de estructuras que no puedan clasificarse en alguno de los tipos descritos se harán de manera congruente con lo que marcan las presentes especificaciones, para los tipos aquí tratados.

VALUACION DE LA RESISTENCIA ESTRUCTURAL

La valuación de los factores de seguridad y de la capacidad de miembros estructurales de concreto, acero y mampostería, se efectuará según se especifica, respectivamente, en el reglamento vigente del Instituto Americano del Concreto ACI (poniendo especial atención a su apéndice A) y en las especificaciones vigentes para diseño estructural del Instituto Americano de la Construcción en Acero AISC.

Cuando se diseñe para los efectos combinados de sismo y carga vertical se seguirán los lineamientos de dichos reglamentos, utilizando los valores que se dan a continuación en substitución de los valores que al respecto especifican los reglamentos antes mencionados.

a) En diseño por esfuerzos de trabajo el incremento de esfuerzos permisibles será de 30% para concreto, 50% para acero de refuerzo y 50% para acero estructural.

b) En diseño por resistencia última se usará un factor de carga de 1.1.

Por lo que respecta a mampostería se usará el capítulo correspondiente del reglamento de las construcciones vigente del Distrito Federal u otro código especializado.

No deberá considerarse la acción simultánea de viento y sismo.

VALORES DE LOS COEFICIENTES SISMICOS

Las construcciones del grupo A, así como aquellas del grupo B deberán ser capaces de resistir:

a) Un sismo de mediana intensidad, después del cual la operación de la planta no debe interrumpirse. (Sismo de operación).

b) Un sismo de gran intensidad en el cual no deberá producirse el colapso de la estructura y en el que los daños sufridos puedan repararse en un periodo de tiempo relativamente corto. (Sismo de diseño).

Las construcciones de los grupos B y C, se diseñarán unicamente para resistir el sismo de diseño según se indica en el párrafo b precedente.

A modo indicativo, la tabla siguiente proporciona los valores del coeficiente basal para construcciones de los grupos B y C.

La tabla es aplicable para estructuras Grupo "B". Para construccione grupo "C" los valores de la tabla deberán multiplicarse por 0.8 y por las de grupo A por 1.3.

Para análisis estático deberá usarse el factor reductivo "D" que toma en cuen a la flexibilidad de la estructura que se definió anteriormente.

REDUCCIONES DE CARGA VIVA

Para el cálculo de las fuerzas sísmicas el valor del peso "W", deberá incluir las cargas muertas (cargas que actúan permanentemente sobre la estructura), más un porcentaje de las cargas vivas utilizadas en el diseño por fuerza vertical. El valor de este porcentaje se da en la tabla siguiente:

DESTINO DEL PISO	POR CIENTO DE LA CARGA VERTICAL
Oficinas, habitaciones, pasillos	20%
Areas de almacenamiento	50%
Techos con pendientes mayores de 5%	20%
Techos con pendientes menores de 5%	40%
Contenidos de Tolvas de almacenamiento tuberías y tanques	100%

COMBINACIONES DE CARGA

Las estructuras se analizarán para las siguientes combinaciones de carga:

1) E.L. (debido a D.L. + L.L.1 + L.L.2A + C.L.) + D.L. + L.L.2 + C.L. + T.L.

2) E.L. (debido a D.L. + L.L.1) + D.L. + L.L.1 + T.L.

3) E.L.H. (debido a D.L. + L.L.1 + L.L.2A + C.L.) + 0.7E.L.V. (debido a D.L. + L.L.1 + L.L.2A + C.L.) + D.L. + L.L.1 + L.L.2. + C.L. + T.L.

En cada caso, se tomarán en cuenta únicamente las cargas que procedan y con el sentido más desfavorable.

E.L. = Efectos de la fuerza sísmica horizontal ó vertical calculados según los requerimientos de este reglamento.

E.L.H. = Efectos de la fuerza sísmica horizontal únicamente.

E.L.V. = Efectos de la fuerza sísmica vertical únicamente.

D.L. = Carga muerta, incluye el peso propio de todos los elementos estructurales de acero, concreto, tabique, etc. También incluye el peso propio de todos los elementos de la Planta que están ligados a la estructura (cables, tuberías, etc.).

T.L. = Efectos debidos a temperatura.

L.L.1 = Incluye el contenido de tuberías, tanques, tovas y otros recipientes.

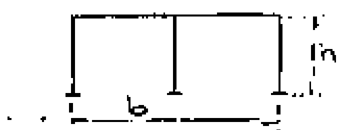
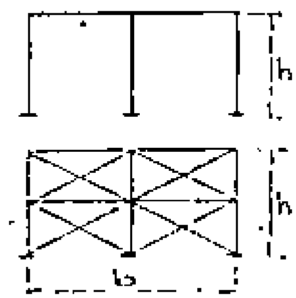
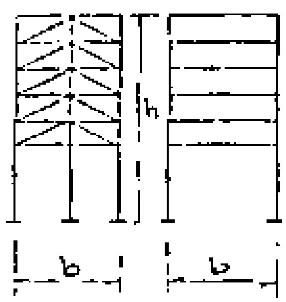
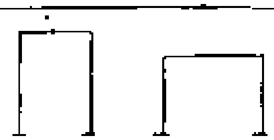
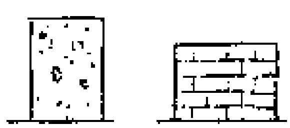
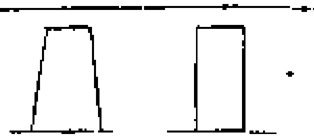
L.L.2 = Incluye equipo móvil, cargas de mantenimiento, cargas en zonas de almacenamiento, etc.

L.L.2A = Es la fracción de L.L.2, usada en el cálculo de las fuerzas sísmicas (ver Art. 19).

C.L. = Cargas de grúas, consta de su peso propio y una fracción de la carga que levanta. (Esta fracción deberá ser aprobada por el cliente.

I.L. = Cargas de Impacto.

VALORES DE CJ./Q PARA CONSTRUCCION DEL GRUPO B

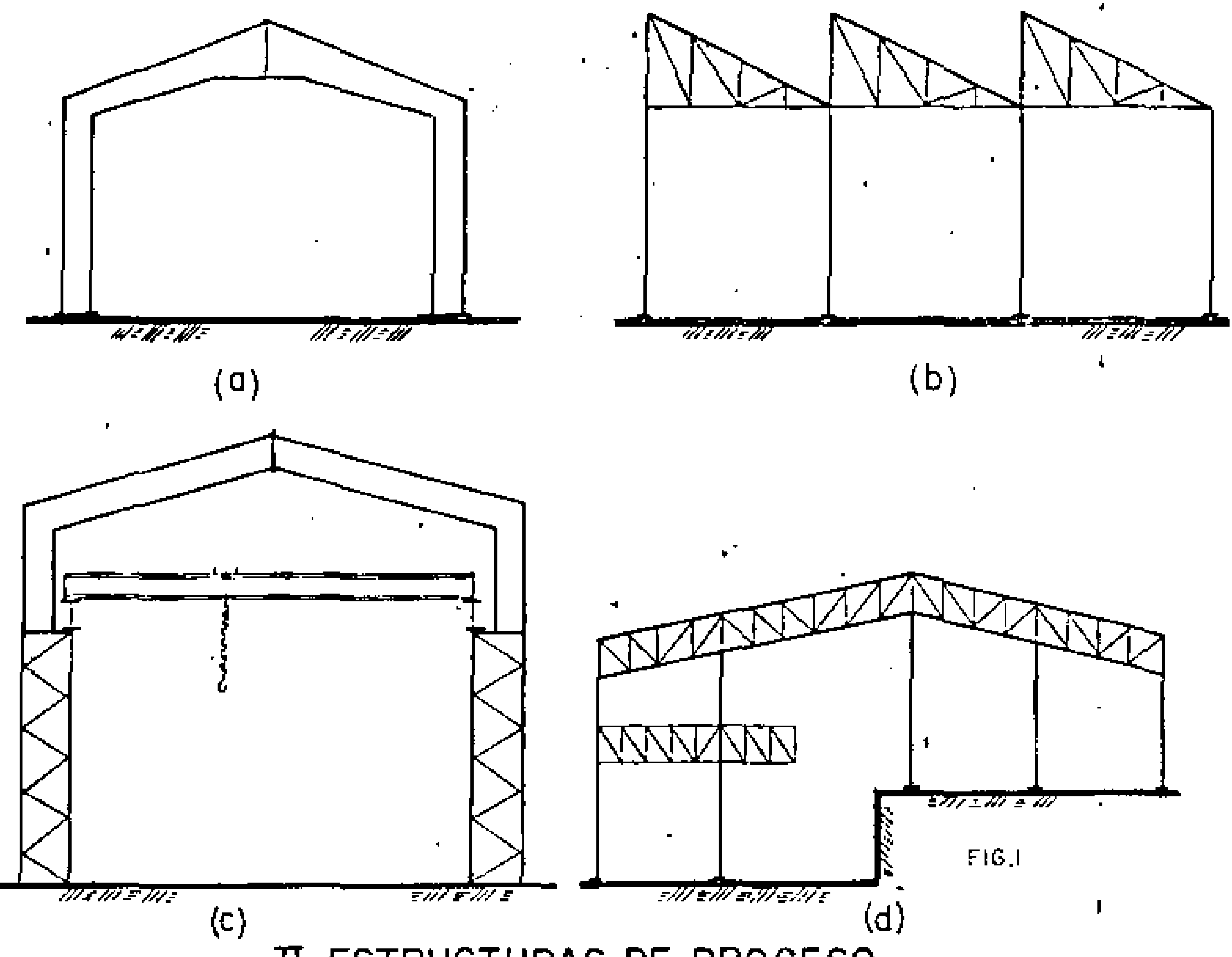
ESTRUCTURACION	TIPO	DESCRIPCION DE LOS ELEMENTOS RESISTENTES	VALOR CJ./Q 1	2	3
	1	MARCOS QUE CUMPLEN CON LOS REQUISITOS PARA Q=5. ADEMAS h/b ≤ 2 Y RIGIDEZ DE VIGAS Y COLUMNAS DEL MISMO ORDEN	0.12	0.10	0.11
	1	MARCOS CONTRAVENTEADOS O NO QUE CUMPLEN CON LOS REQUISITOS PARA Q=4 ADEMAS h/b ≤ 2 Y RIGIDECES DE VIGAS Y COLUMNAS DEL MISMO ORDEN.	0.15	0.12	0.13
	1 ó 2	MARCOS CON h/b ≥ 3, VIGAS Y COLUMNAS CON RIGIDEZ DEL MISMO ORDEN	0.20	0.16	0.18
		CONTRAVENTEADAS O NO QUE CUMPLAN CON LOS REQUISITOS PARA Q=4	0.15	0.12	0.13
		PARA h/b COMPRENDIDAS ENTRE 2 Y 3 SE INTERPOLARA LINEALMENTE ENTRE LOS VALORES.	Y 0.20	Y 0.16	Y 0.18
	1 ó 2	MARCOS O COLUMNAS			
	1 ó 2	MUROS DE CONCRETO O MAMPOSTERIA DE PIEZAS MACIZAS	0.30	0.24	0.27
	3	CHIMENEAS O ESTRUCTURAS TIPO TORRE			

PARA ESTRUCTURAS CLASE A × 1.3
PARA ESTRUCTURAS CLASE C × 0.8

1.- ACERO CON JUNTAS SOLDADAS O ATORNILLADOS POR FRICCION.
2.- ACERO CON JUNTAS REMACHADAS O ATORNILLADAS POR APOYO.
3.- CONCRETO O MAMPOSTERIA.

PRINCIPALES TIPOS DE ESTRUCTURAS INDUSTRIALES

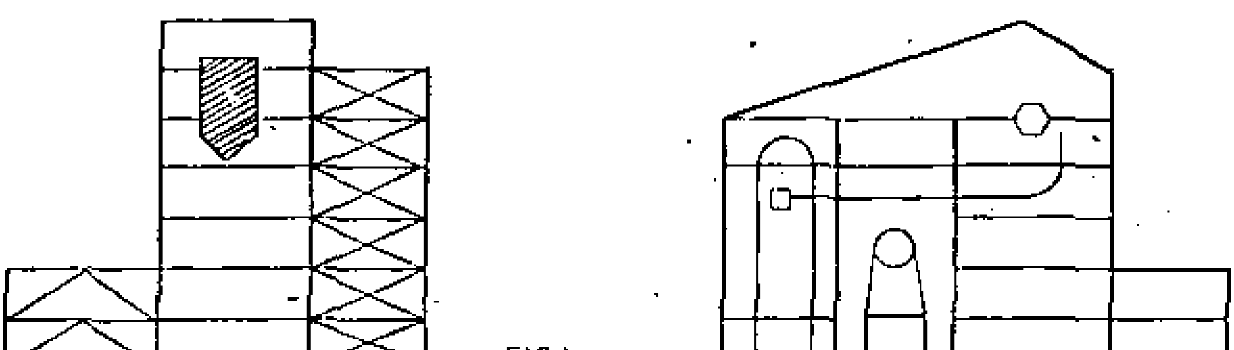
I. NAVES INDUSTRIALES CON Y SIN GRUA



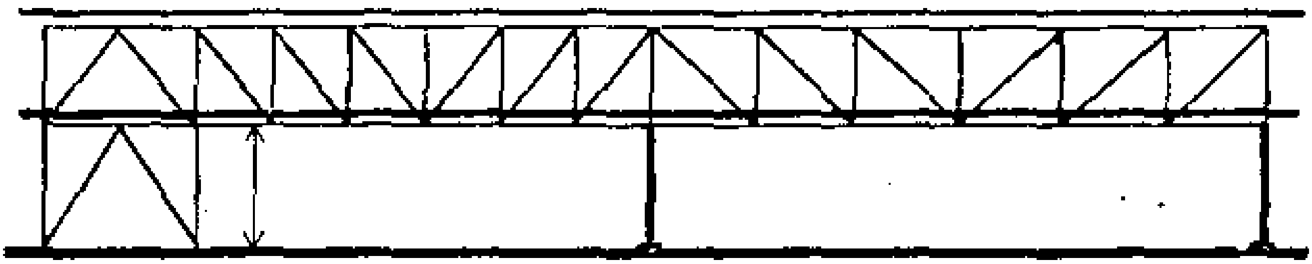
(a) (b) (c) (d)

FIG. I

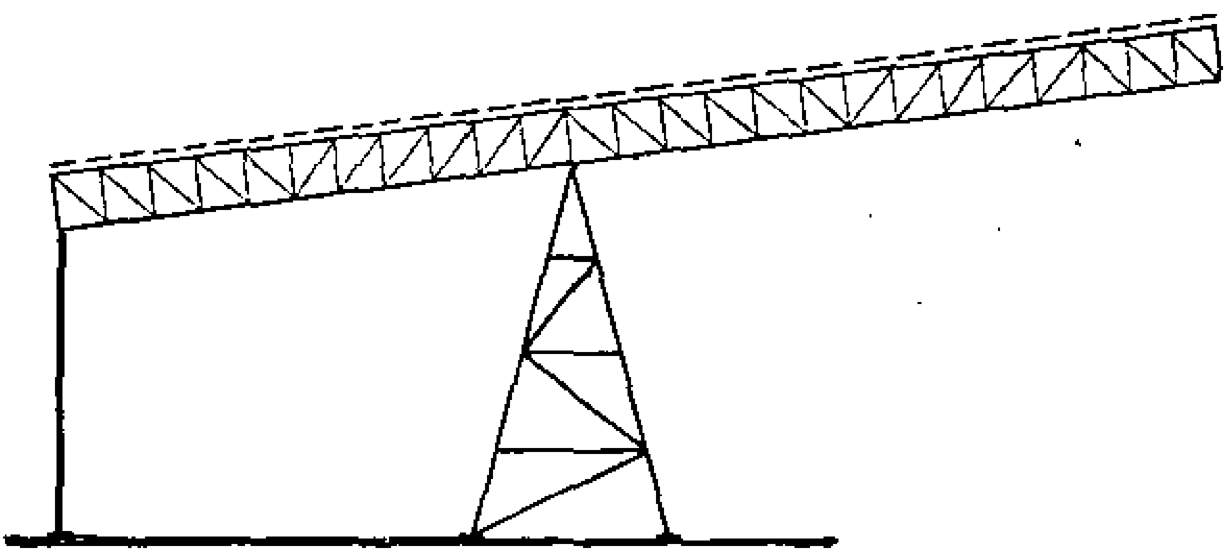
II ESTRUCTURAS DE PROCESO



III. SOPORTES DE TUBERIAS Y TRANSPORTADORES

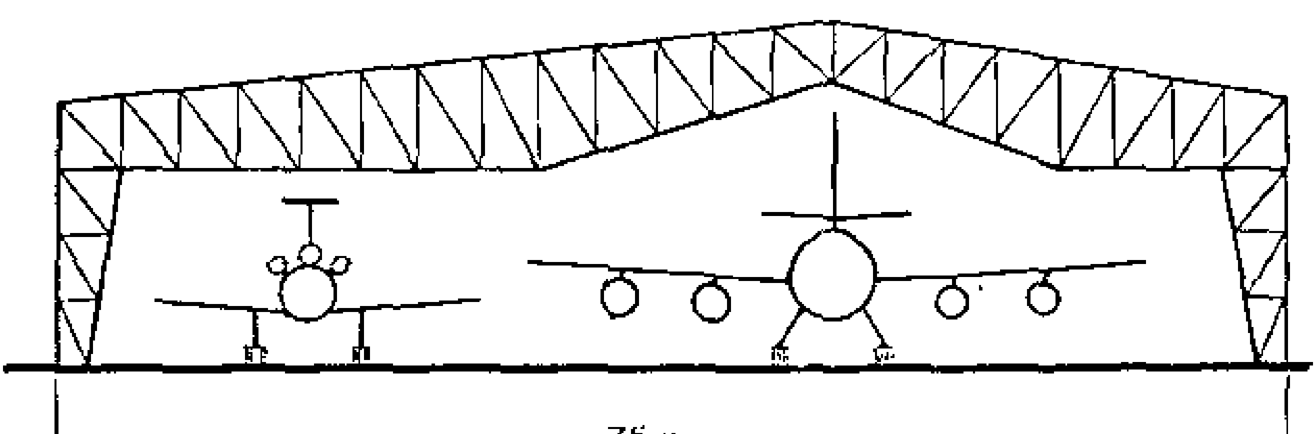


(a)



(b)

IV. ESTRUCTURAS DE GRANDES CLAROS



IX CURSO INTERNACIONAL DE INGENIERIA SISMICA
DISEÑO SISMICO DE ESTRUCTURAS ESPECIALES

TORRES Y CHIMENEAS

PROFESOR: M. en I. NEFTALI RODRIGUEZ CUEVAS

JULIO, 1983

TORRES Y CHIMENEAS

Prof. Neftalí Rodríguez Cuevas

1. Introducción

Las torres y chimeneas son estructuras esbeltas, de funciones múltiples, que se deben diseñar para soportar la acción de fuerzas horizontales, provocadas por viento o sismo; las cuales inducen efectos dinámicos en las estructuras de soporte.

En las figs. 1 a 5 se muestran algunos de los tipos comunes de torres y chimeneas construidas en diversas partes del mundo.

El análisis dinámico de estas estructuras requiere de algunos aspectos que no son comunes a otros tipos de estructuras, y en este trabajo se muestran las consideraciones comunes para su análisis.

2. Idealización para fines de análisis dinámico

Las estructuras de este tipo se idealizan comunmente como vigas Bernoulli-Euler, y su análisis se realiza en base a la teoría elemental de flexión, la cual implica que las secciones transversales permanecen planas al deformarse bajo la acción de fuerzas normales a su eje medio. Se acepta que los esfuerzos son proporcionales a las deformaciones unitarias, con flexión en un solo plano. Se considera además, que los desplazamientos son pequeños y que la deformación en cortante es pequeña.

Se consideran solo los efectos de inercia provocados por la traslación normal al eje de elementos diferenciales de la viga. No se considera el efecto de la inercia rotacional, igual a $-J \frac{\partial^3 v}{\partial x \partial t^2}$ por unidad de longitud, provocado

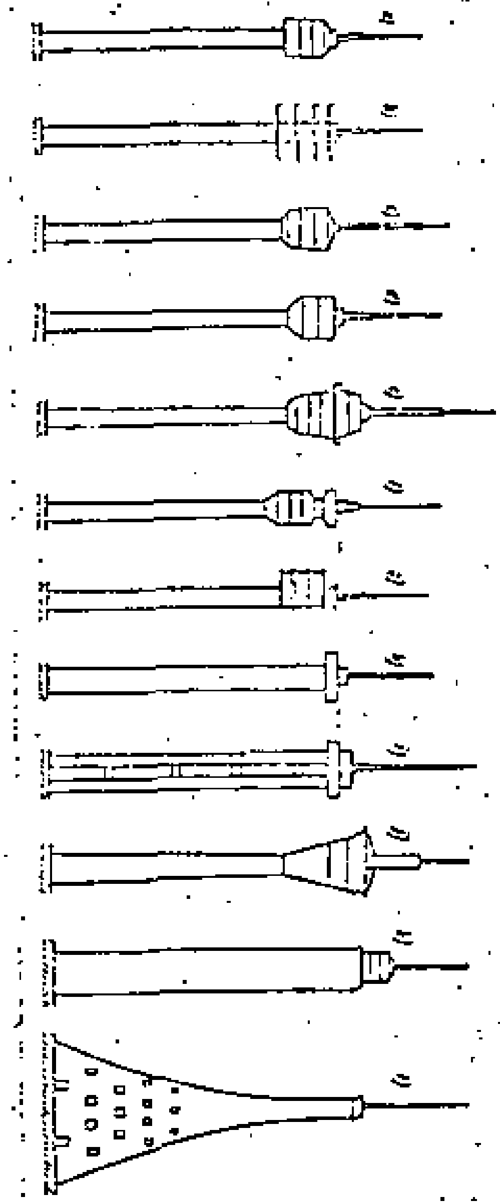


Fig. 1 Algunos tipos de torres construidas en diversas partes del mundo

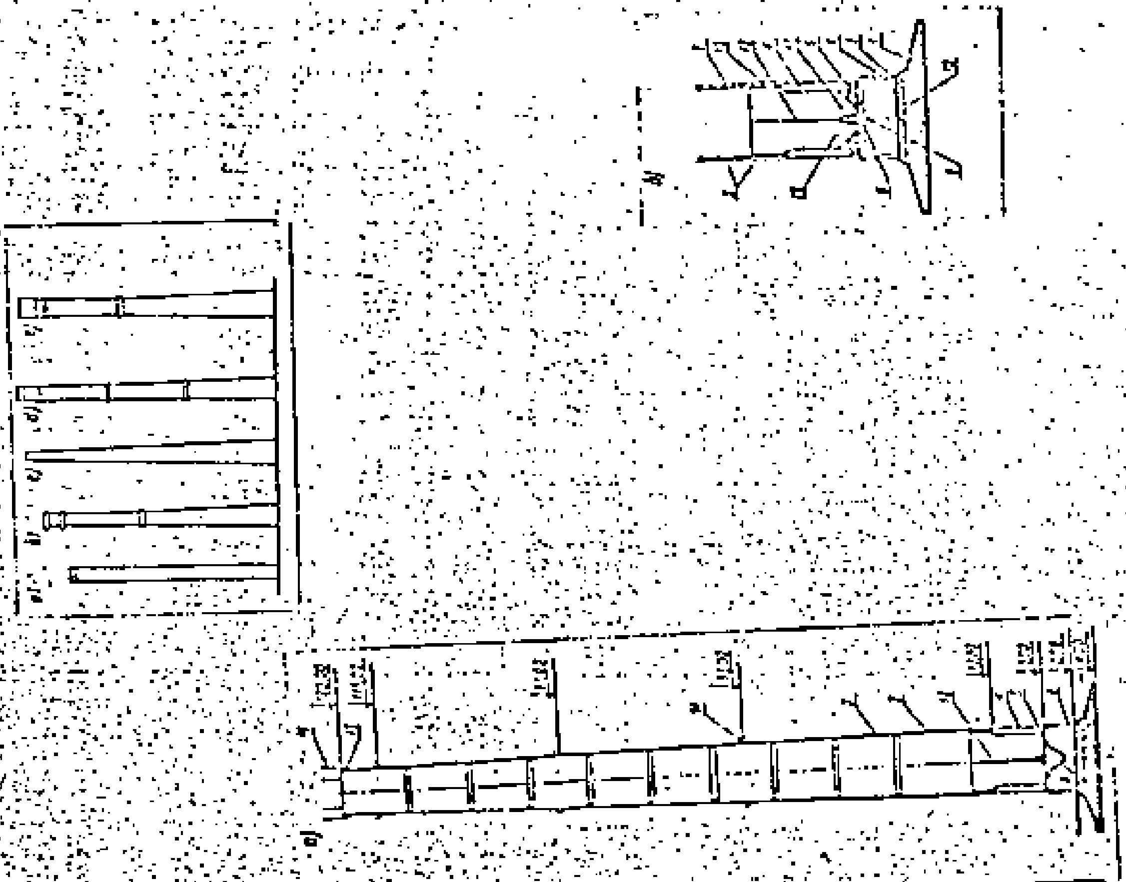


Fig. 3 Características principales de Chimeneas

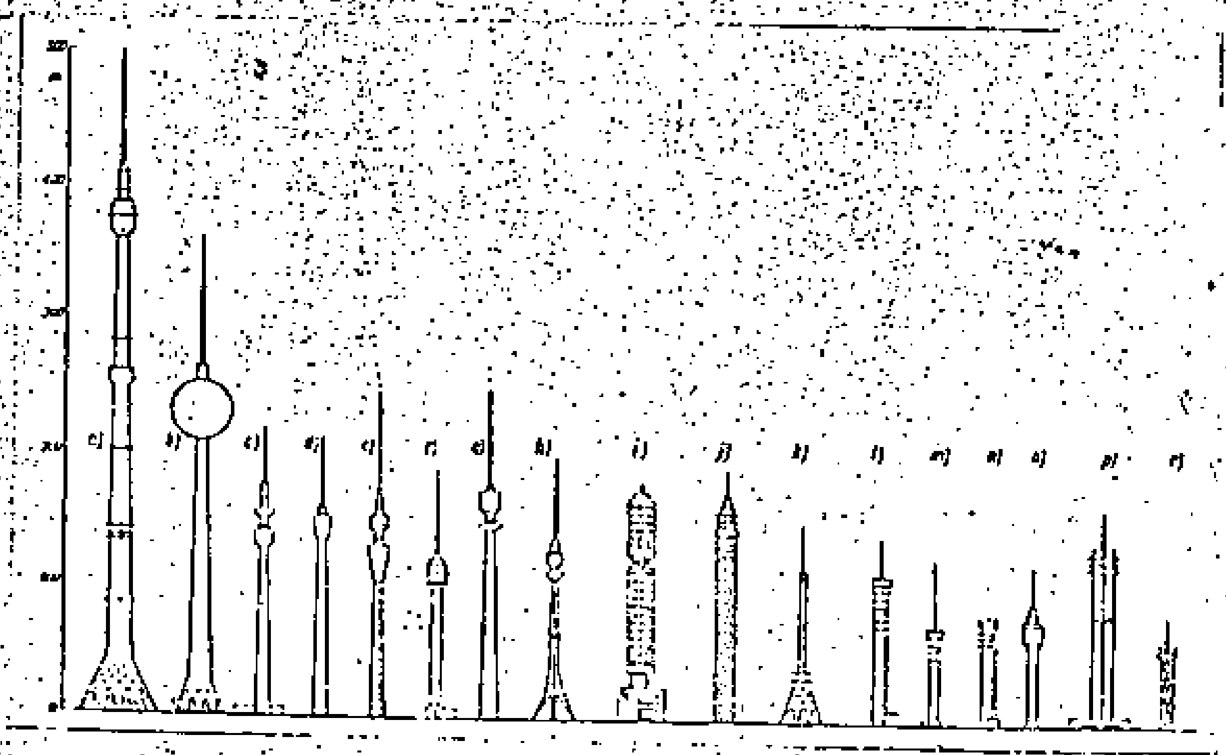


Fig. 2 Características geométricas de torres

a) Moscú
b) Berlín
c) Dortmund
d) Stuttgart
e) Dresden
f) Dequde
g) Donauturm, Viena
h) Belgrado
i) Londres
j) Cairo
k) Polonia
l) Zippendorf
m) Petersberg
n) Rhinow
o) Kulpenberg
p) Estocolmo
r) Hannover

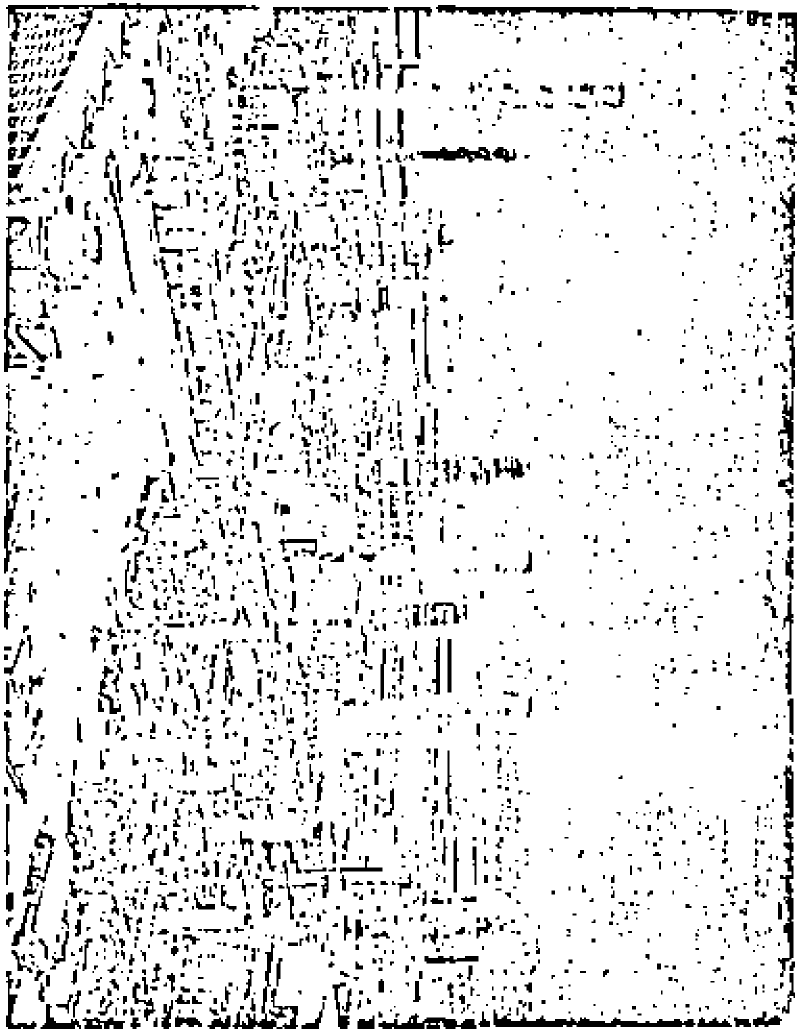
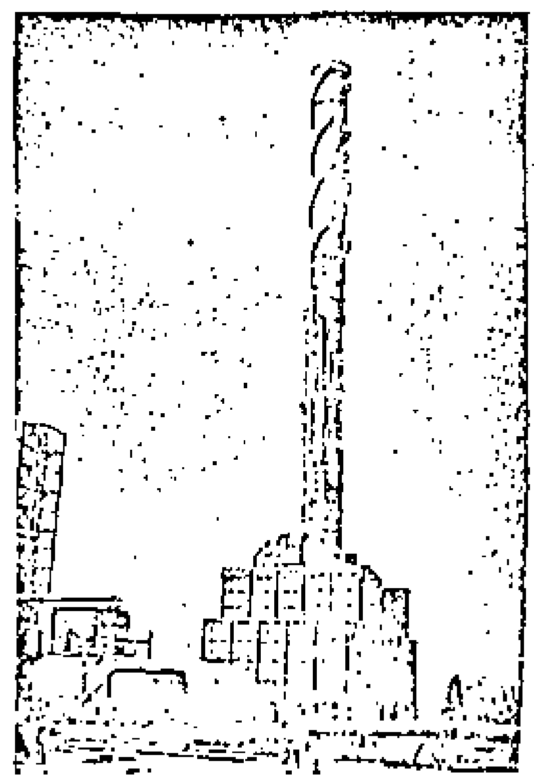


Fig. 4 Refinería en el norte del país, con torres y chimeneas

Fig. 5 Aspectos parciales de chimeneas y torres



por el giro angular $\frac{\partial v}{\partial x}$ de cada elemento, siendo v la translación normal al eje de la barra.

Cuando las dimensiones de la viga en su sección transversal no son pequeñas en comparación con su longitud, análisis que consideran los efectos de la fuerza cortante y la inercia rotacional se deben llevar a cabo.

En este escrito se presentan los aspectos sobresalientes del análisis dinámico de este tipo de estructuras, presentando la influencia relativa de la fuerza cortante y la inercia rotacional, así como de la fuerza normal. Cuando se considera que estos efectos no son significativos, se realizan análisis dinámicos simplificados que permiten conocer los desplazamientos y elementos mecánicos que permiten a su vez, revisar el análisis de las características geométricas y del material que forma a estas estructuras.

Viga Bernoulli-Euler

Al considerar la viga BE, cuyas características se muestran en la fig. 6 sometida a la acción de efectos dinámicos, considérese que $v = v(x,t)$ sea el desplazamiento transversal del eje neutro y $\mu(x)$ la masa por unidad de longitud. Los desplazamientos $v(x,t)$ producidos por la carga $p = p(x,t)$ son gobernados por la ecuación diferencial

$$\frac{\partial^2}{\partial x^2}\left(E\, I\, \frac{\partial^2 v}{\partial x^2} \right) = -\mu \frac{\partial^2 v}{\partial t^2} + P \qquad (3.1)$$

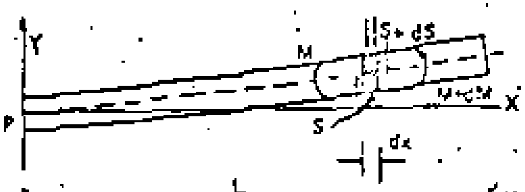


FIG.6

Cuando se generan vibraciones libres, es decir $p = 0$, aparecen modos normales de vibrar del tipo

$$v(x,t) = \phi(x) \;\; \text{sen}\; (\omega t + \epsilon) \qquad (3.2)$$

que al ser sustituidos en (3.1) conducen a la siguiente ecuación diferencial

$$\frac{d^2}{dx^2}\left(E\, I\, \frac{d^2}{dx^2}\phi \right) = \omega^2 \mu \phi \qquad (3.$$

Esta ecuación, junto con las condiciones de frontera de la viga, constituyen un problema de valores característicos, cuya solución conduce al conocimiento de las frecuencias naturales de cada uno de los i-esimos modos de vibrar y a la definición de sus formas características.

Vibraciones libres en piezas de sección constante.

Cuando $E\,I$ = cte. la ecuación 3.3. admite la solución general

$$\phi(x) = C_1\, Ch\left(\frac{\lambda x}{L}\right) + C_2\, Sh\left(\frac{\lambda x}{L}\right) + C_3 \cos\left(\frac{\lambda x}{L}\right) + C_4 \text{sen}\left(\frac{\lambda x}{L}\right) \qquad (3.4$$

donde

$$\lambda = L \sqrt[4]{\mu\omega^2 / EI}$$

Para torres y chimeneas, las condiciones de frontera resultan ser

$$\phi(0) = \phi'(0) = \phi''(L) = \phi'''(L) = 0$$

a partir de las cuales se obtiene la ecuación característica de frecuencias

$$\cos \lambda \; Ch\, \lambda + 1 = 0 \qquad (3.5)$$

cuyas raíces resultan ser :

$$\lambda_1 = 1.8751, \quad \lambda_2 = 4.6941, \quad \lambda_3 = 7.8548, \quad \lambda_4 = 10.9955$$

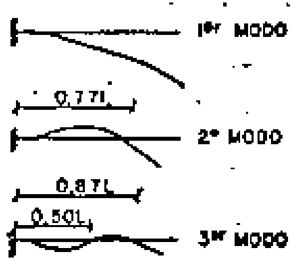
y para valores grandes de n

$$\lambda_n = (2n-1)\,\pi/4 \qquad (3.6)$$

con las formas modales correspondientes

$$\phi(x) = \mathrm{Ch}\left(\frac{\lambda nX}{L}\right) - \mathrm{Cos}\left(\frac{\lambda nX}{L}\right) - \frac{\mathrm{Ch}\,\lambda n + \mathrm{Cos}\,\lambda n}{\mathrm{Sh}\,\lambda n + \mathrm{sen}\,\lambda n}\left[\mathrm{Sh}\left(\frac{\lambda nX}{L}\right) - \mathrm{Sen}\left(\frac{\lambda X}{L}\right)\right] \qquad (3.7)$$

Las frecuencias naturales resultan ser :



$$\omega_n \doteq \frac{(n-\frac{1}{2})^2\pi^2}{L^2}\sqrt{\frac{EI}{\mu}}$$

$$\omega_1 = \frac{(0.597\pi)^2}{L^2}\sqrt{\frac{EI}{\mu}}$$

$$\omega_2 \doteq \frac{(1.494\pi)^2}{L^2}\sqrt{\frac{EI}{\mu}}$$

FIG.7

$$\omega_3 = \frac{(1.86\pi)^2}{L^2}\sqrt{\frac{EI}{\mu}}$$

a partir de los valores anteriores, se definen los períodos correspondientes mediante $T_n = 2\pi/\omega_n$;

Las formas características deben ser funciones que satisfacen las siguientes condiciones de ortogonalidad

$$\int_0^L \mu\,\phi_n\,\phi_m\,dx = 0 \qquad si \quad m \neq n$$

$$= m_n \qquad si \quad m = n$$

$$\int_0^L EI\,\phi_n''\,\phi_m''\,dx = 0 \qquad si \quad m \neq n \qquad (3.8)$$

$$= m_n\,\omega^2 \qquad si \quad m = n$$

donde $\int_0^L \mu\phi_n^2\,dx = m_n$

Vibraciones forzadas sin amortiguamiento

Cuando se considera a la viga sometida a un sistema excitador definido por una carga distribuida $p = p(x,t)$ y a una ó más fuerzas concentradas P_i a distancias a_i del apoyo, la ecuación de Lagrange conduce a la expresión

$$v(x,t) = \sum_{n=1}^{\infty} \phi_n(x)\left[A_n \cos\omega_n t + B_n \operatorname{sen}\omega_n t + \frac{1}{m_n\omega_n}\int_0^t Q_n(\tau)\operatorname{sen}\omega_n(t-\tau)\,d\tau\right] \qquad (3.9)$$

donde Q_n es la fuerza generalizada definida por

$$Q_n(t) = \int_0^L p(x,t)\,\phi_n(x)\,dx + \Sigma_i\,P_i(t)\,\phi_n(a_i) \qquad (3.10)$$

y los valores de A_n y B_n quedan definidos por :

$$A_n = \frac{1}{m_n}\int_0^L v_0\,\mu\,\phi_n\,dx$$

$$B_n = \frac{1}{m_n\omega_n}\int_0^L \dot{v}_0\,\mu\,\phi_n\,dx \qquad (3.11)$$

Vibraciones forzadas con amortiguamiento

Cuando en la viga (BE) existe una fuerza de amortiguamiento distribuida, igual a $C(x)\,\dot{v}$, donde $C(x)$ es un coeficiente de amortiguamiento viscoso, variable en x, definido como $C(x) = \beta\mu(x)$ donde β es una constante positiva, el desplazamiento normal v queda descrito por :

$$v(x,t) = \sum_{n=1}^{\infty} \phi_n(x) \left[e^{-\frac{\beta t}{2}} (A_n \cos p_n t + B_n \operatorname{sen} p_n t) + \right.$$

$$\left. + \frac{1}{m_n p_n} \int_0^L Q_n(\tau)\, e^{-\frac{\beta}{2}(t-\tau)} \operatorname{sen} p_n (t-\tau)\, d\tau \right] \qquad (3.12)$$

donde

$$p_n = \sqrt{\omega_n^2 - \left(\frac{\beta}{2}\right)^2}$$

y

$$A_n = \frac{1}{m_n} \int_0^L v_0\, \mu\phi_n\, dx \qquad B_n = \frac{1}{m_n p_n} \int_0^L \dot{v}_0\, \mu\phi_n\, dx + \frac{\beta A_n}{2 p_n}$$

4. Influencia de las condiciones de cimentación

En torres y chimeneas las condiciones de cimentación son importantes en su comportamiento bajo la acción dinámica de fuerzas horizontales.

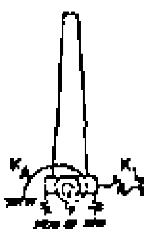


FIG 8

Las propiedades del terreno y el tipo de cimentación seleccionado influyen de manera importante en el análisis dinámico de estas estructuras.

Al considerar resortes que definen la acción del suelo sobre la chimenea, estos alteran los periodos naturales de la estructura y la forma de los modos de vibrar.

La ecuación característica se transforma en

$$x\left(\frac{x^2}{J} - \frac{1}{I}\right)\operatorname{sen} x\, \mathrm{Ch}\, x + x\left[\frac{x^2}{J} + \frac{1}{I}\right]\cos x\, \mathrm{Sh}\, x - \left[\frac{x^4}{IJ} + 1\right]\cos x\, \mathrm{Ch}\, x + \left(\frac{x^4}{IJ} - 1\right) = 0$$

(4.1)

donde

$$J = \frac{K_L}{EI/L^3} \qquad I = \frac{K_A}{EI/L}$$

K_L rigidez del resorte horizontal

K_A rigidez angular del resorte que restringe el giro de la cimentación.

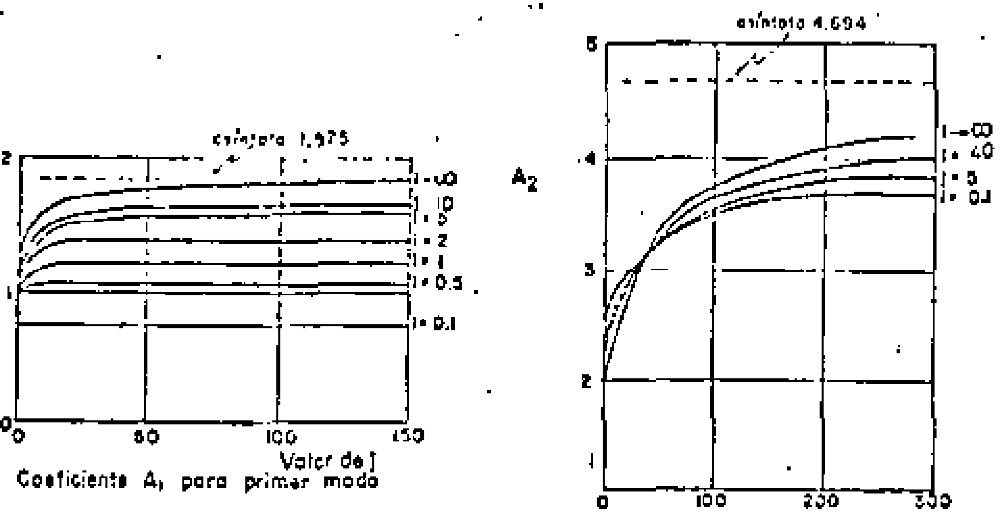
$x = \omega L$

ω frecuencia del primero modo

La frecuencia natural de la estructura puede ser escrita como

$$\omega_i = \frac{A_i^2}{L^2}\sqrt{\frac{EI}{\mu}} \qquad (4.2)$$

Los valores de A_i dependen de las características de los resortes K_A y K_L. Para estimarlos se puede recurrir a los diagramas siguientes; (ref. 1)



Coeficiente A_1 para primer modo

Coeficiente A_2 para el segundo modo

FIG 9

El análisis de este tipo de resultados ha permitido establecer las siguientes condiciones para los análisis dinámicos :

a) Cuando I y J son superiores a 10, para el análisis dinámico se puede recurrir al planteamiento del capítulo 3, a fin de estimar períodos formas características y respuesta dinámica, considerando empotrada la estructura.

b) Cuando $0.1 < I < 10$ y $1 < J < 10$ se deberá considerar la interacción suelo-estructura a fin de efectuar el análisis dinámico.

c) Si $I < 0.1$ y $J < 1$, se recomienda revisar las condiciones de cimentación para alcanzar valores comprendidos en el inciso a) ó b).

Esta última limitación se debe a que la carga crítica vertical de la estructura es sensible a la rigidez de los resortes K_A y K_L ; cuando existe $K_L \neq 0$ y K_A resulta inferior a:

$$(K_A)_{crit} = \sqrt{PEI}\ \tan\sqrt{\frac{PL^2}{EI}} \qquad (4.$$

donde P es la carga vertical, la viga BE se vuelve inestable. Así si se establecen las condiciones c) la estructura resulta inestable y tiende a pr ducir desplazamientos grandes al generarse la acción de fuerzas horizontale Las fuerzas horizontales, al actuar en la sección transversal, y modificar la rigidez, alteran también las frecuencias y modos de la estructura. Para estimar este efecto se utiliza la expresión

$$\omega_p = \frac{A^2}{L^2}\sqrt{\frac{EI}{\mu}}\sqrt{1-\frac{PL^2}{D^2EI}} \qquad (4.$$

donde ω_p frecuencia modificada por la fuerza axial P, coeficiente obtenido la siguiente gráfica.

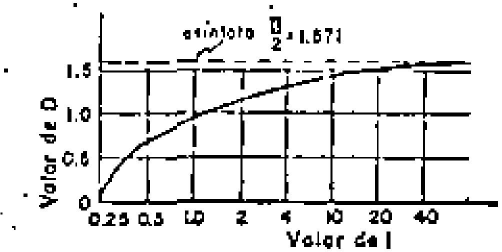


FIG 10

La fuerza axial P queda definida por el peso por unid de longitud de la viga (BE) multiplicado por la altura de la estructura.

En adición a los análisis previos, se debe revisar la estabilidad contra momento de volteo M_v calculado a nivel de la cara inferior de la losa inferior de la subestructura.

En cimentaciones por ampliación de base, en las cuales d es el diámetro exterior medio de la subestructura, es recomendable lograr que :

a) En suelos con capacidad inferior a 50 Kg/cm^2

$M_v \leq 0.3\ Pd$ para zapata circular u octagonal

$M_v \leq$ el menor de $0.3\left[1 + \left(\frac{d_2}{d_1}\right)^2\right] Pd$, ó $0.375\ Pd$ para zapatas anulares, donde d_1 es el diámetro exterior y d_2 el diámetro interior.

b) En suelos con capacidad superior a 50 Kg/cm^2

$M_v \leq 0.375\ Pd$

Cuando en la cimentación se recurre a pilotes se buscará enviter la aparición de tensiones en los pilotes, a menos de que se justifique el anclaje adecuado del pilote a la subestructura, y que el refuerzo sea suficiente.

Es recomendable en este último tipo de cimentación que la distribución de pilotes sea óptima a fin de soportar el momento de volteo, considerando la interacción entre los pilotes que forman a la cimentación. Análisis de grupos de pilotes, mediante algoritmos numéricos, debe efectuarse para verificar que las sobrecargas producidas por fuerzas horizontales sean soportadas sin daño, ni pérdida de capacidad.

5. Efecto del cortante y la inercia rotacional

El análisis clásico de (DE) es inadecuado para aquellas vigas en las cuales sus dimensiones de la sección transversal sean grandes. Rayleigh (ref. 2) introdujo el efecto de inercia rotacional y Timoshenko (ref. 3 y 4) consideró, en adición, el efecto de la distorsión producida por cortante.

Las ecuaciones acopladas para el desplazamiento total v, y la pendiente producida por flexión ψ, desarrolladas por Timoshenko son :

$$EI \frac{\partial^2 \psi}{\partial x^2} + k\left(\frac{\partial v}{\partial x} + \psi\right) AG + \frac{I\gamma}{g} \frac{\partial^2 \psi}{\partial t^2} = 0$$

$$\frac{\gamma A}{g} \frac{\partial^2 v}{\partial t^2} - k\left(\frac{\partial^2 v}{\partial x^2} - \frac{\partial \psi}{\partial x}\right) AG = 0 \tag{5.1}$$

Huang (ref. 5) desacopló las expresiones anteriores, obteniendo

$$EI \frac{\partial^4 v}{\partial x^4} + \frac{\gamma A}{g} \frac{\partial^2 v}{\partial t^2} - \left(\frac{\gamma I}{g} + \frac{EI}{gk} \frac{\gamma}{G}\right) \frac{\partial^4 v}{\partial x^2 \partial t^2} + \frac{\gamma I}{g} \frac{\gamma}{gkG} \frac{\partial^4 v}{\partial t^4} = 0$$

$$EI \frac{\partial^4 \psi}{\partial x^4} + \frac{\gamma A}{g} \frac{\partial^2 \psi}{\partial t^2} \frac{\partial^2 \psi}{\partial t^2} - \left(\frac{\gamma I}{g} + \frac{EI}{gk} \frac{\gamma}{G}\right) \frac{\partial^4 \psi}{\partial x^2 \partial t^2} + \frac{\gamma I}{g} \frac{\gamma}{gkG} \frac{\partial^4 \psi}{\partial t^4} = 0 \tag{5.2}$$

donde:

- E módulo de elasticidad
- G módulo de rigidez al cortante
- I momento de inercia de la sección transversal
- A área de la sección transversal
- γ peso por unidad de volumen
- k constante del factor de forma de la sección
- g aceleración de la gravedad

Haciendo $\quad y = Te^{ipt}$ $\qquad T, \Psi \quad$ formas modales

$\psi = \Psi e^{ipt}$ $\qquad p \quad$ frecuencia angular

$\xi = x/L$ $\qquad i = \sqrt{-1}$

Las ecuaciones (5.2) se transforman en :

$$\frac{\partial^4 y}{\partial\left(\frac{x}{L}\right)^4} + b^2(r^2 + s^2)\,\frac{\partial^2 y}{\partial\left(\frac{x}{L}\right)^2} - b^2(1-b^2r^2s^2)\,y = 0$$

(5.3)

$$\frac{\partial^4 \psi}{\partial\left(\frac{x}{L}\right)^4} + b^2(r^2 + s^2)\,\frac{\partial^2 \psi}{\partial\left(\frac{x}{L}\right)^2} - b^2(1-b^2r^2s^2)\,\psi = 0$$

donde $b^2 = \frac{1}{EI}\,\frac{\gamma A}{g}\,L^4 p^2$; $r^2 = \frac{I}{AL^2}$; $s^2 = \frac{EI}{kAGL^2}$

A partir de estas expresiones y al considerar las condiciones de frontera siguientes :

para $\quad \xi = 0 \qquad T(0) = 0, \qquad \psi(0) = 0$

$\xi = 1 \qquad \frac{1}{L}T'(1) - \psi(1) = 0, \quad \psi'(1) = 0$

Se obtiene la ecuación característica

$$2 + \left[b^2(r^2-s^2) + 2\right] \operatorname{Ch} b\alpha \cos b\beta + \frac{b(r^2+s^2)}{\sqrt{1-b^2r^2s^2}} \operatorname{Sh} b\alpha \operatorname{sen} b\beta = 0$$

donde

$$\begin{matrix}\alpha \\ \beta\end{matrix} = \frac{1}{\sqrt{2}}\sqrt{\mp(r^2+s^2) + \sqrt{(r^2-s^2)^2 + \frac{4}{b^2}}} \qquad (5.4)$$

Las formas características de los modos correspondientes quedan descritas por :

$$T = D\left[\operatorname{Ch} b\alpha\xi - \lambda\delta \operatorname{Sh} b\alpha\xi - \cos b\beta\xi + \delta \operatorname{sen} b\beta\xi\right] \qquad (5.5)$$

$$\Psi = H\left[\operatorname{Ch} b\alpha\xi + \frac{\epsilon}{\lambda\zeta} \operatorname{Sh} b\alpha\xi - \cos b\beta\xi + \epsilon \operatorname{sen} b\beta\xi\right]$$

donde

$$\delta = \frac{\frac{1}{\lambda}\operatorname{Sh} b\alpha - \operatorname{sen} b\beta}{\zeta \operatorname{Ch} b\alpha + \operatorname{Cos} b\beta} \qquad \epsilon = -\frac{\lambda \operatorname{Sh} b\alpha + \operatorname{sen} b\beta}{\frac{1}{\zeta}\operatorname{Ch} b\alpha + \cos b\beta}$$

$$\lambda = \frac{\alpha}{\beta} \qquad \zeta = \frac{\alpha^2+r^2}{\alpha^2+s^2} = \frac{\beta^2-s^2}{\beta^2-r^2}$$

A partir de los resultados previos se pueden obtener

1o El desplazamiento total $\qquad y = \sum_{i=1}^{\infty} a_i\, T_i\, e^{\sqrt{-1}\,p_i t} + c_i$

2o.La pendiente generada por flexión $\qquad \psi = \sum_{i=1}^{\infty} \bar{a}_i\, \Psi_i\, e^{\sqrt{-1}\,p_i t} + \bar{c}_i$

3o La pendiente producida por cortante $\qquad \phi = \frac{\partial y}{\partial x} - \psi$

4o.El momento flexionante $\qquad M = -EI\,\frac{\partial\psi}{\partial x}$

5o La fuerza cortante $\qquad Q = k\,\phi AG$

Las constantes a_i, c_i, $\bar{a}_i$, $\bar{c}_i$, se valuan en términos de las condiciones iniciales de las vibraciones libres en estudio

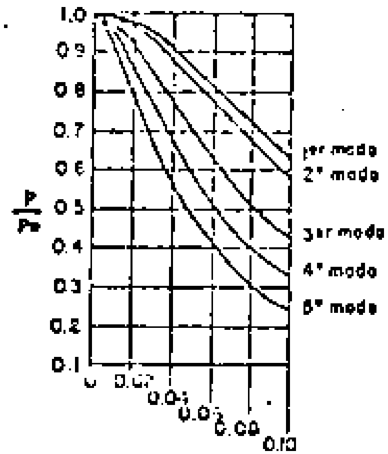


Fig. 11

Relación entre la frecuencia de la viga BZ (P_o) y la de Timoshenko (P)

A fin de ilustrar el efecto de la inercia rotacional y el cortante, a continuación se muestran resultados obtenidos al aceptar

$\frac{E}{kG} = 4$ s=25 en piezas de acero

en el cálculo de los cinco primeros modos

Si r = 0.02 se obtiene

Modo	Primero	Segundo	Tercero	Cuarto	Quinto	rotacion
p/p_o	0.985	0.975	0.930	0.883	0.835	p frecuencia con cortante
% error	3	3	8	13	20	p_o frecuencia (BZ)

Por lo que respecta a la forma distorsionada de la estructura, en la figura siguiente se muestra el efecto de la inercia rotacional y el cortante.

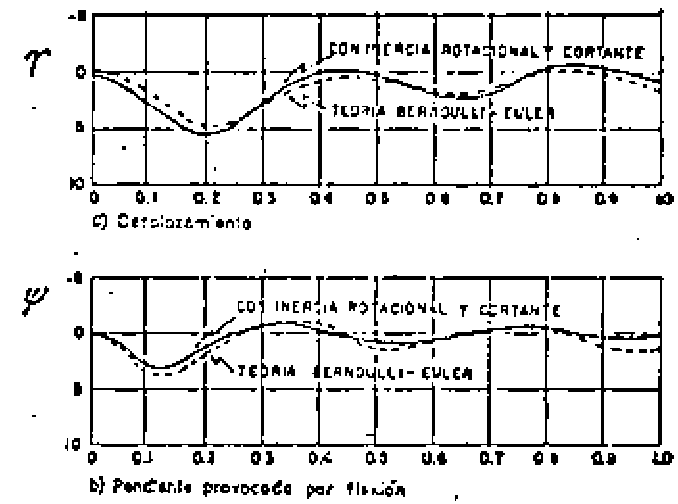


FIG. II

Planteamientos recientes (ref. 6) en vigas donde se considera la aparición de amortiguamiento de un sólido viscoelástico muestran la posibilidad de incluir estos efectos en el análisis dinámico de estructuras esbeltas.

Influencia del cambio en momento de inercia

En ocasiones las chimeneas y torres se hacen con momento de inercia variable con la altura, ocasionado por el cambio en diámetro y espesor de la pared.

En este caso, el análisis dinámico parte de la ecuación diferencial (3.1), y mediante métodos numéricos se encuentra la solución al problema de definir las frecuencias y formas características

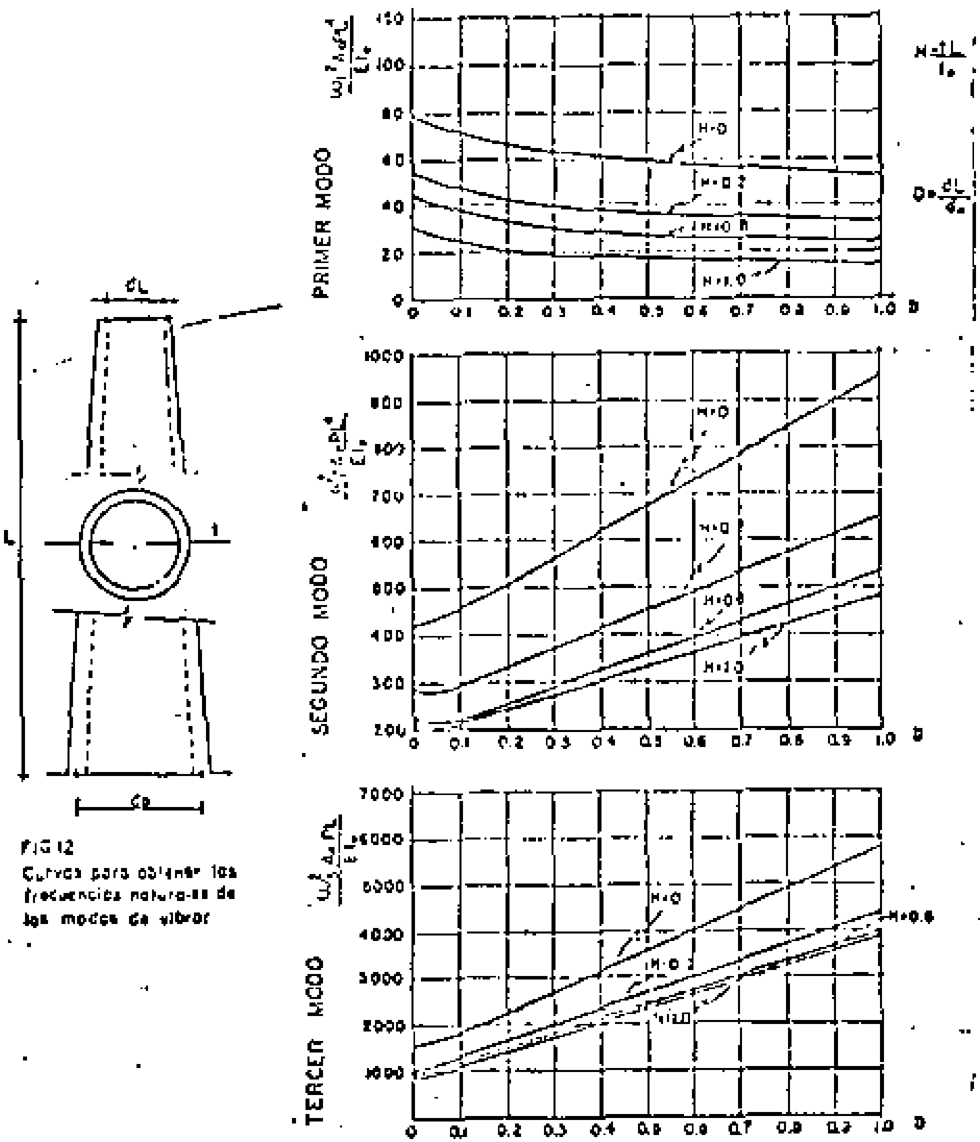


FIG 12
Curvas para obtener las frecuencias naturales de los modos de vibrar

En la referencia 7 se proporcionan tablas de desplazamientos y sus primeras y segundas derivadas de las formas modales así como las frecuencias correspondientes. En la fig. 13 se condensan los resultados para estructuras cónicas truncadas de espesor linealmente variable, que permiten definir las frecuencias de los tres primeros modos de vibrar.

Para chimeneas con porción cilíndrica y cónica, usualmente se recurre a buscar una chimenea de diámetro constante, ds, igual al de la porción cilíndrica y se usa una altura equivalente.

$$H_e = H_i + H_s \left(\frac{2d_s}{d_s + d_b} \right)^2 \qquad (6.1)$$

donde

H_e altura equivalente

H_i altura del cono inferior

H_s altura del cilindro

d_s diámetro medio de la parte cilíndrica

d_b diámetro medio en la base de la chimenea

7. Influencia de la distribución de masa

En torres y chimeneas puede suceder que se presenten masas concentradas a lo largo del eje de la chimenea o torre. Esto puede alterar notablemente la idealización de la estructura y conducir a sistemas masa-resorte, en las cuales sea necesario recurrir a métodos numéricos para resolver el problema de valores característicos. En la fig. 13 se muestra la idealización común de una chimenea con muros de aislamiento sobre ménsulas. En estas estructuras el análisis se realiza concentrando la masa del fuste, el muro de aislamientos y las ménsulas en la posición de estas últimas. Los tramos de fuste, que funcionan como resortes equivalentes, presentan desplazamientos, rotaciones y momentos flexionantes y fuerzas cortantes que son descritos por las siguientes matrices de transferencia:

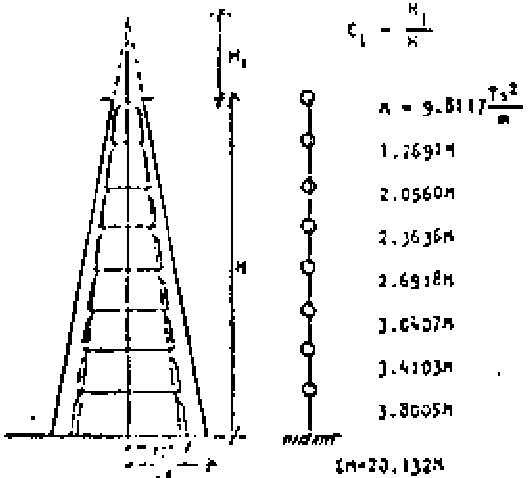


Fig. 13. Chimenea y su idealización en sistema de masas y resortes

a) Vigas Bernoulli-Euler

$$\begin{bmatrix} -v \\ \psi \\ M \\ T \end{bmatrix}_i = \begin{bmatrix} 0 & 1 & 0 & -\frac{1}{GA} \\ 0 & 0 & \frac{1}{EI} & 0 \\ 0 & -\mu\rho^2\omega^2 & 0 & 1 \\ \mu\omega^2 & 0 & 0 & 0 \end{bmatrix} \begin{bmatrix} -v \\ \psi \\ M \\ T \end{bmatrix}_{i-1} \qquad (7.1)$$

donde

i, sección en la que se valúan los elementos mecánicos

v, ψ, M, T desplazamiento, giro, momento y fuerza cortante

ρ radio de giro de la sección transversal

ω frecuencia circular de vibración

μ masa por unidad de longitud

b) Vigas de Timoshenko sometidas a fuerza axial

$$\begin{bmatrix} -v \\ \psi \\ M \\ T \end{bmatrix}_i = \begin{bmatrix} C_0-\sigma C_2 & l\left[C_1-(\sigma+\tau)C_3\right] & aC_2 & \frac{al}{\beta^4}\left[-\sigma C_1+(\beta^4+\sigma^2)C_3\right] \\ \frac{\beta^4}{l}C_3 & C_0-\tau C_2 & \frac{a}{l}(C_1-\tau C_3) & aC_2 \\ \frac{\beta^4}{a}C_2 & \frac{l}{a}\left[-\tau C_1+(\beta^4+\tau^2)C_3\right] & C_0-\tau C_2 & l\left[C_1-(\sigma+\tau)C_3\right] \\ \frac{\beta^4}{al}(C_1-\sigma C_3) & \frac{\beta^4}{a}C_2 & \frac{\beta^4}{l}C_3 & C_0-\sigma C_2 \end{bmatrix} \begin{bmatrix} -v \\ \psi \\ M \\ T \end{bmatrix}_{i-1}$$

donde

$$a = \frac{l^2}{EI} \qquad C_0 = \Lambda(\lambda_2^2 \,\mathrm{Ch}\,\lambda_1 + \lambda_1^2 \cos\lambda_2) \qquad (7.2)$$

$$\beta^4 = \frac{\mu\omega^2}{EI} l^4 \qquad C_1 = \Lambda\left(\frac{\lambda_2^2}{\lambda_1}\,\mathrm{Sh}\,\lambda_1 + \frac{\lambda_1^2}{\lambda_2}\,\mathrm{sen}\,\lambda_2\right)$$

$$\sigma = \frac{\mu\omega^2}{GA} l^2 \qquad C_2 = \Lambda(\mathrm{Ch}\,\lambda_1 - \cos\lambda_2)$$

$$\tau = \frac{Pl^2}{EI} + \frac{\rho^2\mu\omega^2}{EI} l^2 \qquad C_3 = \Lambda\left[\frac{1}{\lambda_1}\,\mathrm{Sh}\,\lambda_1 - \frac{1}{\lambda_2}\,\mathrm{sen}\,\lambda_2\right]$$

$$\lambda_{1,2} = +\sqrt{\sqrt{\beta^4 + \frac{1}{4}(\sigma-\tau)^2} \mp \frac{1}{2}(\sigma+\tau)} \qquad \Lambda = \frac{1}{\lambda_1^2+\lambda_2^2}$$

l es la longitud entre las secciones i e $i-1$

P es la fuerza normal media en el tramo

G módulo de rigidez al esfuerzo cortante

A área de la sección transversal

Se observa que el cálculo de las constantes de resorte resulta muy laborioso, cuando se incluye el efecto de inercia rotacional, fuerza cortante y fuerza normal.

Conocidas las masas y las constantes de resorte se plantean las ecuaciones del movimiento reducidas y se obtienen los valores característicos y las formas modales correspondientes.

En la práctica es común recurrir al método de Newmark para valuar las constantes de resorte; para resolver la ecuación de frecuencias y obtener los modos naturales, se recurre a programas que resuelven el problema en ordenadores digitales. Así, para una chimenea de 80m de altura, cuya distribución de masas aparece en la fig. 3, se obtuvieron las frecuencias, períodos y factores de participación de modo que aparecen en la siguiente tabla

Modo	Frecuencia ($\frac{rad}{seg}$)	Período (seg)	Coeficiente de participación modal
1°	3.4756	1.807765	0.845943
2°	15.3283	0.403913	-0.023441
3°	38.2289	0.164355	-0.003045
4°	71.8242	0.087482	-0.002545
5°	115.7396	0.054287	+0.000196
6°	163.0762	0.037132	+0.000075
7°	225.5712	0.027854	+0.000035
8°	295.8693	0.021236	-0.000020

Se observa que la participación de los modos superiores es poco significativa en la respuesta, debido a la diferencia notable en los coeficientes de participación modal.

Por ello, en ocasiones para estimar el período del primer modo se recurre al método de Dunkerly en el cual

$$\omega_1 \leq \frac{1}{\sqrt{\sum_{i=1} m_i \, v_i^*}} \qquad (7.3)$$

donde

m_i es la iésima masa

v_i^* el desplazamiento de la chimenea, en la i-ésima masa, al ser sometida a la acción de su peso propio

Estudios en más de 40 chimeneas mostraron que el período natural del primer modo varía linealmente con la altura, una vez que se definen H, r_a y e, obteniéndose valores comprendidos entre

$$0.008\, H < T \leq 0.020H \qquad (7.4)$$

donde

T período natural en seg.

H altura de la chimenea, en m.

8. Consideraciones sobre análisis sísmico

La respuesta de torres y chimeneas es compleja con aspectos dinámicos importantes. Existen demasiadas incógnitas para predecir con certidumbre la respuesta de estas estructuras bajo la acción de sismos futuros.

Se tiene que depender en aspectos cualitativos, en los cuales el buen juicio debe estar presente y de análisis cuantitativos de respuesta en base a sismos registrados en el pasado.

Normalmente el ingeniero recurre a simplificaciones contenidas en reglamentos, como el del diseño en el Distrito Federal, o al SCAGC en los cuales se establecen espectros de diseño en base a los cuales se define la respuesta estructural.

En lo que se sigue se presenta un análisis simplificado y la secuencia de análisis dinámico comúnmente usada en nuestro medio.

Las torres y chimeneas se analizarán de manera independiente en dos direcciones ortogonales, y se verificará que las estructuras sean capaces de resistir cada una de estas condiciones por separado.

En la revisión se deberá buscar los desplazamientos, y elementos mecánicos en diversas secciones transversales, así como las aceleraciones que se presentan en los conos de aislamiento. Se revisarán además las condiciones de estabilidad de la cimentación, para ello se dispone de los siguientes procedimientos.

a) Estático equivalente

b) Dinámico espectral

c) Dinámico bajo la acción de sismos registrados

El primer procedimiento basado en la experiencia obtenida al resolver decenas de chimeneas, es aplicable cuando la cimentación satisface las condiciones descritas en el cap. 4, cuando I y J son superiores a 10.

Para fines de diseño inicial, se aceptará la existencia de una carga estática que actúe lateralmente contra la chimenea, con una distribución bilineal definida a continuación :

a) En la base, la fuerza será nula. Aumenta linealmente con la altura hasta 0.3H, donde la carga será igual al 15% del valor máximo en la parte superior de la chimenea y es igual, a la altura 0.3H, a 0.35 $C_M W/H$, siendo C_M el coeficiente sísmico mínimo, W , el peso total de la chimenea sobre la cimentación y H la altura total de la chimenea.

b) Desde 0.3 H hasta H, se aceptará otra variación lineal de la fuerza sísmica, con un valor máximo en la parte superior, igual a 2.35 $C_M W/H$.

La distribución de fuerzas cortantes y momentos flexionantes, así como los desplazamientos horizontales, se estimarán en base a la distribución bilineal antes descrita.

El momento de volteo en la base de la chimenea resulta próximo a $M_v = C_M WH/\sqrt{2.15}$

Cuando no exista mejor información, es posible estimar el valor de C_M , en base a la siguiente tabla, en la que aparecen las cuatro regiones sísmicas en las que se ha dividido el país.

Zona sísmica	A	B	C	D
Coeficiente C_M	0.03	0.05	0.09	0.18

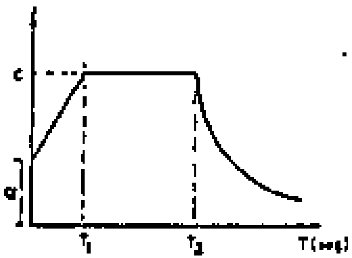
El análisis dinámico espectral, considera a las estructuras como sistemas masas-resortes, en los cuales se aplican aceleraciones definidas por espectros de diseño. Este procedimiento es válido cuando las condiciones de cimentación tienen I y J mayores a 10, y considera tres tipos de suelos.

Tipo I Terreno firme, similar a conglomerados compactos, areniscas medianamente cementadas, o arcillas compactas.

Tipo II Suelos de baja rigidez, como arenas sin cementar, limos de mediana o alta compacidad ó arcillas de mediana compacidad.

Tipo III Arcillas blandas muy compresibles

Los coeficientes de diseño sísmico se definen mediante espectros cuyas características se describen en la tabla siguiente.



Forma del espectro

Fig. 14

Zona Sísmica	Tipo de Suelo	c	T_1	T_1	a
A	I	0.10	0.40	0.60	0.05
	II	0.16	0.75	1.50	
	III	0.21	1.00	2.50	
B	I	0.21	0.40	0.60	0.10
	II	0.26	0.75	1.50	
	III	0.31	1.00	2.50	
C	I	0.31	0.30	0.50	0.15
	II	0.29	0.60	1.20	
	III	0.47	0.80	2.20	
D	I	0.62	0.20	0.40	0.30
	II	0.73	0.40	1.00	
	III	0.83	0.60	2.00	
			seg	seg.	

Se considera que las zonas del espectro, en cada intervalo, queda definida en forma por las expresiones :

$$C_b = a + (c - a)\frac{T}{T_1}, \quad \text{si} \quad T < T_1$$

$$C_b = c \qquad \text{si} \quad T_1 < T < T_2$$

$$C_b = c\left(\frac{T_2}{T}\right) \qquad \text{si} \quad T > T_2$$

donde T es el período natural de alguno de los modos de vibración, en seg.

Ya que en estos espectros se han considerado efectos inelásticos, considerando una ductilidad definida por un factor de ductilidad Q = 2, solo los momentos flexionantes y fuerzas cortantes se dividirán entre 2 si $T > T_1$, ó entre $1 + T/T_1$ en caso contrario.

Finalmente el procedimiento de análisis dinámico bajo la acción de sismos registrados es aconsejable para aquéllas estructuras en las cuales debe considerarse la interacción suelo-estructura, como puede verse en la ref. 8.

9. Análisis dinámico simplificado

A fin de ilustrar la aplicación del procedimiento espectral simplificado, existe un programa elaborado en el Instituto de Ingeniería, UNAM, que permite realizar el análisis dinámico modal de chimeneas, siguiendo la siguiente secuencia :

a) Calcula el volumen de fuste y de las mensulas y lo multiplica por la masa específica para definir la masa asociada a cada ménsula.

b) Obtiene la masa de los conos de aislamiento y la agrega a la masa de la estructura en cada ménsula.

c) Calcula la matriz de rigideces del sistema de resortes equivalentes, recurriendo al método de Newmark

d) Resuelve el problema de valores característicos y define las frecuencias y modos naturales de vibración

e) Obtiene la respuesta, a partir de un espectro de diseño, pudiendo seguir cualquiera de los siguientes criterios:

$$R_1 = \sqrt{\sum_{i=1}^{n} R_i^2} \qquad R_2 = \left|\sum_{i=1}^{n} R_i\right| \qquad R_3 = (R_1 + R_2)/2$$

f) Calcula momentos flexionantes, fuerzas cortantes y desplazamientos y los grafica automaticamente.

A continuación se muestran los resultados obtenidos en el análisis dinámico modal de una chimenea de concreto de 80 m. de altura, de sección variable, con un radio exterior en la base de 4.625 m y un radio interior en la base igual a 4.125 m. Se considera H_1 = 202 m para el cono exterior y 246.4 m en el cono interior. En el análisis se aceptó E = 2.51 T/m^2, ν = 0.15; un peso volumétrico del fuste de 2.4 T/m^3 y una resistencia del concreto igual a 2500 T/m^2.

Se dividió a la chimenea en 8 tramos de 10 m colocando muros aislantes con un peso de 2.3 T/m^3 y en el recubrimiento exterior; 2 T/m^3. Para el mortero se consideró 0.55 T/m^3. El ancho del tabique refractario se consideró de 23 cm; el ancho del recubrimiento adicional de 0.065 m y 0.003 m de mortero.

El análisis modal proporcionó los siguientes resultados

Modo	Frecuencia	Período	Coeficientes de Participación Modal
1	5.4415	1.1547	+ 0.405973
2	25.9761	0.2423	+ 0.0117654
3	61.5485	0.1043	+ 0.001328
4	97.6793	0.0642	+ 0.000450
5	116.2322	0.0540	- 0.000022
6	128.9721	0.0487	+ 0.0000043
7	149.9023	0.0419	+ 0.0000005
8	189.8007	0.0331	+ 0.00000001

Se seleccionó un espectro de diseño correspondiente a la zona D, con un suelo tipo II, considerando un valor máximo de C = 0.730 y se empleo un factor de ductilidad igual a 2.

Se hicieron análisis comparativos considerando la participación de 1 hasta 8 modos, y se calcularon las respuestas R_1, R_2 y R_3, las cuales aparecen en las siguientes tablas:

Desplazamientos máximos en las masas

Masa	Un solo modo			Todos los modos		
	R_1	R_2	R_3	R_1	R_2	R_3
1	0.4327	0.4327	0.4327	0.4328	0.4446	0.4387
2	0.3518	0.3518	0.3518	0.3518	0.3568	0.3543
3	0.2723	0.2723	0.2723	0.2723	0.2732	0.2727
4	0.1974	0.1974	0.1974	0.1974	0.2017	0.1956
5	0.1307	0.1307	0.1307	0.1308	0.1361	0.1335
6	0.0755	0.0755	0.0755	0.0756	0.0805	0.0781
7	0.0343	0.0343	0.0343	0.0344	0.0376	0.0360
8	0.0089	0.0089	0.0089	0.0089	0.0103	0.0094

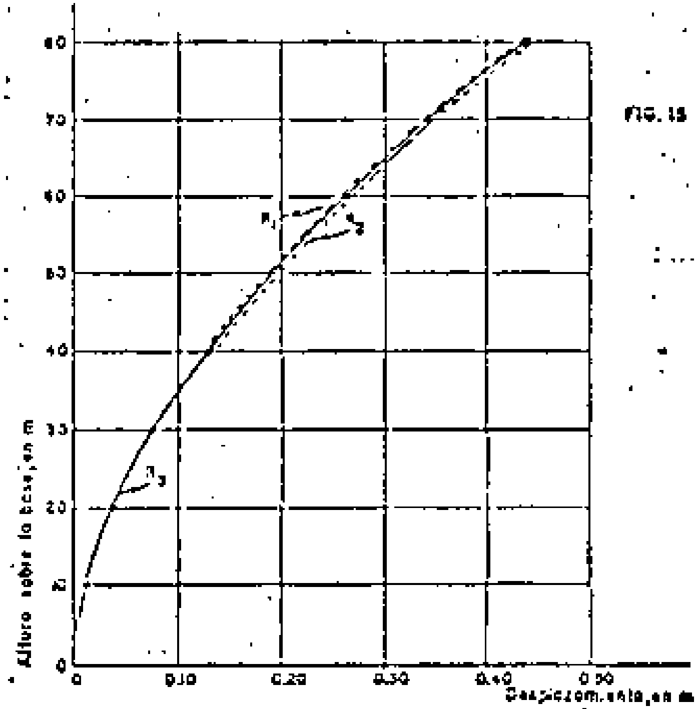


FIG. 15

Variación de los desplazamientos con la altura, con los tres tipos de respuesta.

Fig. 15

Momentos flexionantes en las masas

Masa	Un solo modo			Todos los modos		
	R_1	R_2	R_3	R_1	R_2	R_3
1	0	0	0	0	0	0
2	355.5	355.5	355.5	474.6	909.6	692.1
3	2042	2042	2042	2327	3718	3022
4	4694	4694	4694	5197	7051	6124
5	8660	8660	8660	8878	10960	9919
6	13120	13120	13120	13200	15120	14160
7	16020	18320	18020	18320	10875	18450
8	23130	23130	23130	23230	25270	24230
BASE	28300	28300	28300	28560	33500	31030

En ton-m

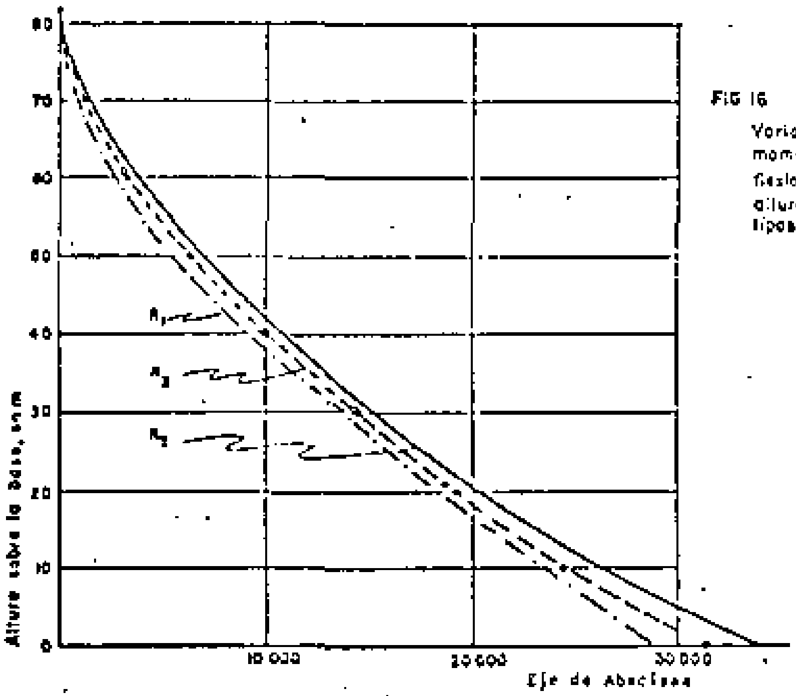


FIG 16 Variación del momento flexionante con la altura, con los tres tipos de respuesta

Momento flexionante en Ton-m

Finalmente, para la variación de la fuerza cortante se obtuvieron los siguientes resultados, en toneladas.

Masa	Un solo modo			Todos los modos		
	R_1	R_2	R_3	R_1	R_2	R_3
1	0	0	0	0	0	0
2	35.55	35.55	35.55	47.46	[illegible]	[illegible]
3	168.7	168.7	168.7	197.4	283.9	234.1
4	284.2	284.2	284.2	294.2	392.8	343.5
5	377.6	377.6	377.6	381.1	464.1	422.6
6	446.0	446.0	446.0	452.1	574.2	513.1
7	489.6	489.6	489.6	507.9	651.1	583.5
8	511.3	511.3	511.3	545.6	780.3	662.9
BASE	517.4	517.4	517.4	563.3	883.3	723.3

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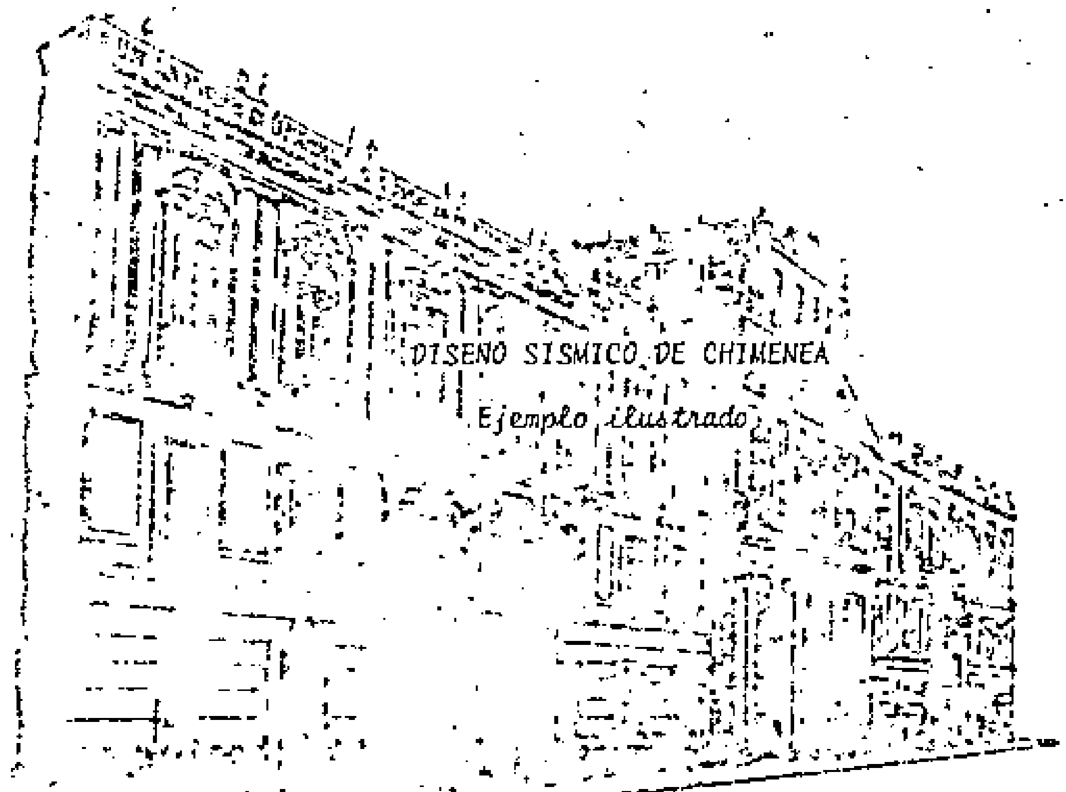
IX CURSO INTERNACIONAL DE INGENIERIA SISMICA

DISEÑO SISMICO DE ESTRUCTURAS ESPECIALES



DISEÑO SISMICO DE CHIMENEA

Ejemplo ilustrado

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EJEMPLO DE ANALISIS DINAMICO DE UNA CHIMENEA

a) Aspectos generales

Se ilustra a continuación la secuela de análisis que previamente se discutió, aplicada al cálculo dinámico de una chimenea de concreto de 80 m de altura, construida en la zona del Bajo Río Balsas, en donde se presentan las condiciones más severas de diseño desde el punto de vista sísmico.

La geometría de la chimenea queda definida por las alturas $H_1 = 202.4$ m y $H_2 = 246.40$ m que son las distancias desde la parte superior de la chimenea, hasta el vértice de los conos que definen el manto exterior e interior respectivamente, del fuste de la chimenea.

El radio inferior externo del fuste se consideró igual a $R_{11} = 4.625$ m, mientras que el radio interior $R_{12} = 4.125$ m en la sección transversal de la base de la chimenea.

Se consideró que el módulo de elasticidad del concreto era igual a $2.5X10^6$ ton/m^2 y su resistencia última $f'_c = 2500$ ton/m^2, con un peso volumétrico de 2.4 ton/m^3 y un módulo de Poisson igual a 0.15.

Se aceptó la existencia de conos de tabique refractario, con un peso volumétrico de 2.3 ton/m^3, mortero con peso volumétrico de 0.55 ton/m^3 y recubrimiento de la cámara de ventilación con peso volumétrico de 2 ton/m^3. Se aceptó que los conos de material refractario tuviesen un espesor de 0.23 m, con capa de mortero de 0.003 m y recubrimiento de 0.065 m. Para todas las cámaras de ventilación, se consideró una separación de 0.07 m entre el fuste y el cono de material refractario.

Se aceptó la existencia de ménsulas rectangulares de soporte de los conos, con dimensiones b=0.365 y h= 0.40, situadas a distancias de 10 m. Esto forzó la idealización de la chimenea como un sistema de masas concentradas, situadas a 10 m de distancia. En la tabla I se muestran las características geométricas y distribución de masas de la chimenea en estudio.

Para obtener la respuesta del análisis dinámico, se seleccionó un espectro con coeficiente sísmico máximo igual a 0.73, con ordenada al origen igual a 0.30; el periodo característico inferior se consideró igual a 0.4 seg y el periodo característico mayor igual a 1 seg; la rama descendente del espectro se consideró representada por un exponente 1.0 y se aceptó un factor de ductilidad igual a 2.0

La respuesta de la chimenea se analizó tomando en consideración el efecto de la fuerza cortante y de la flexión y se despreció el efecto de inercia rotacional. Para el cálculo de las deformaciones de cortante se consideró un factor de forma igual a 1.5.

b) Secuencia de cálculo

El análisis fué realizado mediante un ordenador digital que sigue la siguiente secuencia:

1) Calcula el volúmen de concreto del fuste y de las ménsulas de la chimenea, así como del revestimiento interior, mediante el teorema de Pappus, y con estos datos obtiene la masa, a la cual concentra en las ménsulas, seleccionadas como puntos de concentración de las masas de la chimenea.

2) Calcula, en base a métodos de flexibilidades, la matriz de rigideces de la estructura, usando métodos numéricos

3) Transforma al problema en un problema de valores característicos y define los valores de las frecuencias naturales, las formas características modales y los coeficientes de participación modal mediante el algoritmo de Jacobi.

4) A partir de un espectro de coeficiente sísmico, calcula la respuesta sísmica modal, siguiendo tres criterios diferentes, para el cálculo de la respuesta. En el primer caso, la respuesta la obtiene mediante $R_1 = \sqrt{\sum_{i=1}^{n} R_i^2}$; en el segundo caso $R_2 = \left|\sum_{i=1}^{n} R_i\right|$ y el tercer criterio resulta ser el valor medio de los dos casos anteriores.

5) Grafica las curvas que definen a la configuración deformada máxima, la representativa del diagrama de fuerza cortante y el diagrama de momento flexionante inducidos por la acción sísmica.

c) Resultados obtenidos

Tabla [illegible] Características geométricas y distribución de masa en una chimenea de 80 m de altura

Sección	Radio exterior	Radio interior	Volúmen	Peso	Masa del fuste	Momento de inercia	Masa de la ménsula	Masa del revestimiento
1	3.3148 3.3967	3.1140 3.1772	45.366	100.879	11.099	24.519	0.727	12.[illegible]
2	3.4785 3.5605	3.2403 3.3035	55.432	113.035	13.591	32.674	0.756	12.[illegible]35
3	3.6424 3.7242	3.3667 3.4299	65.178	159.823	15.103	42.393	0.784	13.[illegible]59
4	3.8061 3.8880	3.4931 3.5563	77.607	186.257	18.986	53.847	0.812	14.1[illegible]4
5	3.9699 4.0518	3.6195 3.6827	89.713	215.332	21.949	67.220	0.841	14.[illegible]37
6	4.1337 4.2156	3.7459 3.8091	102.510	246.024	25.079	82.702	0.869	15.174
7	4.2975 4.3793	3.8722 3.9354	115.954	278.361	28.375	100.493	0.897	15.7[illegible]
8	4.4612 4.5431	3.9986 4.0618	130.170	312.335	31.838	120.302	0.000	16.[illegible]

$$
[K] = \begin{bmatrix}
+.789170E+5 & -.154570E+6 & +.671640E+5 & +.121050E+5 & -.193190E+4 & -.150400E+4 & -.340690E+3 & +.105950E+2 \\
-.154570E+6 & +.390750E+6 & -.293700E+6 & +.323110E+5 & +.233215E+5 & +.332730E+4 & +.954420E+3 & -.559150E+3 \\
+.671645E+5 & -.293700E+6 & +.452300E+6 & -.260650E+6 & +.533300E+4 & +.227750E+5 & +.754760E+4 & +.593570E+3 \\
+.121090E+5 & +.323110E+5 & -.260650E+6 & +.433260E+6 & -.233500E+6 & -.117250E+5 & +.155900E+5 & +.601640E+4 \\
-.193190E+4 & +.233215E+5 & +.533300E+4 & -.233500E+6 & +.410430E+6 & -.214930E+6 & -.276610E+5 & +.350920E+4 \\
-.150400E+4 & +.332730E+4 & +.227750E+5 & -.117250E+5 & -.214930E+6 & +.405120E+6 & -.159910E+6 & -.355570E+5 \\
-.340690E+3 & +.954420E+3 & +.754760E+4 & +.155900E+5 & -.276610E+5 & -.159910E+6 & +.419130E+6 & -.124920E+6 \\
+.105945E+2 & -.559150E+3 & +.593570E+3 & +.601640E+4 & +.350920E+4 & -.355570E+5 & -.124920E+6 & +.556410E+6
\end{bmatrix}
$$

b) MATRIZ DE MASAS

$$
[M] = \begin{bmatrix}
.556942E+1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & .255570E+2 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & .286660E+2 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & .319420E+2 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & .353040E+2 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & .339940E+2 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & .427760E+2 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & .467130E+2
\end{bmatrix}
$$

a) Formas características de los modos naturales de vibración

Modo / Masa	Primero	Segundo	Tercero	Cuarto	Quinto	Sexto	Séptimo	Octavo
1	+.142876E+0	-.172315E+0	-.172702E+0	-.149679E+0	-.828470E-1	+.103971E+0	+.131317E+0	+.209272E+0
2	+.121003E+0	-.702738E-1	-.271531E-1	+.875726E-1	+.134676E-1	-.239781E-1	-.469370E-1	-.121570E+0
3	+.936599E-1	+.550104E-2	+.709533E-1	-.713142E-1	+.393975E-1	-.444623E-1	-.350265E-1	+.105249E+0
4	+.678957E-1	+.623951E-1	+.702043E-1	+.785634E-3	-.734921E-1	+.317363E-1	+.112399E+0	+.430010E-1
5	+.449583E-1	+.641899E-1	-.139953E-2	-.751990E-1	-.465734E-1	+.464097E-1	-.930010E-1	+.855370E-2
6	+.259979E-1	+.733409E-1	-.699560E-1	-.319290E-1	-.313164E+0	-.104304E+0	+.423090E-1	+.331354E-3
7	-.117912E-1	+.633170E-1	-.730401E-1	+.722947E-1	+.593516E-1	+.794592E-1	-.652277E-1	-.231274E-3
8	+.332111E-2	+.135540E-3	-.353337E-1	+.771730E-1	-.115900E+0	-.253205E-1	-.910322E-3	-.594255E-4

b) Período natural de vibración de cada modo, en seg

.115460E+1	.742340E+0	.101426E+0	.541932E-1	.540340E-1	.407174E-1	.419152E-1	.331604E-1

c) Factores de participación modal

+.405973E+0	+.117654E-1	+.132206E-2	+.449310E-3	+.273732E-3	-.482421E-3	+.464006E-4	+.152335E-2

SE CALCULA LA RESPUESTA PARA COMBINACION DE 8 MODOS

7

*** DESPLAZAMIENTOS

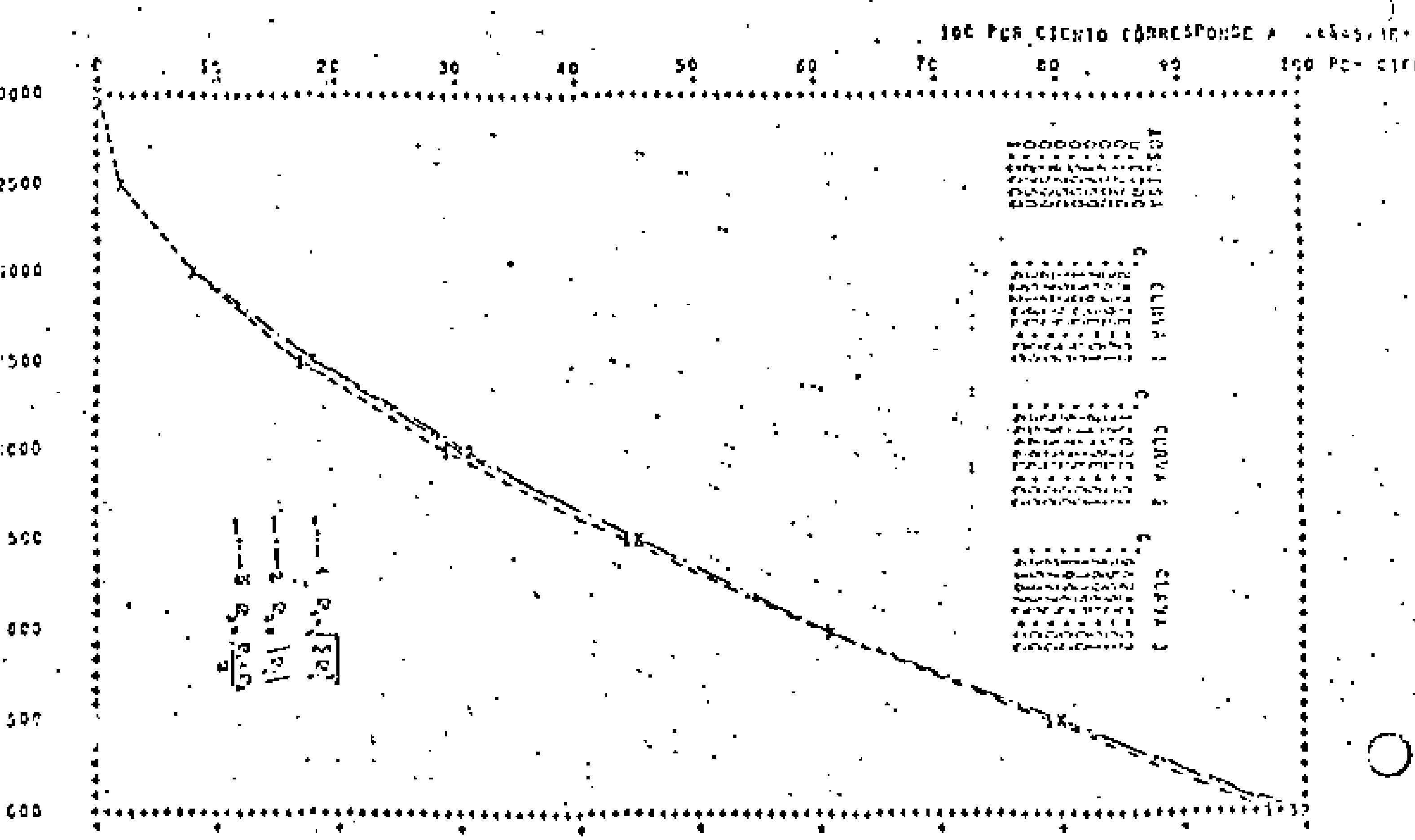
DESPLAZAMIENTOS MAXIMOS MODALES

MO	MASA (1)	. . .	MASA (N)		
1	[illegible]	[illegible]	[illegible]	.197393E+00	.1307[illegible]E+00
2	[illegible]	[illegible]	[illegible]	.389608E-02	.521520E-02
3	[illegible]	[illegible]	[illegible]	-.377968E-03	.761596E-05
4	[illegible]	[illegible]	[illegible]	.128419E-05	-.122445E-03
5	[illegible]	[illegible]	[illegible]	.104637E-04	.366632E-04
6	[illegible]	[illegible]	[illegible]	-.863670E-06	-.675296E-06
7	[illegible]	[illegible]	[illegible]	-.184456E-06	.160428E-06
8	[illegible]	[illegible]	[illegible]	-.195803E-08	.385504E-05

RESPUESTA MODAL

MASA	R1=RC(SUMA(R**2))	R2=ABS(R)	R3=(R1+R2)/2.0
1	[illegible]	[illegible]	[illegible]
2	[illegible]	[illegible]	[illegible]
3	[illegible]	[illegible]	[illegible]
4	[illegible]	[illegible]	[illegible]
5	[illegible]	[illegible]	[illegible]
6	[illegible]	[illegible]	[illegible]
7	[illegible]	[illegible]	[illegible]
8	[illegible]	.100253E-01	.542225E-02

8



VALORES NORMALIZADOS DE LA ALTURA --RELACION DE Z/H

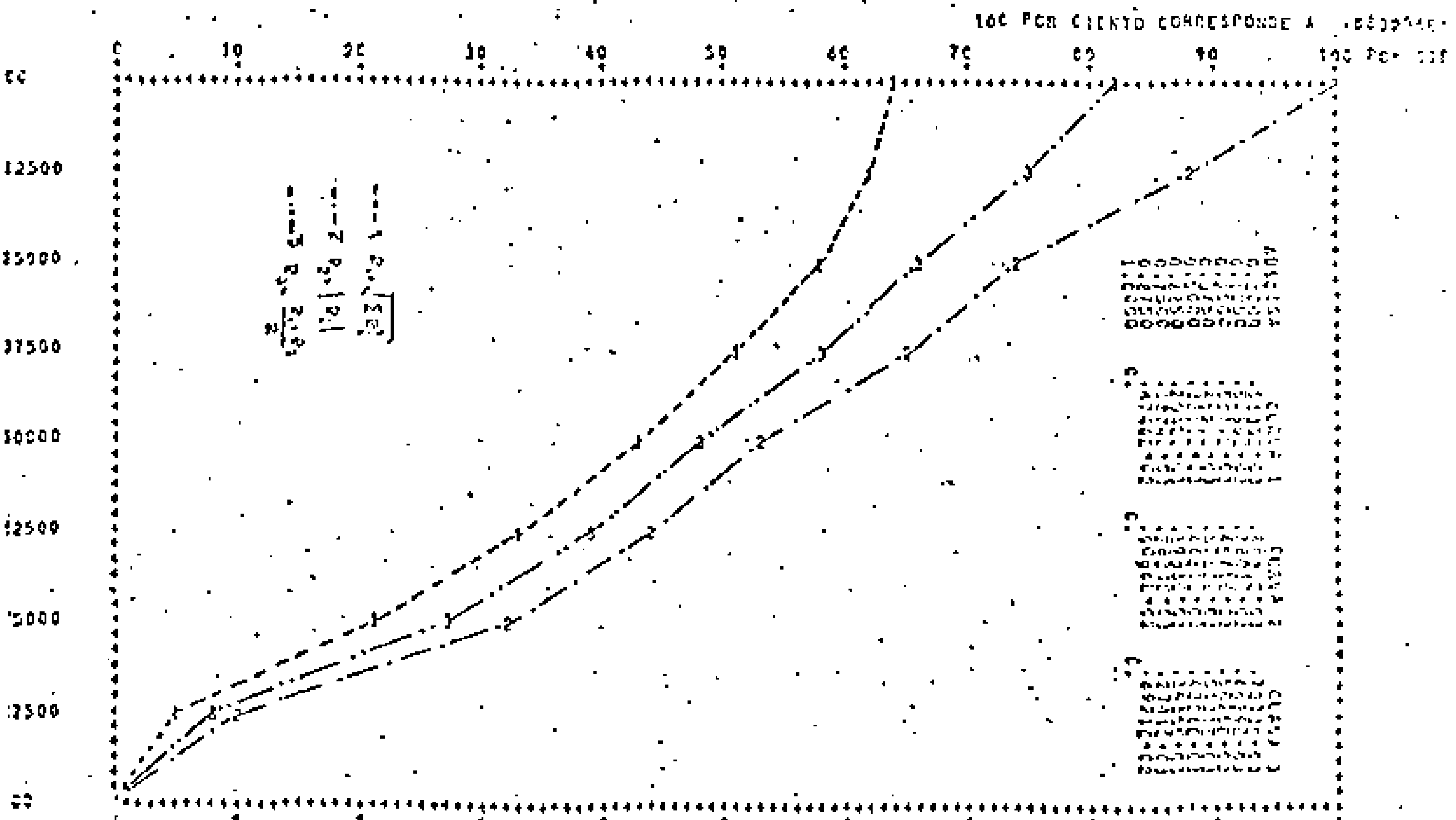
CORTANTES

CORTANTES MAXIMOS

MODO	MASA (1)	. . .	MASA (N)		
1	[illegible]	[illegible]	[illegible]	.755115E+03	.692C63E+03
2	[illegible]	[illegible]	[illegible]	-.324305E+02	.515561E+02
3	[illegible]	[illegible]	[illegible]	-.541370E+02	-.531700E+02
4	[illegible]	[illegible]	[illegible]	.236660E+02	-.175233E+02
5	[illegible]	[illegible]	[illegible]	-.239124E+01	.146558E+02
6	[illegible]	[illegible]	[illegible]	-.956713E-01	-.692133E+00
7	[illegible]	[illegible]	[illegible]	-.780373E-01	.490386E-01
8	[illegible]	[illegible]	[illegible]	-.496697E-03	.400574E-06

RESPUESTA MODAL

MASA	R1=RC(SQRA(R**2))	R2=ABS(R)	R3=(R1+R2)/2.0
1	.474526E+02	[illegible]	.692090E+02
2	[illegible]	[illegible]	[illegible]
3	[illegible]	[illegible]	[illegible]
4	[illegible]	[illegible]	[illegible]
5	[illegible]	[illegible]	[illegible]
6	[illegible]	[illegible]	[illegible]
7	[illegible]	[illegible]	[illegible]
8	[illegible]	[illegible]	[illegible]



VALORES NORMALIZADOS DE LA ALTURA --RELACION DE Z/H

MOMENTOS

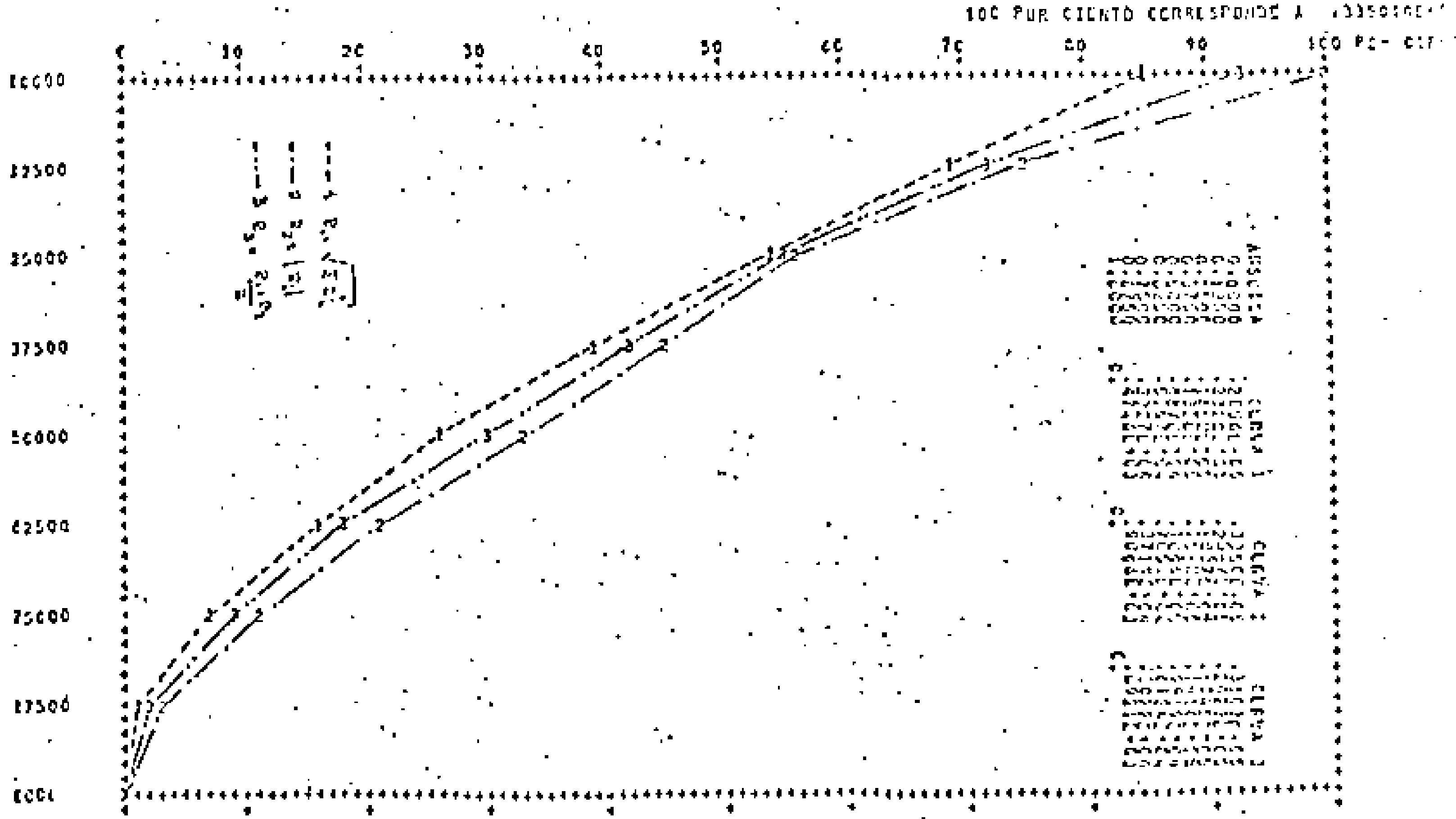
MOMENTOS FLEXIONANTES MAXIMOS

NODO	MASA (1)	. . .	MASA (N)		
1	[illegible]	[illegible]	[illegible]	.173195E+05	.262401E+05
2	[illegible]	[illegible]	[illegible]	-.311492E+04	-.215932E+04
3	[illegible]	[illegible]	[illegible]	-.250151E+02	-.616215E+03
4	[illegible]	[illegible]	[illegible]	.251131E+03	.628554E+02
5	[illegible]	[illegible]	[illegible]	-.764059E+02	.701422E+02
6	[illegible]	[illegible]	[illegible]	.117026E+01	-.374106E+01
7	[illegible]	[illegible]	[illegible]	-.356805E+00	.123572E+00
8	[illegible]	[illegible]	[illegible]	-.255662E-03	-.251693E-03

RESPUESTA MODAL

MASA	R1=RC(SUMA(R**2))	R2=ABS(R)	R3=(R1+R2)/2.0
1	.474500E+03	[illegible]	.492090E+03
2	[illegible]	[illegible]	[illegible]
3	[illegible]	[illegible]	[illegible]
4	[illegible]	[illegible]	[illegible]
5	[illegible]	[illegible]	[illegible]
6	[illegible]	[illegible]	[illegible]
7	[illegible]	[illegible]	[illegible]
8	.285634E+05	.335016E+05	.310322E+05

100 POR CIENTO CORRESPONDE A .335016E+



VALORES NORMALIZADOS DE LA ALTURA --RELACION DE 2/H

DIVISION DE EDUCACION CONTINUA
FACULTAD DE INGENIERIA U.N.A.M.

IX CURSO INTERNACIONAL DE INGENIERIA SISMICA

DISEÑO SISMICO DE ESTRUCTURAS ESPECIALES

DISEÑO SISMICO DE ESTRUCTURAS DE TIPO INDUSTRIAL

PROF. M. EN C. MAURICIO NANES G.

AGOSTO, 1983

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DISEÑO SISMICO DE ESTRUCTURAS DE TIPO INDUSTRIAL

Mauricio Nanes G.*

RESUMEN

Se describen algunas características que presentan las estructuras de tipo industrial por las que difieren en su tratamiento, desde el punto de vista sísmico, de las estructuras de tipo urbano.

Se plantea la problemática existente por falta de reglamentación para estructuras industriales.

* FERMA, Ingenieros Consultores, S.A.

1. CLASIFICACION DE ESTRUCTURAS

Las estructuras para plantas industriales son muy variadas y diferentes entre sí. Pueden haber edificios semejantes a los de tipo urbano, y hay edificios muy irregulares cuya función básica es el albergar el equipo de proceso de la planta. En estos edificios la estructura tiene que adaptarse a las configuraciones y arreglos caprichosos del equipo.

Existen otro tipo de estructuras que no pueden considerarse como edificios, tales como: puentes para transportadores elevados de banda, plataformas de operación alrededor de equipos que se encuentran al descubierto, torres para el almacenamiento de flúidos o gases, estructuras para cama de tuberías, etc.

A continuación se enlistan algunos tipos de plantas industriales:

- Papeleras
- Ingenios Azucareros
- Químicas
- Petroquímicas
- Petroleras
- Cerveceras y Malteras
- De fuerza
- Mineras
- Metalúrgicas
- Manufactureras
- Etc.

En todas estas plantas hay edificios que son básicamente del mismo tipo, como serían los edificios de servicios al personal : oficinas, comedor, baños y vestidores; así como edificios de bodegas, casetas de entrada y/o vigilancia.

Sinembargo, hay edificios que son muy característicos de cada tipo de planta. Algunos ejemplos serían: el edificio de máquina de papel (en Plantas Papeleras), el edificio de fuerza de los Ingenios Azucareros, el edificio de Salas Frías en Cerveceras, edificios de trituración en Mineras, etc.

Estos edificios se caracterizan por estar sujetos a slicitaciones de carga muy elevadas; consecuentemente el aspecto sísmico adquiere una especial importancia.

2. ASPECTOS ESPECIALES DE EDIFICIOS INDUSTRIALES

En forma simplista se puede pensar que todo edificio consta de trabes y columnas, por lo que no tendría ninguna importancia el distinguir un edificio de tipo urbano de otro de tipo industrial.

Existen diferencias importantes en lo que se refiere a cargas y estructuración, que implican un tratamiento diferente.

2.1 Cargas.

En edificios de tipo industrial se tienen que considerar una serie de condiciones de carga que generalmente en edificios urbanos no aparecen; como serían: cargas de grúa en diferentes posiciones y condiciones de izaje, pesos de equipo en condiciones de operación y de prueba, equipos con características vibratorias, variantes en la carga viva para diferentes condiciones de operación y de montaje de equipo, etc.

El número de condiciones de carga para el análisis de este tipo de edificios generalmente es mayor a 8, y en ocasiones pueden llegar a ser hasta doce (12), y el número de combinaciones de carga pueden ser hasta de 25. Las solicitaciones sísmicas generalmente se tienen que considerar como varias condiciones de carga.

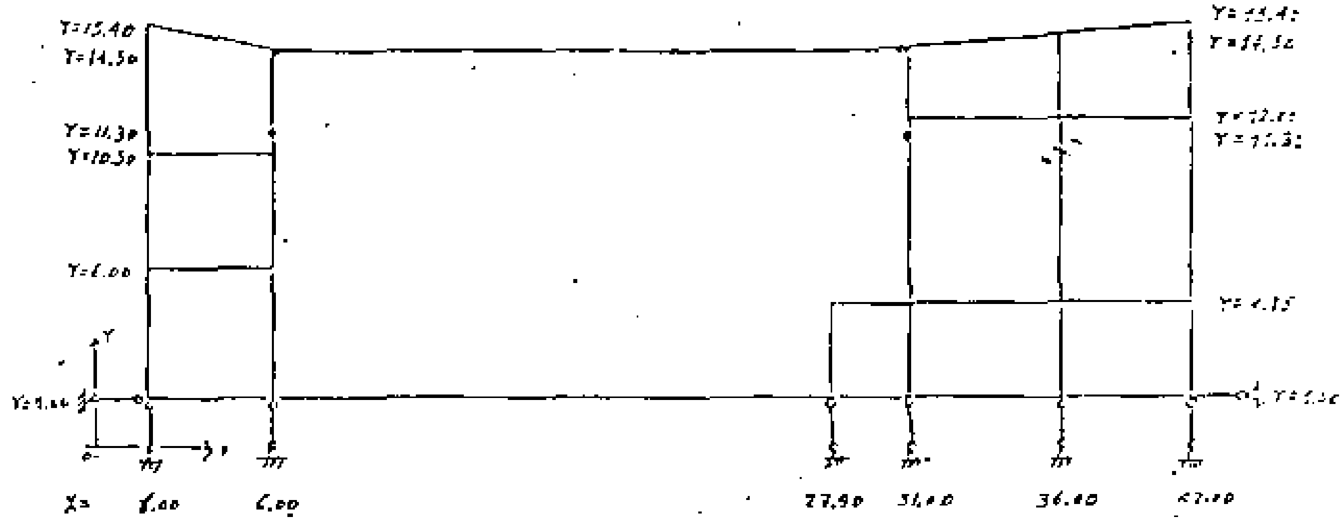
2.2 Estructuración.

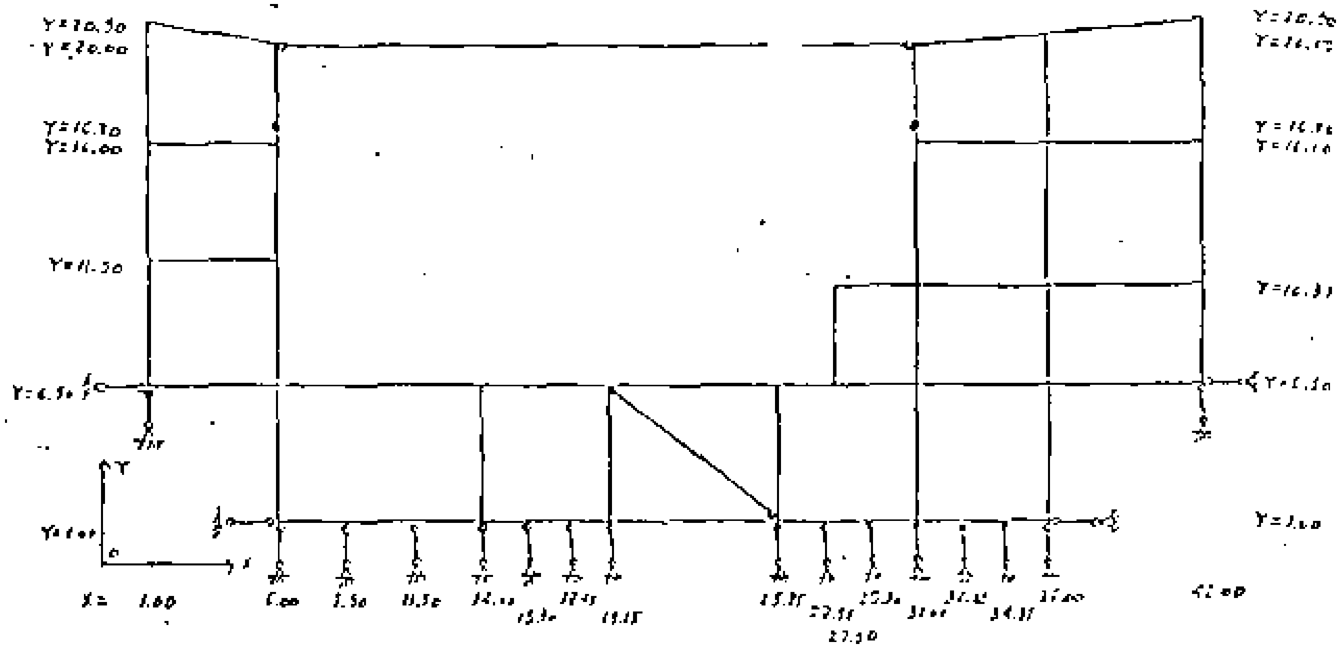
Las estructuraciones que resultan en los edificios de tipo industrial son muy variadas y al mismo tiempo diferentes al las de edificios convencionales. Esto resulta de la necesidad de adaptarse al arreglo de equipo que haya que albergar en el edificio, y que como resultado se obtienen irregularidades importantes de estructuración. A continuación se enlistan algunas características de estructuración de edificios industriales:

- Ausencia de diafragmas rígidos en los sistemas de piso por la presencia de huecos de importancia, así como por la falta de losas de concreto.

- Estructuración no uniforme en planta.
- Estructuración no uniforme en elevación: falta de algunos niveles en ciertas crujías, diferencias en alturas de entrepiso, falta de tramos de columnas en ciertos entrepisos, etc.
- Distribución no uniforme de masas en los pisos y a lo alto del edificio.
- Rigideces de entrepiso muy diferentes de los marcos de un entrepiso y a lo alto de un mismi marco, causadas por la presencia irregular de contraventeo vertical, cambio en la posición del sistema de contraventeo de entrepiso a entrepiso.

Como ilustración se mustra en la siguiente figura dos cortes de un edificio de máquina de papel.





2.3 Criterios de análisis.

Edificios con las características descritas con anterioridad, requieren de criterios de análisis más racionales, eliminando la posibilidad de consideraciones simplificatorias, como en ocasiones se utilizan en edificios de tipo urbano.

Algunos conceptos que deben incluirse en el análisis de edificios industriales son:

- Incorporar dentro del análisis el comportamiento -no rígido- de los sistemas de piso.
- Análisis dinámico sísmico en lugar de un análisis estático equivalente; esto no necesariamente debido a una gran altura de los edificios, sino mas bién por las irregularidades de rigidez y de masa.
- Análisis dinámico sísmico en la dirección vertical.
- Incorporar la interacción suelo-estructura en el análisis.

3. REGLAMENTACION

Los códigos y reglamentos existentes, no solo en México sino en general, están orientados hacia estructuras regulares como las que se presentan en la construcción urbana, y en muchos aspectos no son aplicables a edificios y estructuras de tipo industrial. A falta de la reglamentación correspondiente, se ha intentado extrapolar las especificaciones erroneamente conduciendo a consideraciones peligrosas para las estructuras. Los criterios de análisis descritos con anterioridad, implican una mayor compenetración en los principios básicos en que se fundamentan los reglamentos y hace posible utilizar procedimientos más racionales, primordialmente en lo que se refiere al aspecto sísmico.

Se requiere definir por medio de ciertos parámetros, el sismo para el cual se tiene que efectuar el diseño, que en el caso de un análisis dinámico, serían los que definen el "Espectro de Diseño".

En el Reglamento de Construcciones para el Distrito Federal (México), se definen los parámetros del espectro de diseño para diferentes tipos de suelos. En la reglamentación para construcciones del INFONAVIT, y en otras, se amplia a la regionalización sísmica de todo el país.

Estos "Espectros de Diseño", tienen implicito un porcentaje de amortiguamiento del 5%, y consecuentemente un cierto nivel de daños estructurales aceptables (agrietamiento), que para edificios de construcción urbana puede ser correcto, pero no necesariamente para edificios de tipo industrial. La filosofía básica de los reglamentos, en el diseño de estructuras para resistir sismos, es evitar el colapso aún cuando ocurran pequeños daños a la estructura. Este criterio no puede aceptarse en algunos tipos de plantas industriales y se intenta reflejar en la siguiente clasificación de estructuras.

3.1 Clasificación de las estructuras según su uso

A continuación se clasifican las estructuras, tanto urbanas como industriales basandose en la importancia de las mismas en lo que se refiere al evento que quedara inoperable o inhabitable a consecuencia de un sismo de gran intensidad.

GRUPO A.1

Construcciones urbanas importantes, que en caso de falla por sismo, causarían pérdidas directas o indirectas excepcionalmente altas, tales como:

Centrales telefónicas, estaciones de bombeo y archivos, hospitales, escuelas, estadios, auditorios, templos, salas de espectáculos, estaciones terminales de transporte, monumentos y museos.

GRUPO A.2

Construcciones industriales, cuya falla por sismo causaría una contaminación ambiental tal que pondría en peligro la vida de los habitantes de la región, tales como:

Plantas que generan gases tóxicos como cloro y derivados clorados, insecticidas, etc.

GRUPO A.3

Construcciones industriales cuya falla por sismo causaría un potencial de explosión y/o incendio, tales como refinerías.

GRUPO A.4

Construcciones industriales cuya falla por sismo dejaría inoperantes plantas completas y complejos industriales, tales como:

Cuartos o edificios de control, subestaciones eléctricas, casas de fuerza.

GRUPO B.1

Construcciones urbanas que en caso de falla por sismo causarían pérdidas de magnitud intermedia, tales como:

Comercios, bancos, restaurantes, casa habitación, edificios de departamentos y oficinas.

GRUPO B.2

Construcciones industriales, no incluidas en los Grupos A, cuya falla por sismo causaría una pérdida local, pero que pueden poner en peligro construcciones de los grupos A.2 a A.4.

GRUPO B.3

Construcciones industriales, no incluidas en los Grupos A, cuya fa-la por sismo causaría una pérdida local unicamente, tales como:

Edificios de proceso, bodegas, casetas de entrada e instalaciones exteriores aisladas.

GRUPO C.

Construcciones cuya falla por sismo implicaría un costo pequeño y no pueda causar daños a construcciones de los Grupos A y B, tales como:

Bodegas provisionales, bardas con altura menor a 2.5 m.

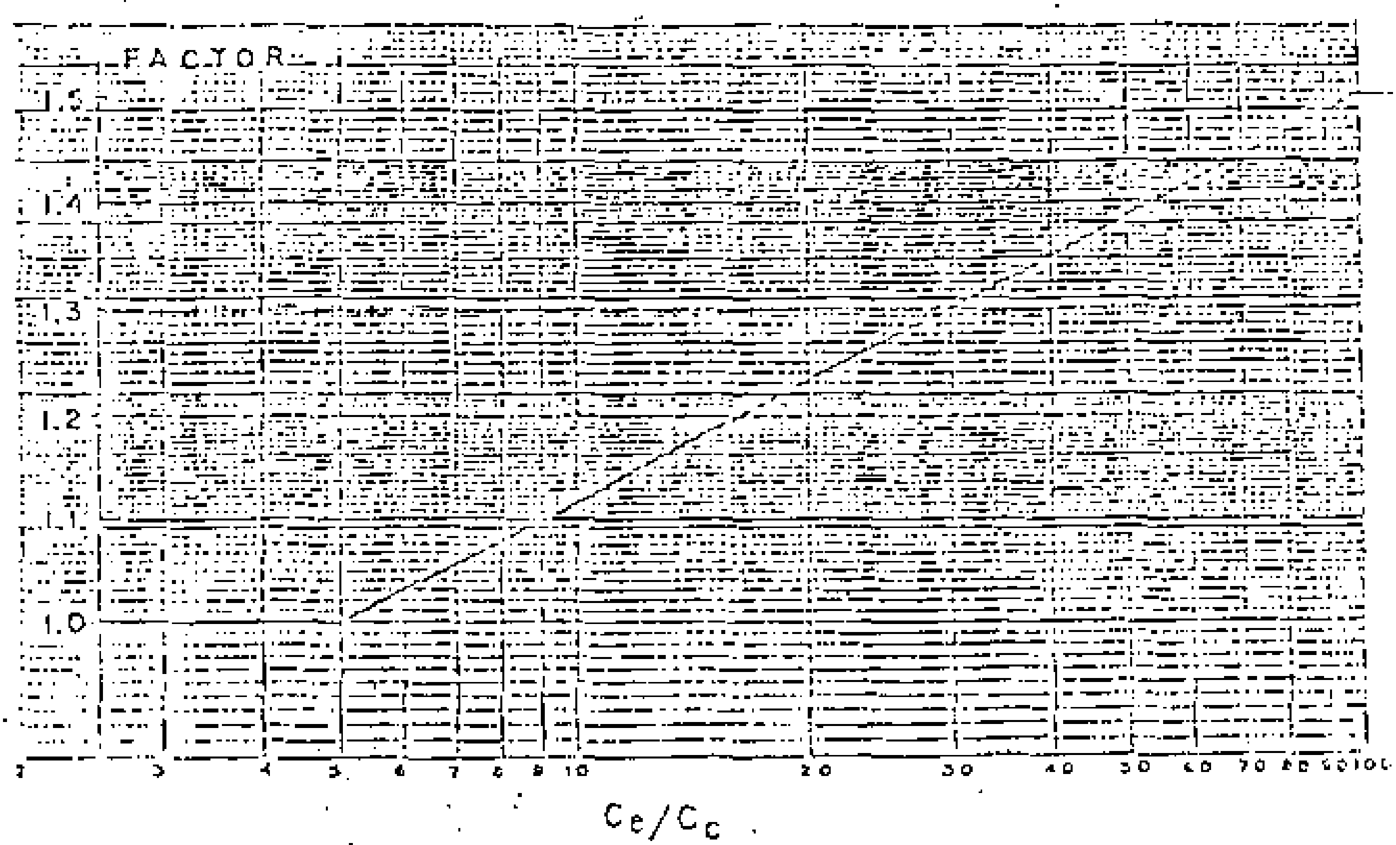
3.2 Afectación al espectro de diseño para diferentes tipos de estructuras.

En la siguiente hoja se presenta como propuesta los coeficientes por los que se debieran multiplicar las ordenadas espectrales máximas en la definición de los "Espectros de Diseño" dependiendo de la clasificación de las estructuras según la sección 3.1 de este artículo.

FACTORES MULTIPLICADORES DE ESPECTROS.

GRUPO	FACTOR
A.1	1.3
A.2	1.5
A.3	1.4
A.4	1.3
B.1	1.0
B.2	1.2
B.3	≥1.0*
C	0

*Para edificios de proceso el "factor" podrá ser mayor que uno (1.0), dependiendo de la relación costo equipo - costo obra civil.



3.3 Ductilidad.

En las reglamentaciones nacionales (México) los espectros de diseño no estan reducidos de acuerdo al efecto favorable de la ductilidad que cada estructura pueda desarrollar, y los coeficientes de ductilidad especificados estan orientados a estructuras de construcción urbana.

A continuación se presentan como propuesta algunos casos de estructuras especiales de tipo industrial no cubiertas por el reglamento:

DESCRIPCION	FACTOR DE DUCTILIDAD
Chimeneas y torres de proceso de acero (un solo "elemento resistente")	2.0
Chimeneas, silos, torres cilíndricas de concreto (un solo "elemento" resistente)	2.0
Péndulos invertidos cuya estructura de soporte de la masa concentrada superior está compuesta de:	
Marcos de acero con o sin contraventeo	4.0
Marcos de concreto	4.0
Cimentación soportante de recipientes horizontales alargados, hornos rotatorios, etc.	
Dirección transversal:	
Muro H/b 7	2.0
Muro H/b 7	3.0
Muro con hueco	4.0
Dirección longitudinal	4.0
Pedestales de concreto de turbos	2.0
Muros de piezas macizas confinados por castillos y dalas	2.0
Muros de piezas huecas confinados o con refuerzo interior	1.5

DISTRIBUCION DE FUERZAS SISMICAS EN EDIFICIOS CON DIAFRAGMAS FLEXIBLES

Mauricio Nanes G*.

En edificios tipo industrial de elementos de acero, y con sistemas de piso de rejilla, la acción de las fuerzas laterales, generalmente debe tomarse, de manera que cada marco absorba, las fuerzas correspondientes al area tributaría.

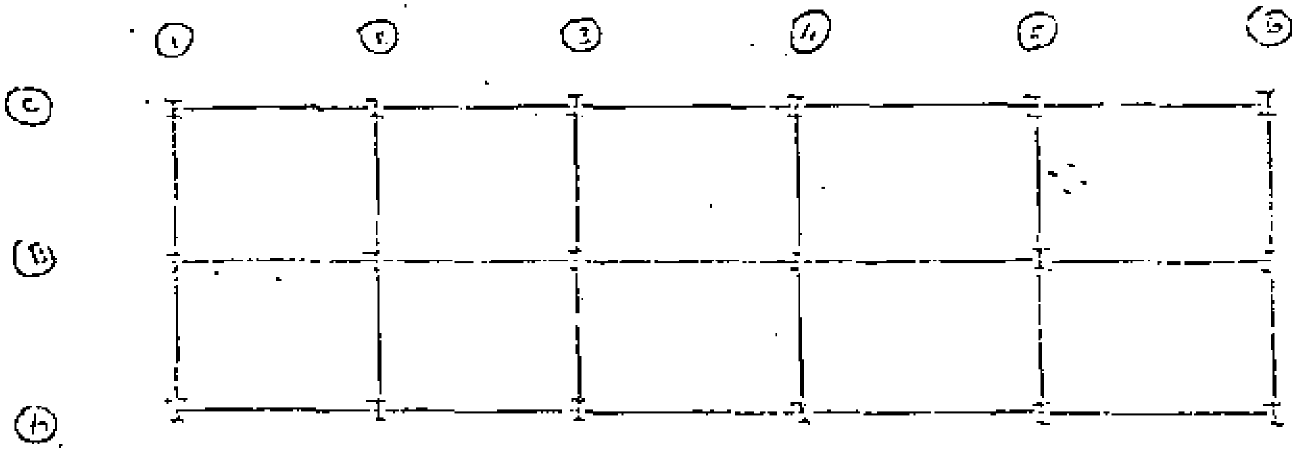
Sin embargo en edificios donde existen concentraciones fuertes de cargas en ciertas areas discretas, conviene forzar la presencia de diafragmas de piso flexibles por medio de contraventeo horizontal en ciertos pisos (o niveles) discretos.

La distribución de fuerzas sísmicas según el procedimiento descrito en el folleto complementario, solo es aplicable a edificios que cada nivel sea un diafragma infinitamente rígido.

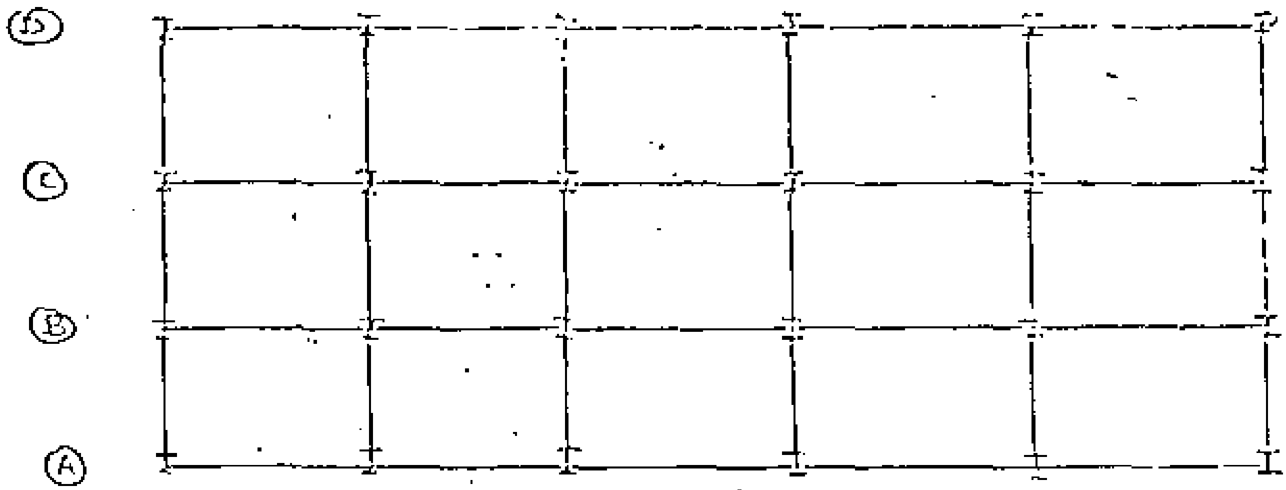
A continuación se describe el procedimiento para determinar las fuerzas sísmicas que actuan en cada marco cuando hay diafragmas flexibles en niveles discretos.

* FERMA, Ingenieros Consultores, S.A.

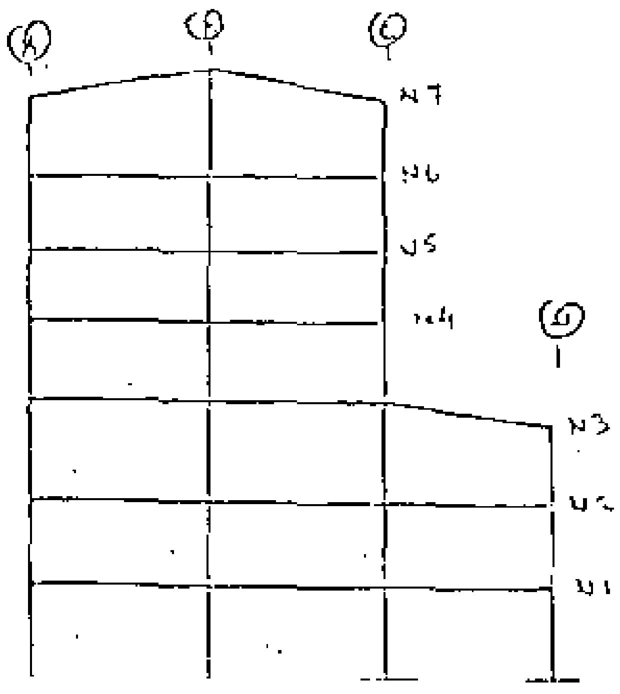
Para visualizar más claramente el procedimiento, se plantea a continuación la siguiente estructura:



PLANTA CUBIERTA -N7-



PLANTA DIAFRAGMA INTERMEDIO -N3-



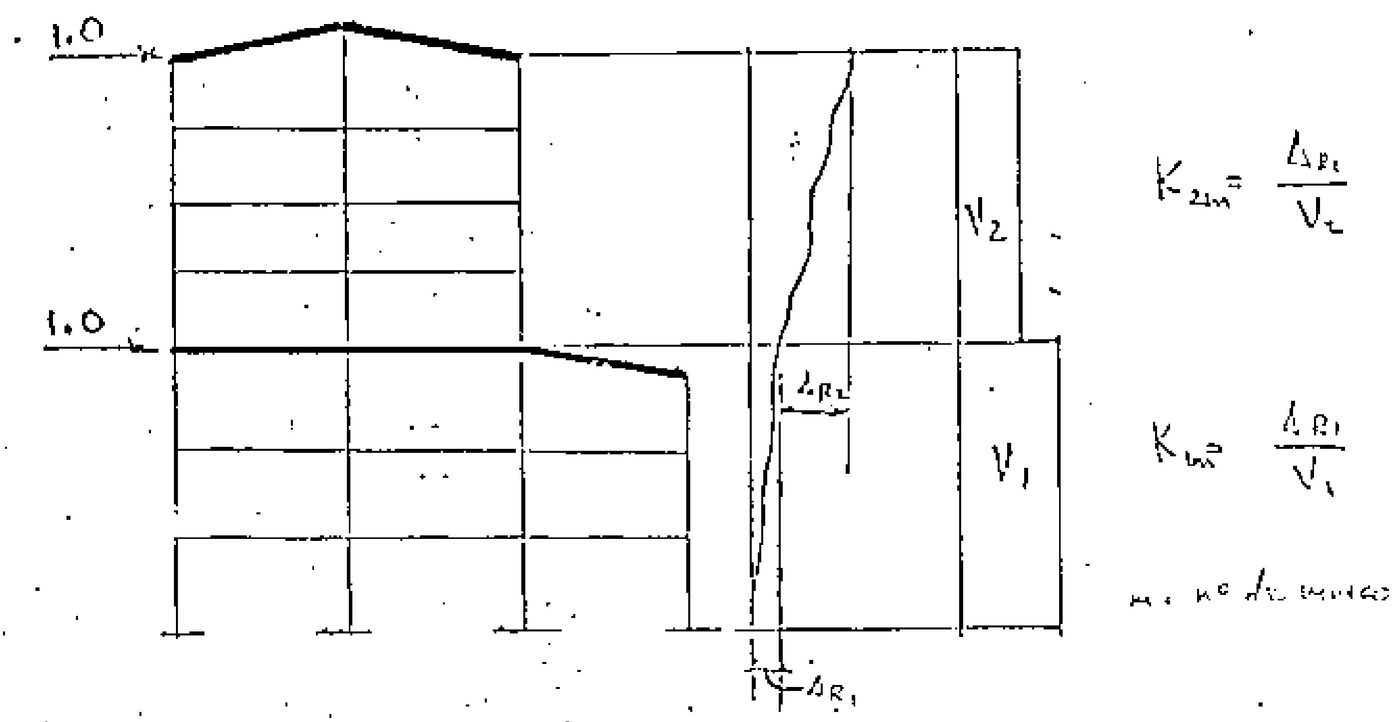
MARCO EJE A - contraventeado
EJE B - ✓
EJE C - contraventeado de N3 a N7

MARCOS EJES ① Y ⑥ - contrav.

SECUELA:

Determinar rígideces de entrepiso, considerando como "entrepisos" unicamente entre diafragmas flexibles:

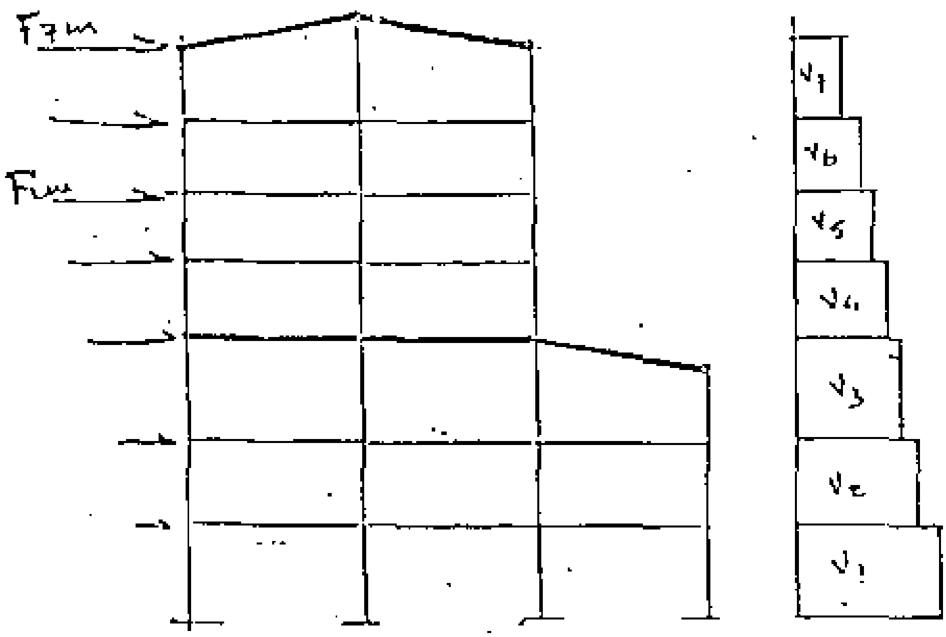
Así por ejemplo para el marco transversal mostrado en la hoja anterior:



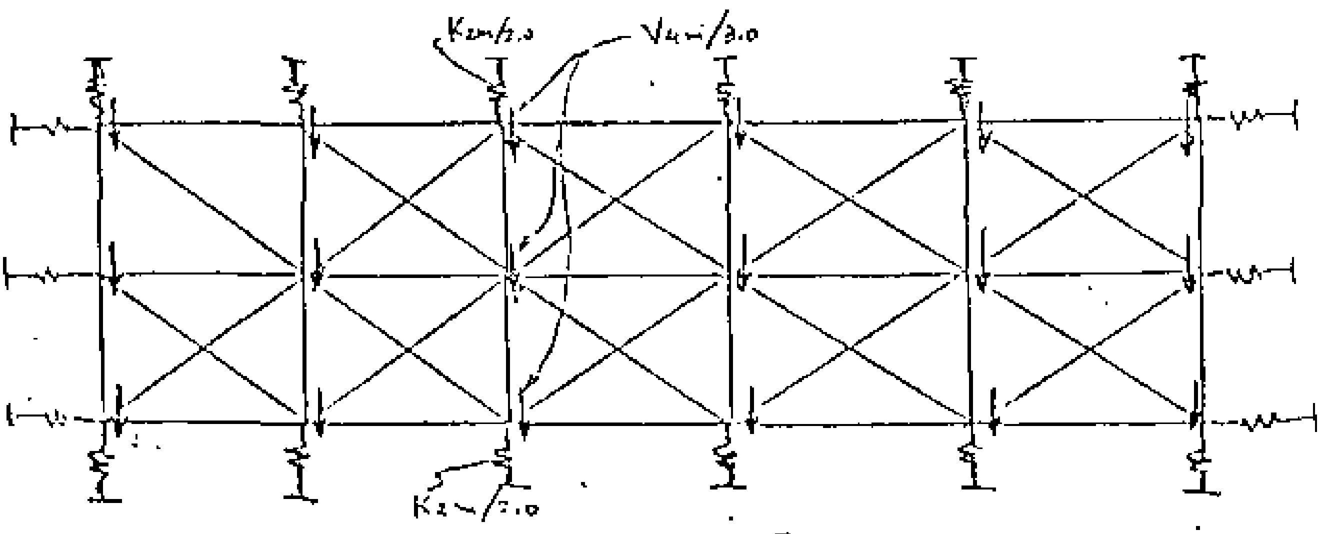
Se determinan así las rígideces de "Superentrepisos" de todos los marcos, tanto transversales como longitudinales, por medio de un análisis de marcos planos con las cargas unitarias laterales a nivel de los diafragmas flexibles.

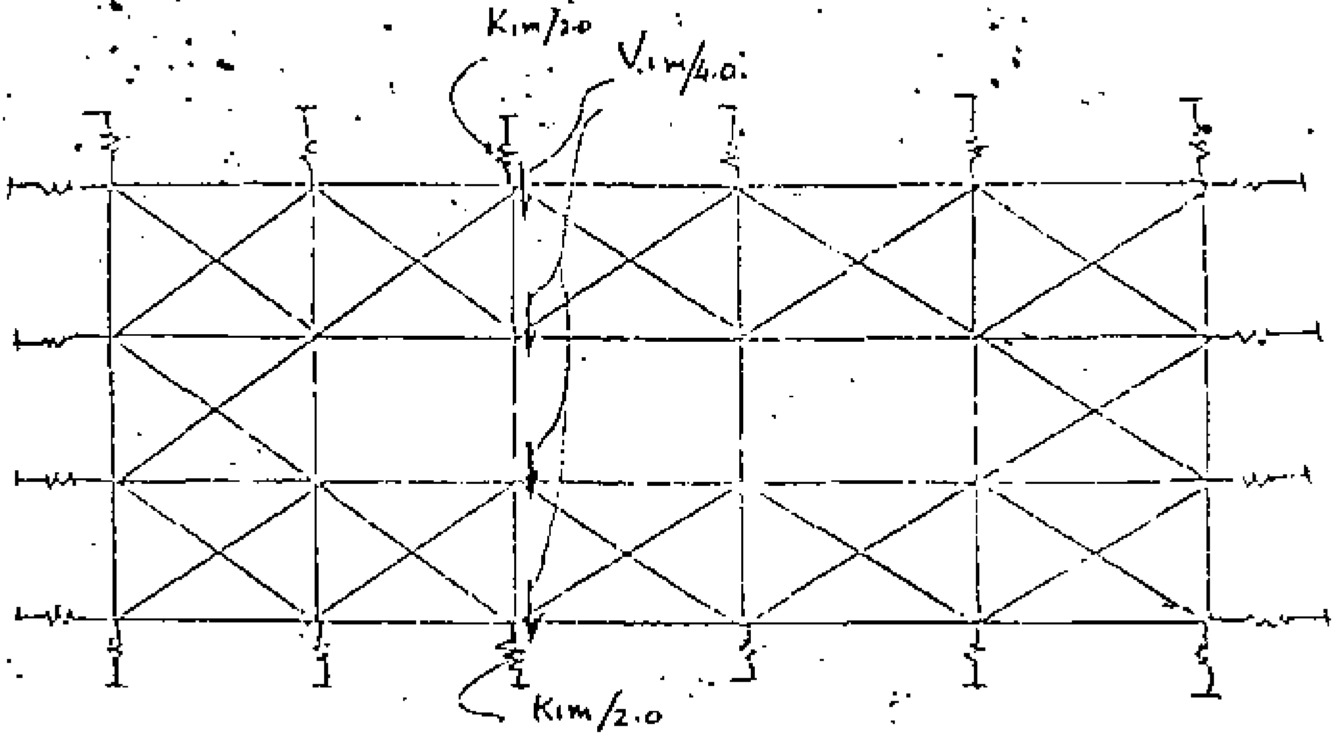
2. Determinar las fuerzas sísmicas laterales de cada marco considerando pesos por area tributaría al marco.

$$F_{im} = c W_T \frac{W_i h_i}{\Sigma W_i h_i}$$



3. Efectuar el análisis de cada diafragma flexible aplicando en cada marco el cortante sísmico entre superentrepisos como fuerzas de análisis.





DIAFRAGMA - N3 -

Del análisis de diafragmas se obtienen las fuerzas en los "resortes" (que simulan las rigideces de "Superentrepisos")

$$- \; F_{R_m} \; -$$

4. Obtener la diferencia de fuerzas cortantes Q de cada marco a nivel del diafragma (de cada diafragma).

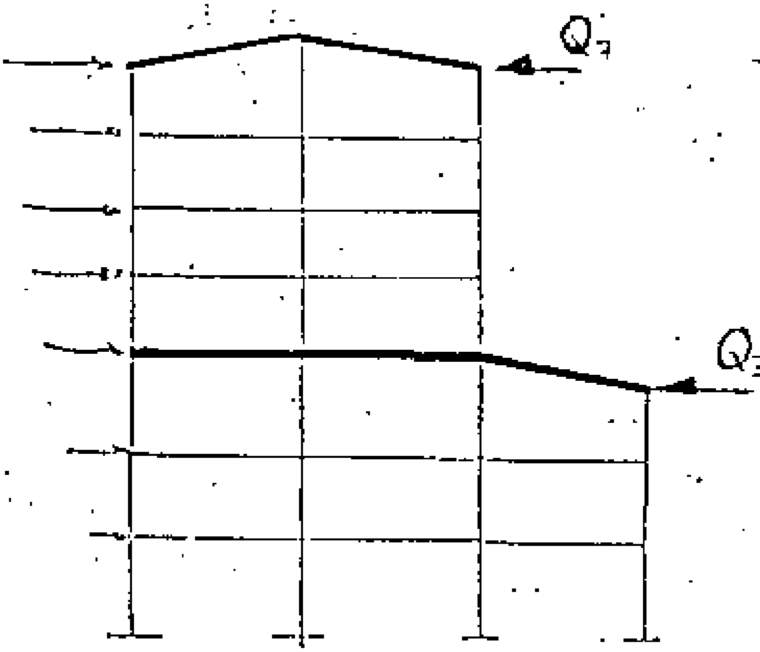
$$Q_{m_7} = F_{R_{m7}} - V4_m \qquad \text{- para el diafragma N7.}$$

$$Q_{m_3} = F_{Rm_3} - V1_m \qquad \text{- para el diafragma N3.}$$

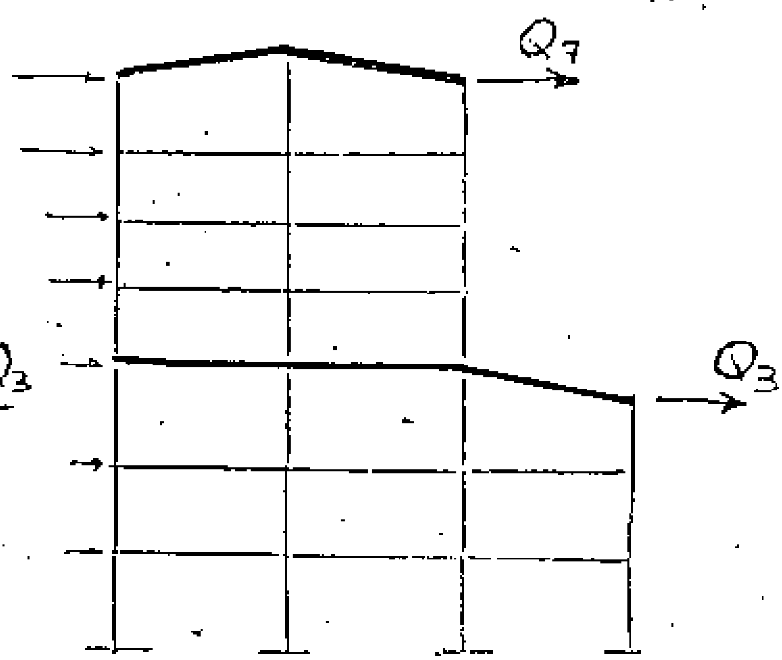
donde F_{R_m} = fza. resultante del análisis de diafragma en el resorte correspondiente al marco _m_.

Qm - Será negativa para marcos flexibles (en contra de las fuerzas sísmicas calculadas por area tributaria).

5. Analizar cada marco (Vertical) en forma aislada aplicandole las fuerzas sísmicas calculadas en base a areas tributaría, y además dos fuerzas concentradas Q (una en cada nivel donde existen los diafragmas) como se ilustra a continuación.



MARCO FLEXIBLE



MARCO RIGIDO

Las fuerzas Q representan el efecto del diafragma sobre el marco; en los marcos flexibles, los diafragmas " detienen " al marco, pero en los marcos rígidos es donde los diafragmas se apoyan e incrementa sus fuerzas laterales.

Utilizar los signos de las $Q^1{}_S$ que resulten de las diferencias de fuerzas anteriores.

ANALISIS DE ESTRUCTURAS CON SISTEMAS DE PISO CONTRAVENTEADOS CONSIDERADOS COMO DIAFRAGMAS FLEXIBLES

Mauricio Manes G.*

RESUMEN

Se describe el comportamiento de estructuras con sistèmas de piso contraventeados, como afectan la distribución de fuerzas sísmicas y de viento. Se plantean modelos estructurales para el análisis de los sistemas de piso con diafragmas flexibles enfocados a satisfacer la compatibilidad de desplazamientos de los marcos transversales producidos tanto por cargas horizontales como por cargas verticales.

Se discuten dos de las funciones básicas del contraventeo horizontal de piso, que son: concentrar fuerzas laterales uniformes en marcos de mayor rigidez y el distribuir fuerzas laterales concentradas entre varios marcos paralèlos. Se describe un estúdio paramétrico en el que se aprecia el efecto de tipos de contraventeo, y su geometría en la concentración o distribución de dichas fuerzas laterales. Finalmente se aplican estos conceptos a Naves Industriales.

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Se ilustro en los puntos anteriores el análisis de diafragmas para sísmo paralelo al ladc corto. Se debe efectuar un análisis semejante para sísmo en la dirección larga (Como otra condición de carga del " marco-diafragma ").

Las fuerzas que se obtengan en las barras diagonales del análisis de diafragma, se pueden utilizar para el diseño del -- contraventeo horizontal.

I N D I C E

1)

¿Como surgió la necesidad de estudiar este tema?

1. Generalmente en edificios de tipo urbano no se duda de que los sistemas de piso actuan basicamente como diafragmas rígidos,

   pero surge la duda de como atacar problemas de distribución de cortantes sísmicos y/o de fuerzas de viento en estructuras cuyos sistemas de piso son metálicos con o sin contraventeos, o bien edificios de concreto con relación de dimensiones en planta muy grandes o con huecos grandes.

2. Otra preocupación que ha existido en edificios industriales que tienen gruas viajeras es:

   como aliviar los esfuerzos en los marcos producidos por las cargas concentradas horizontales de la grua.

3. También ha sido un punto de preocupación la optimización ~~en~~ del diseño de naves industriales.

Existe la tendencia de analizar las estructuras metálicas con sistemas de piso metálico en base a:

marcos planos aislados cargados por area tributaria.

despreciando el comportamiento de conjunto.

1. INTRODUCCION.

El análisis estructural de edificios industriales con sistemas de piso contraventeados presenta una serie de características especiales en lo referente a las condiciones de carga que involucren fuerzas horizontales. El concepto de distribución de cortantes sísmicos del método estático equivalente, no es aplicable tal como se ha planteado (1)*, ya que tiene implícito que los sistemas de piso son indeformables en su plano, y en general el considerar las cargas estáticas equivalentes de viento, calculadas en base a áreas tributarias, es válido en ciertos casos únicamente.

En realidad los sistemas de piso de edificios industriales de acero, en pocas ocasiones pueden ser considerados como indeformables en su plano; por el contrario, resultan ser bastante flexibles, ya sea por la necesidad funcional de dejar huecos de acceso o para equipo, o por la utilización de pisos tipo rejilla o placa antiderrapante. En ocasiones es necesario contraventear el sistema de piso para aumentar su rigidez para lograr una mejor distribución de cargas entre los diferentes marcos.

Este tipo de estructuras, tradicionalmente son analizadas y diseñadas considerando cada marco en forma independiente (por área tributaria), sin tomar en cuenta el comportamiento del conjunto de marcos y sistemas de piso. (En el límite (sistemas de piso sin losa de concreto ni contravientos), los marcos tanto transversales como longitudinales se comportan en forma independiente; sin embargo existe un rango amplio en el que el contraventeo de piso afecta, en menor o mayor grado, la interacción entre marcos paralelos y ortogonales redistribuyendo cargas y alejándose del criterio de considerar las cargas por área tributaria, y sin llegar al caso de diafragmas rígidos.

El objetivo básico es el poder efectuar una serie de análisis de estructuras planas de tal manera que simulen, en la forma más realista posible, el comportamiento tridimensional de la estructura, y lograr de esta manera diseños más económicos.

En este artículo se presentan las herramientas para poder efectuar la distribución de cortantes sísmicos y de viento incluyendo la flexibilidad de los sistemas de piso, se presenta una evaluación cualitativa y cuantitativa del comportamiento de diafragmas flexibles por medio de su modelación y análisis estructurales.

* Números en paréntesis son referencias enlistadas posteriormente.

2. CONCEPTO DE DIAFRAGMA

2.1 Generalidades

Todo entrepiso de un edificio tiene una cierta rigidez a flexión en el plano del sistema de piso en consideración. En el caso de pisos de concreto, estos pueden visualizarse como trabes de concreto muy peraltadas. En el caso de sistemas de piso contraventeados, se pueden considerar como armaduras horizontales. El parámetro importante es la rigidez relativa entre el sistema de piso y la rigidez de los marcos.

En edificios industriales de acero, cuando no existe losa de concreto ni contraventeo horizontal, cada marco se comportará como si estuviera aislado (Fig. 1a); consecuentemente, los desplazamientos laterales, a un cierto nivel de la estructura, de todos los marcos paralelos, serán función de las fuerzas horizontales a que esté sujeto cada marco, consideradas por área tributaria únicamente.

En edificios cuyos sistemas de piso sean de concreto, y cuando la rigidez a flexión en el plano del piso sea grande comparada con las rigideces de entrepiso de los marcos, el sistema de piso puede considerarse como un diafragma rígido, el cual hace que los desplazamientos laterales, a un cierto nivel de la estructura de todos los marcos paralelos, sigan una variación lineal, como se ilustra en la Fig. 1b para el caso que no haya torsión en planta, y en la Fig. 1c cuando sí hay torsión. El diafragma rígido hace que las fuerzas laterales totales se distribuyan a cada marco de acuerdo a sus rigideces de entrepiso relativas.

Los dos tipos de edificios descritos son los casos extremos; sin embargo, existe un gran número de estructuras cuyos sistemas de piso tienen cierta rigidez a flexión, pero no pueden considerarse como diafragmas rígidos. En la Fig. 2 se muestran dos casos de diafragmas flexibles, uno sujeto a cargas simétricas y otro con cargas asimétricas, se muestra también la configuración de desplazamientos laterales, y las fuerzas que absorben cada marco en función de la rigidez de entrepiso K y del desplazamiento lateral en el nivel en consideración.

En estructuras con diafragmas rígidos o flexibles, las fuerzas que absorben los marcos ya no son en función de las cargas laterales calculadas en base a áreas tributarias, sino son tales que deben satisfacer la compatibilidad de desplazamientos laterales de diafragma.

2.2 Funciones del diafragma.

Un diafragma, sea rígido o flexible, hace que las fuerzas horizontales totales en un cierto nivel sean transmitidas a los marcos dependiendo de sus rigideces de entrepiso y de la rigidez del diafragma.

Los dos usos más importantes que se puede hacer del comportamiento de diafragma son:

a. Concentración de fuerzas laterales "uniformes" en los marcos más rígidos.

b. Distribución de fuerzas laterales concentradas entre varios marcos adyacentes a la localización de la fuerza concentrada.

La primera aplicación sería por ejemplo, en edificios industriales en los que se proporciona contraventeo en el plano vertical en los marcos transversales cabecera o exteriores; esto resulta en una diferencia grande de rigideces de entrepiso entre los marcos transversales intermedios y los cabecera.

Las fuerzas sísmicas y de viento se concentrarán en los marcos cabecera a través de los diafragmas de piso. Este es el comportamiento real de la estructura y deberá tomarse en consideración.

La segunda aplicación sería el considerar el diafragma para que la fuerza horizontal sísmica de grúa aplicada en un marco no solo sea absorbida por dicho marco, sino entre varios marcos paralelos.

2.3 Compatibilidad de desplazamientos laterales debidos a cargas verticales.

El comportamiento de diafragma (rígido o flexible) del sistema de piso, forza las condiciones de compatibilidad de desplazamientos laterales de los marcos cuando están sujetos tanto a fuerzas horizontales como a cargas verticales.

El análisis de estructuras compuestas por marcos planos de marcada diferencia geométrica, sujetos a cargas verticales, conduce a desplazamientos laterales distintos en cada marco analizado aisladamente (Fig. 3). En este caso también debe considerarse el comportamiento diafragma para forzar la compatibilidad de desplazamientos.

Una forma de lograr dicha compatibilidad para el caso particular que se tenga diafragmas rígidos y que no exista torsión en planta, se presenta en el modelo estructural de la Fig. 4. Se modelan todos los marcos paralelos una a continuación de otro unidos con elementos ficticios de rigidez infinita que simulan la acción del diafragma rígido, y forzan a que los desplazamientos horizontales de todos los marcos en cada nivel sean iguales. En este caso las propiedades geométricas y las cargas para el modulo del marco 1Y-5Y y 2Y-4Y deben duplicarse.

Cuando el diafragma no se puede considerar como rígido, y para el caso en que no haya torsión en planta, se puede modelar a la estructura como se indica en la Fig. 5, en la que las barras de unión entre marcos tienen una rigidez finita obtenida a través de un análisis del diafragma flexible.

3. MODELACION Y ANALISIS DE DIAFRAGMAS FLEXIBLES.

Un sistema de piso de un edificio industrial diseñado como estructura metálica y que esté contraventeado en su plano, será un diafragma flexible. Este diafragma puede considerarse como una armadura en el plano horizontal soportada sobre una serie de soportes elásticos, los cuales simulan las rigideces del entrepiso inmediatamente abajo del piso en consideración.

En la Fig. 6 se muestra el modelo estructural de dicho diafragma. Las barras verticales y horizontales en las líneas de los ejes del edificio, representan las vigas de los marcos. Todas las diagonales y otras barras son parte del contraventeo horizontal del sistema de piso. La rigidez de los resortes mostrados, son las rigideces de entrepiso calculadas como se indica esquemáticamente en la figura.

A este modelo estructural se le aplican las cargas correspondientes al efecto que se quiera analizar, sea concentración o distribución de fuerzas sísmicas o de viento. Se obtiene del análisis las fuerzas en los resortes, que a su vez son las cargas horizontales que se aplican posteriormente a cada marco para ser analizados como estructuras planas. También se obtienen las fuerzas actuantes en el contraventeo para efectuar su diseño por resistencia.

En edificios industriales de acero los sistemas de piso a veces pueden contraventearse uniformemente como se muestra en la Fig. 7, pero muchas veces resultan diafragmas flexibles tan irregulares como el mostrado en la Fig. 8. Para estos dos casos particulares se muestran los porcentajes obtenidos de las fuerzas totales que absorben los resortes y se comparan con los porcentajes que corresponderían al caso particular de diafragma rígido.

En cierto tipo de industrias, los edificios de proceso de concreto reforzado, requieren de huecos grandes en varios niveles para alojar equipo como se indica en la Fig. 9. Existiría la duda si se puede o no considerar a estos pisos como diafragmas rígidos. En la Fig. 10 se muestra el modelo estructural en forma esquemática, como un marco cerrado con tramos de sección variable en forma escalonada, soportado sobre resortes elásticos; el modelo estructural detallado se muestra en la Fig. 11, el cual fué analizado (2) como diafragma flexible y las fuerzas que se obtuvieron en los resortes se aplicaron posteriormente como cargas horizontales a los marcos planos.

Todo sistema de piso puede ser modelado y analizado, sea como armadura o como placa (elementos finitos) o como modelo simplificado.

4. DIAFRAGMA COMO ELEMENTO CONCENTRADOR DE FUERZAS LATERALES UNIFORMES.

4.1 Estudio Paramétrico.

En ciertas ocasiones es conveniente el acentuar la diferencia de rigideces de entrepiso entre marcos paralelos, para aliviar a los marcos menos rígidos de esfuerzos producidos por cargas laterales, y concentrar en los marcos más rígidos mayor porcentaje de la carga lateral total de un cierto nivel.

Una condición óptima sería el diseñar a cierto número de marcos intermedios para cargas verticales, y con estas secciones de miembros calcular la intensidad de las fuerzas laterales que dichos marcos resistirían con un sobreesfuerzo del 33%. La diferencia entre la fuerza horizontal total y la absorbida por los marcos intermedios, deberá ser absorbida por los marcos más rígidos en los que se quiere concentrar las fuerzas laterales.

Un caso particular sería el de naves industriales en las que se contraventea los marcos exteriores y se procura que todos los marcos intermedios no sean penalizados por absorber cargas laterales.

Se efectuó un estudio paramétrico (3) de la concentración de carga uniforme lateral en los marcos exteriores a través de varias configuraciones y rigideces de diafragmas flexibles.

Los parámetros que intervienen en este estudio son los siguientes:

a. Tipo de contraventeo.- Se consideraron cuatro configuraciones geométricas mostradas en las Figuras 12 y 13.

b. Geometría de contraventeo.- Se consideró un rango amplio de propiedades geométricas de las secciones transversales de contraventeo (Area, momento de inercia).

c. Rigideces de entrepiso.- La rigidez de entrepiso de los marcos intermedios considerada fué de 42 T/M y 420 T/M. Los marcos exteriores se consideraron cinco y cincuenta veces más rígidos que los intermedios.

d. Dimensiones en planta del diafragma.- Tres relaciones de lado largo a corto (A/B) fueron consideradas, A/B=1.5, 2.2 y 3.0.

El modelo estructural utilizado en el análisis fué simplificado de una armadura plana, a un marco cerrado cuyas barras tienen asociadas propiedades geométricas de área y momento de inercia equivalente a las de la armadura. En las Figuras 12 y 13 se muestran los dos modelos superpuestos para los cuatro tipos de contraventeo. Esta equivalencia fue hecha con el objeto de reducir el número de nudos y miembros en el modelo de análisis para reducir tiempo de computadora (2).

Las cargas a las que se sujetó el modelo fueron fuerzas concentradas en todos los nudos simulando una carga uniforme como sería la de sismo o viento.

En las Figuras 14 a 16 se muestra como afectan los diferentes parámetros al porcentaje de la fuerza horizontal total que absorben los marcos exteriores o cabecera (%CA). Se entiende por geometría (abscisas) en esta curvas, los valores relativos de áreas y momentos de Inercia del contraventeo, tomando como valor unitario o punto de partida los mostrados en la Figura 17.

Se observa en estas curvas que a medida que se robustece el contraventeo (aumento de geometría), aumenta el porcentaje de fuerza (%CA) que absorbe el marco transversal exterior contraventeado. A mayor rigidez de entrepiso de los marcos intermedios Kmi, se requiere mayor geometría de contraventeo para alcanzar el mismo % CA.

También se observa que para mayores relaciones de rigideces de entrepiso de marco exterior a interior (Kme/Kmi), se logra una mayor concentración de fuerza en los marcos cabecera (%CA), o bien si dicha relación Kme/Kmi es baja, o sea que las rigideces de entrepiso de todos los marcos es más uniforme, habrá menos concentración de fuerza en los marcos cabecera (% CA).

La condición de diafragma rígido es el límite superior de estas curvas. Se observa que este límite es alcanzado más rápidamente cuando los marcos intermedios tienen menor rigidez; o sea, un diafragma menos rígido (menor geometría) es suficiente para lograr la condición de diafragma rígido. A medida que aumenta la rigidez de los marcos intermedios (Kmi), se requiere de una mayor geometría (diafragma más robusto) para alcanzar la condición límite de diafragma rígido.

El tipo de contraventeo no es muy importante para diafragmas con A/B bajos y con marcos intermedios de baja rigidez (Fig. 14). Pero a medida que aumenta la relación de dimensiones en planta A/B, y que aumenta la rigidez de los marcos intermedios (Kmi), los contraventeos tipo 3 y 4 son los más eficientes, como se aprecia en las Figuras 15 y 16.

Es conveniente reducir el número de parámetros que intervienen en las Figuras 14 a 16, para esto se calcularon las rigideces de los diafragmas (Kd) en función de la geometría y del tipo de contraventeo. Estas rigideces se muestran en la Fig. 18.

Finalmente, los resultados de este estudio paramétrico se condensan en las Figuras 19 y 20. En la abscisa se tiene la relación adimensional: rigidez de diafragma a rigidez de marco intermedio (Kd/Kmi), y en las ordenadas el porcentaje de la carga total absorbida por uno de los dos marcos cabecera (%CA). Se muestran dos curvas para diferentes relaciones de rigidez de marcos exteriores o cabecera a marcos intermedios (Kme/Kmi).

4.2 Distribución de fuerzas sísmicas.

Una de las funciones de los diafragmas es el concentrar fuerzas laterales "uniformes", en los marcos de mayor rigidez. Las fuerzas sísmicas son función de la masa la cual, en general, puede considerarse como una fuerza lateral "uniforme", excepto cuando haya pesos concentrados de consideración.

Cuando los sistemas de piso pueden considerarse como diafragmas rígidos, la fuerza sísmica total se distribuye a los marcos únicamente de acuerdo a sus rigideces relativas de entrepiso tal como se indica en la Figura 21 (1).

En estructuras cuyos sistemas de piso sean diafragmas flexibles, habrá que efectuar primero un análisis de diafragma en cada nivel aplicándole la fuerza sísmica total en el nivel en consideración como fuerzas concentradas en todos los nudos y cuyas intensidades sean tales que su resultante quede localizada en el centro de masas del nivel. De este análisis se obtienen las fuerzas en los resortes, las cuales se aplican posteriormente a los marcos como cargas para efectuar su análisis como estructuras planas independientes.

4.3 Distribución de fuerzas de viento.

Las cargas estáticas equivalentes de viento también pueden considerarse como fuerzas "uniformes". Los sistemas de piso de los edificios, al comportarse como diafragmas rígidos o flexibles, hacen que las fuerzas de viento también se distribuyan entre los marcos de acuerdo a sus rigideces relativas de entrepiso y dependiendo también de la rigidez del diafragma.

Análogamente a las fuerzas sísmicas, las fuerzas de viento no pueden considerarse por área tributaria, sino se debe tomar en cuenta el comportamiento de diafragma de los sistemas de piso. La excepción sería el caso de diafragmas rígidos y marcos de igual rigidez de entrepiso, en el que si se puede distribuir las fuerzas de viento por área tributaria.

A continuación se describe la secuela para distribuir las fuerzas de viento:

a. Aplicar las fuerzas laterales totales de viento (P) en cada diafragma calculada en base a área tributaria definida como una franja horizontal a lo ancho del edificio, con una altura igual a la del promedio de los entrepisos adyacentes (Fig. 22).

b. Por medio de un análisis de diafragma flexible, obtener las fuerzas Fi que absorben los resortes que simulan las rigideces del entrepiso inferior al diafragma. En el caso particular de diafragma rígido sin torsión:

$$Fi = \frac{ki}{\Sigma ki} P$$

c. Calcúlese la fuerza concentrada de nudo en base a área tributaria:

$$Fi \Big]\, t = P_{pr.}\ H\ \ S$$

d. Obténgase la diferencia entre estas dos fuerzas concentradas:

$$Qi = Fi - Fi \Big]\, t$$

Esta fuerza diferencial es la que, a través del diafragma, se está distribuyendo hacia otros marcos. Para marcos más rígidos Qi será positiva, y para marcos menos rígidos Qi será negativa.

e. Efectuar el análisis definitivo de cada marco plano en forma aislada sujeto a presiones calculadas en base a área tributaria como franja vertical, y aplicándole en cada nivel las fuerzas concentradas correctivas como se indica en la Figura 23.

5. DIAFRAGMA COMO ELEMENTO DISTRIBUIDOR DE FUERZAS LATERALES CONCENTRADAS.

5.1 Estudio Paramétrico

En edificios industriales de proceso hay necesidad de transportar cargas pesadas de un lugar a otro por medio de una grúa viajera, ya sean cargas del producto terminado, o bien de equipo en etapa de instalación y/o mantenimiento.

La fuerza sísmica horizontal de grúas de alta capacidad puede llegar a ser la condición de carga que rija el diseño de los marcos transversales. Esta fuerza concentrada horizontal puede distribuirse entre varios marcos paralelos a través del diafragma de manera que se reduzcan sus efectos en el diseño.

Se efectuó un estudio paramétrico (3) de la distribución de fuerzas laterales concentradas a varios marcos paralelos a través del diafragma. Los parámetros que se hicieron intervenir en este estudio son los mismos que en el estudio de concentración de fuerzas laterales descrito en la sección 4.

El modelo estructural utilizado fué simplificado aún más que el marco cerrado equivalente a la armadura horizontal. Se consideró una viga continua apoyada en soportes elásticos con rigidez lineal únicamente, los cuales simulan las rigideces de entrepiso de los marcos. La rigidez de la viga se igualó a la rigidez del diafragma Kd que a su vez se correlaciomó con el tipo de contraventeo y su geometría (Fig. 18).

En la Figura 24 se muestra el modelo estructural así como la variación de la relación de rigideces de trabe equivalente a diafragma (EI/Kd) con respecto a la longitud total de la trabe. Conocida la rigidez del diafragma Kd y la longitud L se puede definir el momento de inercia de la trabe equivalente.

Se consideraron dos condiciones de carga en este estudio paramétrico, la primera aplicando una carga concentrada en el centro de la viga (coincidiendo con un resorte), la segunda aplicando una carga concentrada en el extremo de la viga, simulando la grúa en una posición central y en otra posición extrema.

En la Figura 25 se muestran los resultados de este estudio paramétrico. Las ordenadas son porcentajes de la fuerza concentrada que absorbe el resorte que coincide con el punto de aplicación de la carga; o sea, que del 100% de la fuerza aplicada, el marco absorbería únicamente los porcentajes indicados en la Figura. Las abscisas son valores del momento de inercia de la viga equivalente.

Se observa de la Figura 25, que para un mismo diafragma (valor de la abscisa), el porcentaje de carga que absorbe el marco es mayor entre mayor sea su rigidez de entrepiso. O bien, para poder obtener un mismo porcentaje de carga que absorba un marco (ordenada) se deberá hacer más robusto el diafragma (aumentar I) a medida que aumente la rigidez de entrepiso del marco.

También se observa de la Figura 25 que para diafragmas más rígidos (I mayor a 1), cuando la carga concentrada está en el extremo del edificio, el porcentaje de carga %P que absorbe el marco en el que coincide la carga, es mayor al porcentaje de carga %P que absorbe el marco cuando la carga está aplicada al centro del edificio. Sin embargo para diafragmas muy flexibles (Imenor a 0.1) el efecto puede llegar a invertirse.

Es conveniente reducir el número de parámetros que intervienen en la Fig. 25 de manera de obtener una gráfica con mayor utilidad práctica. Todas las curvas de la Fig. 25 se transformaron a dos, al graficar en las abscisas el parámetro adimensional Kd/Kmi (relación de la rigidez del diafragma a la rigidez de entrepiso de los marcos). De esta figura, conocidas las rigideces de diafragma y del marco típico, se puede obtener el porcentaje de la fuerza concentrada aplicada (%P) que absorbe el marco en consideración. Este porcentaje para diafragmas medianamente contraventeados es menor del 30% para carga al centro del edificio, y menor de 40% para carga en el extremo del edificio.

5.2 Distribución de fuerzas concentradas.

Es estudio paramétrico descrito en la sección anterior puede utilizarse para determinar el porcentaje de la fuerza concentrada horizontal P que absorbe el marco que coincide con la posición de la carga. Este marco es el de interés, ya que otros paralelos absorben un porcentaje de carga menor, y siendo la carga concentrada la producida por grúa, al cambiar la posición de la grúa a otro marco paralelo adyacente, se obtendría prácticamente el mismo valor de %P.

El hecho que el %P que absorbe el marco en el que coincide la carga concentrada horizontal permanece prácticamente constante, independientemente de cual de los marcos centrales es el que se carga, será cierto a partir de un cierto número mínimo de marcos paralelos, y de la rigidez relativa entre diafragma y marcos: lo cual también puede apreciarse de la configuración de desplazamientos laterales del diafragma.

En la Figura 27 se muestran dos configuraciones de desplazamientos. En el croquis superior de la Figura se aprecia que el diafragma es bastante rígido y alcanza a distribuir la fuerza entre todos los marcos. En el croquis inferior el diafragma es más flexible y distribuye la carga entre menos marcos.

Para diafragmas más rígidos, a mayor número de marcos entre los que se distribuye la carga, menor será el porcentaje del marco más cargado %P, pero en diafragmas muy flexibles el valor máximo de %P depende menos del número de marcos.

En el caso límite de diafragma rígido, cuando las rigideces de todos los marcos son iguales, y para una carga simétrica:

$$\%P = \frac{P}{n}$$

Donde: P = Carga concentrada horizontal.
n = Número de marcos.

Cuando la carga está aplicada en el extremo de un diafragma rígido, se puede obtener el porcentaje de dicha carga que absorbe el marco extremo (%P) como la suma de la componente rotacional. En la Fig. 28 se muestran dichos valores para diferente número de marcos (n).

6. APLICACION A NAVES INDUSTRIALES CON GRUA.

6.1 Estructuración.

La estructuración de una nave industrial se muestra en forma esquemática en la Fig. 29. Consta de marcos a dos aguas orientados en la dirección corta o transversal de la nave con claros que pueden ser de 10 o 30 metros o mayores, y cuya separación generalmente está entre 4 a 7 metros o mayor, dependiendo del material de la cubierta y los largueros que la soportan.

Las vigas inclinadas de los marcos soportan a los largueros, que a su vez soportan la cubierta de lámina de asbesto cemento o lámina metálica acanalada. La separación de los largueros es función del material de la lámina de cubierta. Debido a la inclinación de los largueros, se proporcionan tirantes para reducir su flexión en el plano de la cubierta.

Los dos marcos longitudinales se contraventean en crujías discretas, ya que las columnas de sección I están orientadas de manera que el menor momento de inercia coincide con la dirección longitudinal.

En la dirección transversal, los marcos cabecera o exteriores, se consideran en este caso que también están contraventeados.

La cubierta de este tipo de estructuras siempre se contraventea; sin embargo la configuración del contraventeo depende de la función que sea necesario que desarrolle. En la Fig. 30 se muestran dos tipos de contraventeo. La función básica del contraventeo Tipo A es para reducir la longitud libre de pandeo lateral de la viga del marco. La función básica del contraventeo Tipo B es el concentrar o distribuir fuerzas horizontales entre los marcos transversales y disminuir los desplazamientos horizontales relativos entre dichos marcos. El diafragma flexible queda compuesto por todo el sistema de contraventeo de la cubierta.

6.2 Compatibilidad de desplazamientos laterales.

6.2.1 Cargas verticales.

Particularizando al caso de cargas verticales de Grúa, se tendrá una fuerza lineal y un momento a nivel de ménsula o cambio de sección de la columna. La máxima asimetría se logra moviendo el carro de la grúa hasta un extremo del puente. En la Fig. 31a se muestra un marco transversal con las cargas de grúa y desplazamiento lateral Δg.

El mismo marco se carga con una fuerza horizontal unitaria y se obtiene su desplazamiento lateral Δ_1 (Fig. 31b), con el objeto de evaluar la fuerza horizontal (F_h) a nivel de la cubierta que produciría el mismo desplazamiento horizontal que producen las cargas verticales (Δg); o sea:

$$F_h = \frac{\Delta g}{\Delta_1} \times 1$$

El marco de la Fig. 31a se analizó en forma aislada para obtener el desplazamiento Δg, sin embargo este desplazamiento se debe reducir ya que el marco no se encuentra aislado, sino a través de la cubierta (diafragma flexible) se hace participar a otros marcos paralelos adyacentes. Por medio de un análisis de diafragma flexible (Fig. 31c) se obtiene el porcentaje de la fuerza F_h que absorbe el marco en consideración (αi). El análisis definitivo de dicho marco se hará aplicándole las cargas verticales de grúa y una fuerza horizontal de restricción en dirección contraria a la de Δg con valor $(1 - \alpha i)F_h$. O sea, si el marco en consideración absorbe αi de la fuerza horizontal equivalente F_h todos los otros marcos absorben $(1 - \alpha i)F_h$ (Ver Fig. 31d).

En realidad, debido a las dimensiones del puente de la grúa, para una cierta posición del puente, se deben cargar dos o tres marcos adyacentes (Fig. 32a). Por medio de un análisis de diafragma, considerando la posición de cada marco cargado como una condición de carga de fuerza horizontal, se puede obtener líneas de influencia de fuerzas en los resortes. El porcentaje total αi se puede obtener por superposición lineal, (Ver Fig. 32b):

$$\alpha i = \beta o \, \alpha io + \beta 1 \, \alpha i1 + \beta 2 \, \alpha i2$$

Donde: $\beta i = Pi/P \max$

$\beta o = 1$

El análisis definitivo del marco se hace como se describe en los párrafos anteriores.

6.2.2 Cargas horizontales debidas a grúa.

Las fuerzas horizontales, en la dirección transversal del edificio, que produce el cabeceo de la grúa o la fuerza sísmica de la grúa, están aplicadas a nivel de la ménsula que soporta la trabe-carril, y en general el diafragma distribuidor se encuentra a nivel de cubierta.

El diafragma, en conjunto con los marcos paralelos adyacentes, funciona como elemento restringente de los desplazamientos laterales del marco; sin embargo la flexión local en las columnas del marco producida por la carga horizontal concentrada existirá a su máxima intensidad, a menos que se proporcione una armadura horizontal a cada lado de la trabe-carril, cuyo peralte sea igual a la distancia entre dicha trabe y los paties exteriores de las columnas.

El efecto de restricción del desplazamiento lateral del marco transversal en consideración puede cuantificarse y tomarse en cuenta en el análisis de la siguiente forma:

Aplíquese la fuerza horizontal debida a grúa F_g al marco como si estuviera aislado y obténgase el desplazaminto lateral a nivel de cubierta Δ_s (Fig. 33a). Aplíaquese una carga unitaria al mismo marco a nivel de cubierta y el desplazamiento que produce en dicho nivel Δ_1 (Fig. 33b).

La fuerza horizontal F_h aplicada a nivel de cubierta que produciría el mismo desplazamiento horizontal en la cubierta que el desplazamiento de la fuerza de grúa es:

$$F_h = \frac{\Delta s}{\Delta 1} \times 1$$

Esta fuerza F_h es la que se aplica al modelo del diafragma para obtener el porcentaje de fuerza que absorbe el marco en consideración αi (Análogo a la sección 6.2.1). Para la obtención de αi se puede hacer uso de la Fig. 26, (Fig. 33c).

El análisis definitivo del marco se hace aplicando las fuerzas actuantes F_g, y una fuerza horizontal de restricción en dirección contraria a la del desplazamiento Δs, con valor $(1 - \alpha i) F_h$. Esta última fuerza es la que todos los otros marcos paralelos al marco en consideración absorben.

6.3 Procedimiento para el análisis de naves industriales.

A continuación se describe un procedimiento racional y pseudooptimizado de naves industriales. La filosofía básica consiste en considerar y hacer uso del comportamiento tridimensional de la estructura por medio de artificios que permiten reducir el análisis tridimensional a una serie de análisis de estructuras planas. El análisis de la cubierta considerada como diafragma flexible, permite que se efectúe el análisis de los marcos transversales como parte integral de la estructura y no en forma independiente como se ha hecho tradicionalmente.

El objetivo básico es el diseñar todos los marcos transversales para los elementos mecánicos producidos por las cargas gravitacionales únicamente, y permitir que absorban una carga lateral tal que únicamente produzca un sobre-esfuerzo del 33%. La diferencia entre la carga lateral total y la que absorben todos los marcos intermedios con el criterio descrito, se considera que es la carga que deben absorber los dos marcos exteriores o cabecera.

La transmisión del exceso de carga lateral de los marcos transversales intermedios a los dos cabecera se hace por medio del diafragma en la cubierta. Los marcos cabecera deberán estar contraventeados para forzar su diferencia de rigidez lateral con respecto a los marcos intermedios, y al mismo tiempo, para que puedan absorber dicha carga lateral mayor.

A continuación se describe la secuela de análisis y diseño del procedimiento propuesto:

a) Proponer un contraventeo preliminar en la cubierta, determinar la rigidez de la cubierta como diafragma flexible (Kd).

b) Efectuar el estudio de compatibilidad de desplazamientos laterales producidos por grúa como se describe en las secciones 6.2.1 y 6.2.2 para definir las fuerzas de restricción correctivas.

c) Analizar y diseñar los marcos intermedios transversales para cargas gravitacionales y de grúa, así como determinar su rigidez lateral (Kmi). Las cargas de grúa que se deben considear son las propias más las correctivas de restricción descritas en el paso b.

d) Evaluar la carga horizontal total debida a sismo y la debida a viento (P_t).

e) Calcular la carga horizontal en los marcos transversales intermedios que produciría un sobre-esfuerzo del 33% en sus elementos (P_{33}).

f) Evaluar la carga horizontal que deben resistir los marcos exteriores o cabecera (Pe) se tendrá:

$$M_i \; P_{33} + 2Pe = P_t$$

despejando Pe:

$$Pe = \frac{P_t - M_i \; P_{33}}{2}$$

donde:

M_i = Número de marcos intermedios.

Entonces, cada marco exterior deberá absorber:

$$\%CA = \frac{Pe}{P_t}$$

g) Hacer el análisis y diseño preliminar de los marcos cabecera, incluyendo su contraventeo, necesarios para absorber Pe, así como su rigidez lateral Kme.

h) Revisar el contraventeo de la cubierta de manera que se garantice la transmisión de %CA a los marcos cabecera.

Con relación Kme/Kmi conocida, y para el %CA necesario, determinar Kd/Kmi de las Figuras 19 ó 20, o su interpolación. Conocido Kmi se puede determinar la rigidez del diafragma a partir de la relación Kd/Kmi obtenida.

Si dicha rigidez Kd difiere de la supuesta inicialmente repetir el ciclo una vez.

7. CONCLUSIONES.

El considerar los sistemas de piso de estructuras tipo industrial como diafragmas flexibles conduce a una utilización más racional de los elementos estructurales, y consecuentemente a diseños más económicos.

El proceso de análisis requiere de ciertos pasos adicionales, que incluye el análisis de los sistemas de piso como estructuras planas, y el forzar la compatibilidad de desplazamientos laterales de los marcos al analizarlos también como estructuras planas. Este trabajo adicional resulta rutinario con el uso de las computadoras, y a pesar del ligero incremento del costo de análisis, se logran ahorros importantes en el costo de la estructura.

AGRADECIMIENTO

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3. A. Villicaña, "Análisis Estructural de Naves Industriales Considerando el Efecto de Diafragma Flexible", Tesis Profesional, Facultad de Ingeniería, UNAM, 1977.

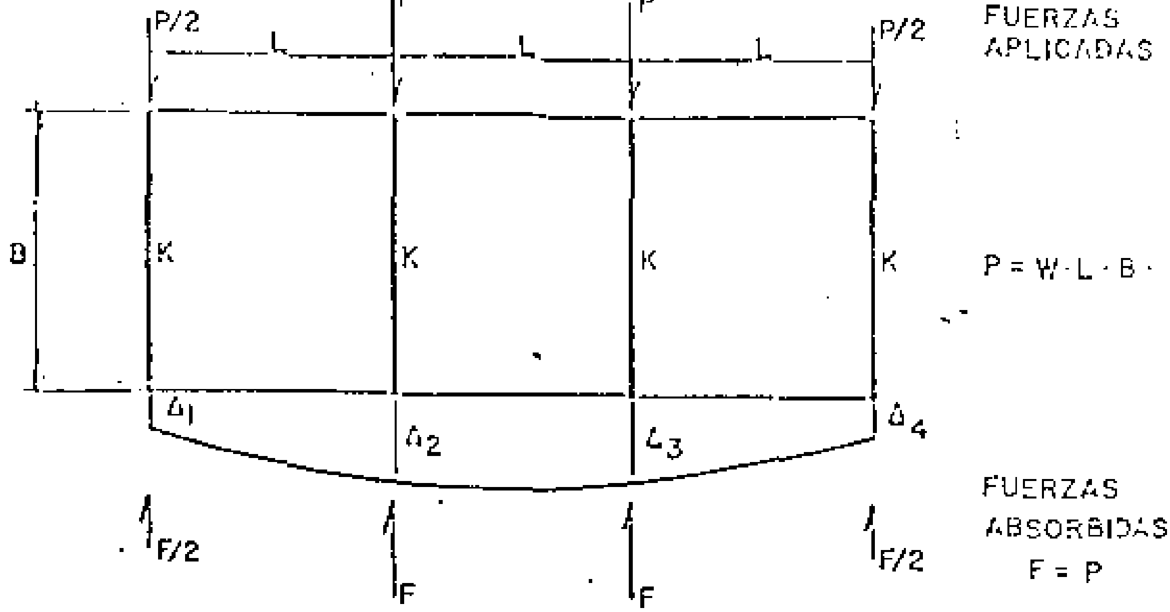


FIG. 1a — NIVEL SIN LOSA DE PISO SIN DIAFRAGMA.

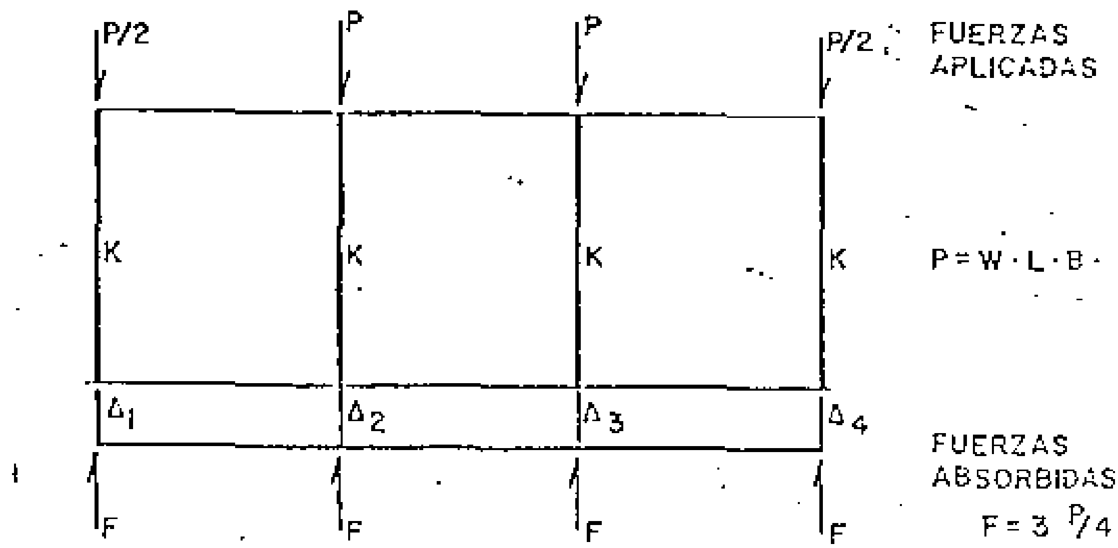


FIG. 1b — NIVEL CON DIAFRAGMA INFINITAMENTE RIGIDO SIN TORSION.

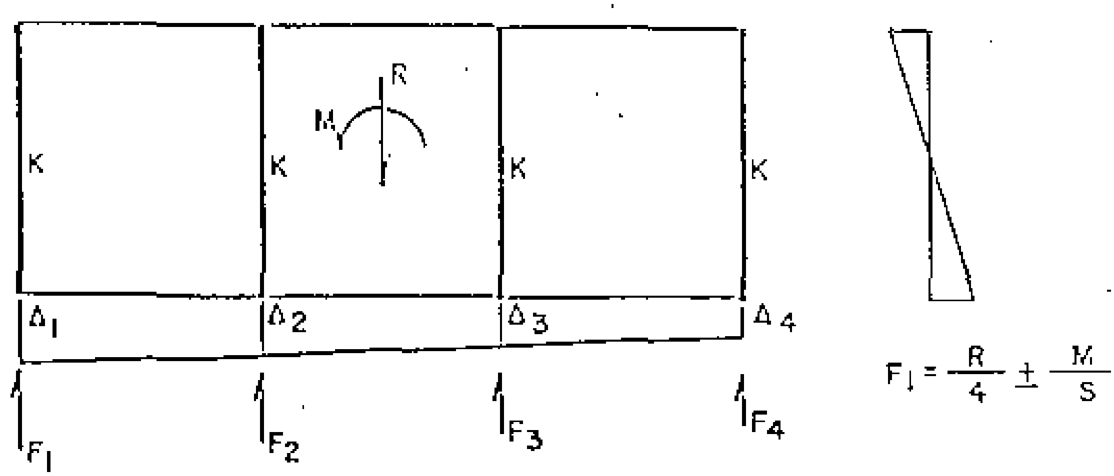
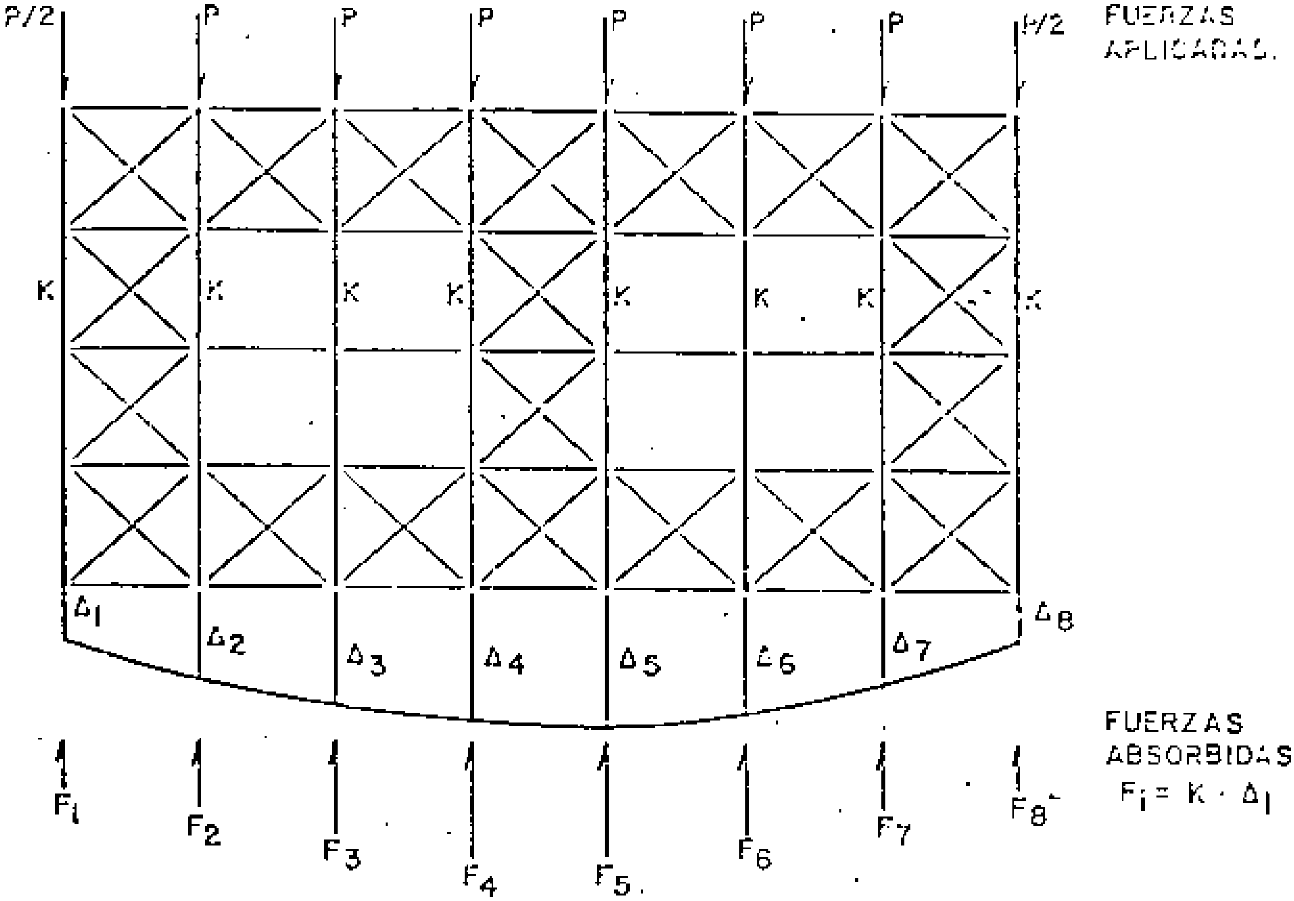
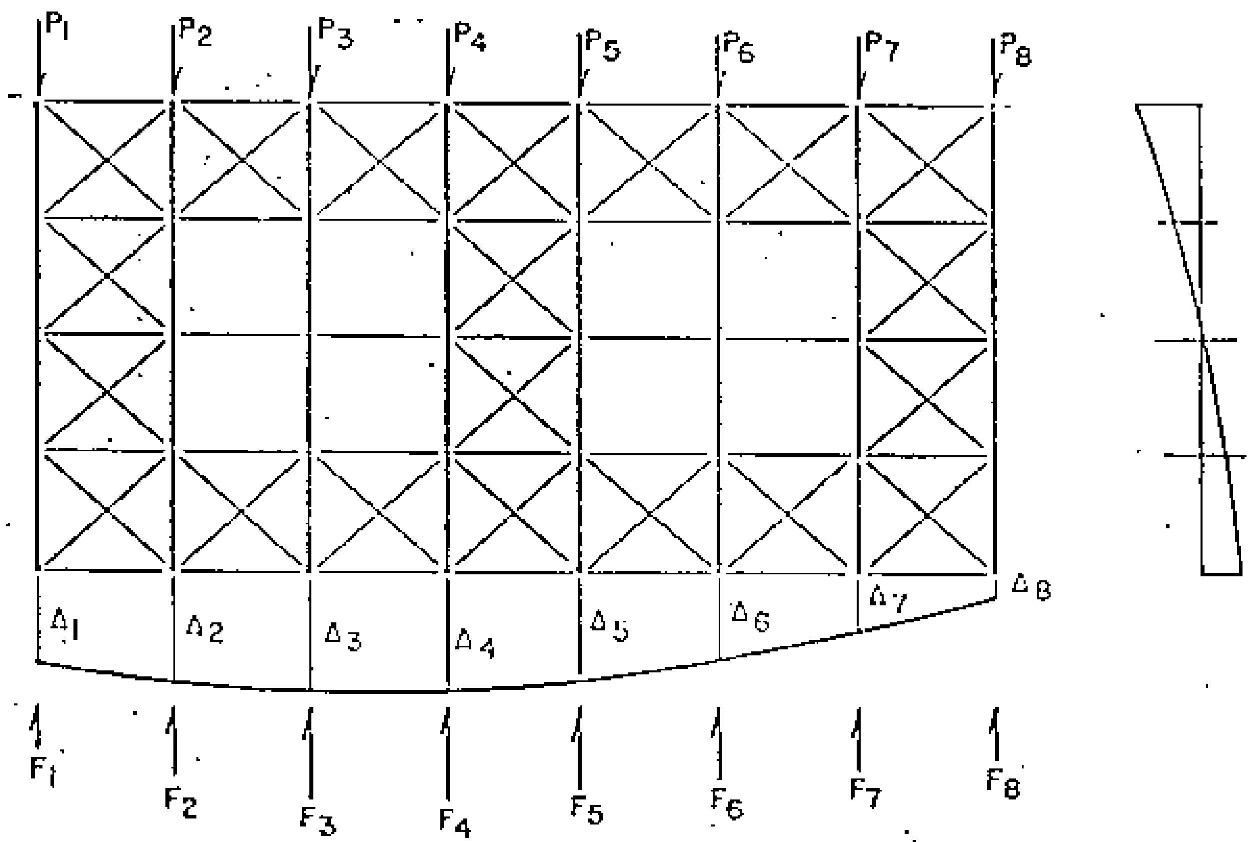


FIG. 1c — NIVEL CON DIAFRAGMA INFINITAMENTE RIGIDO CON TORSION.

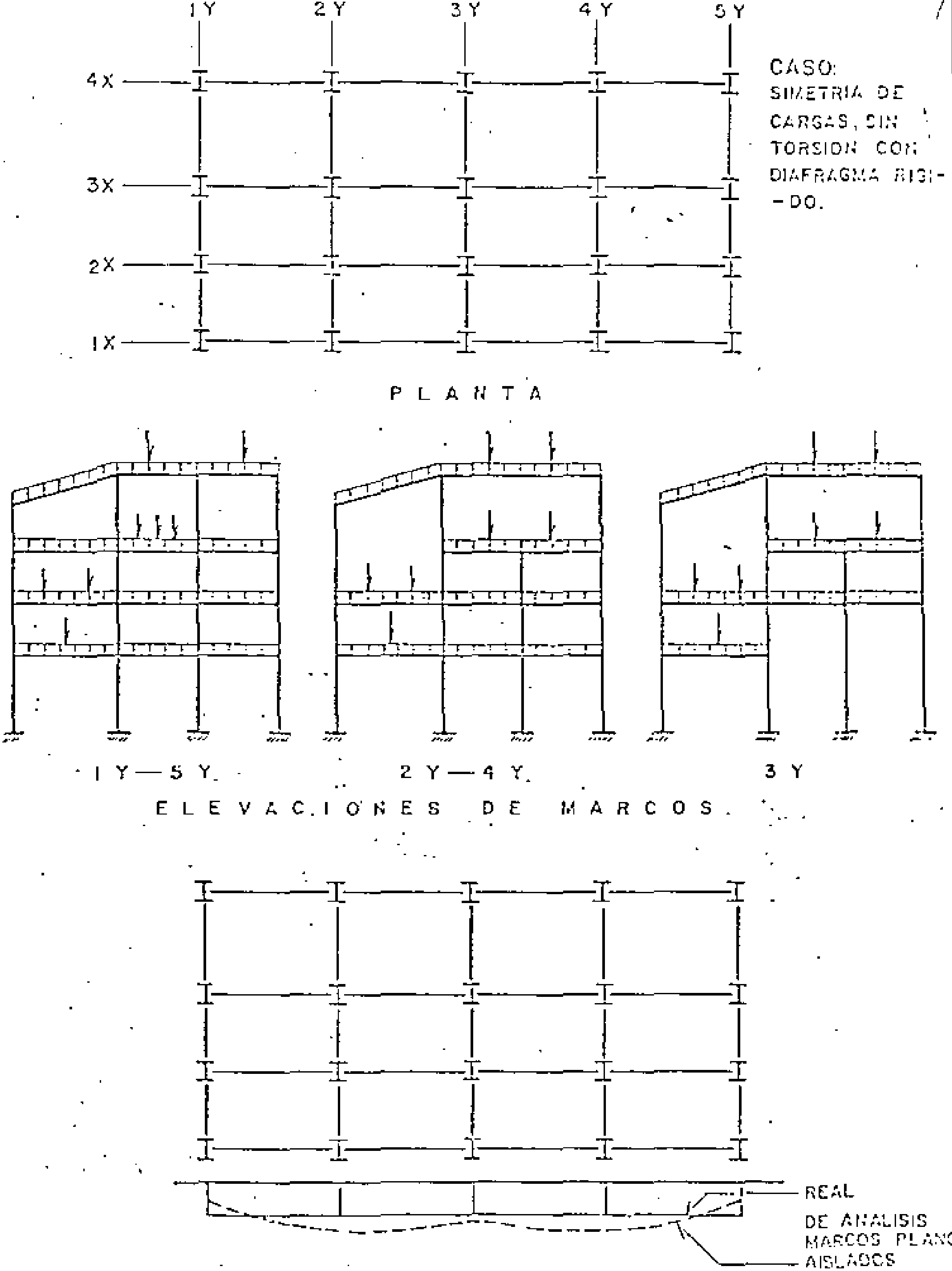


DIAFRAGMA FLEXIBLE CARGAS SIMETRICAS.



DIAFRAGMA FLEXIBLE CARGAS ASIMETRICAS

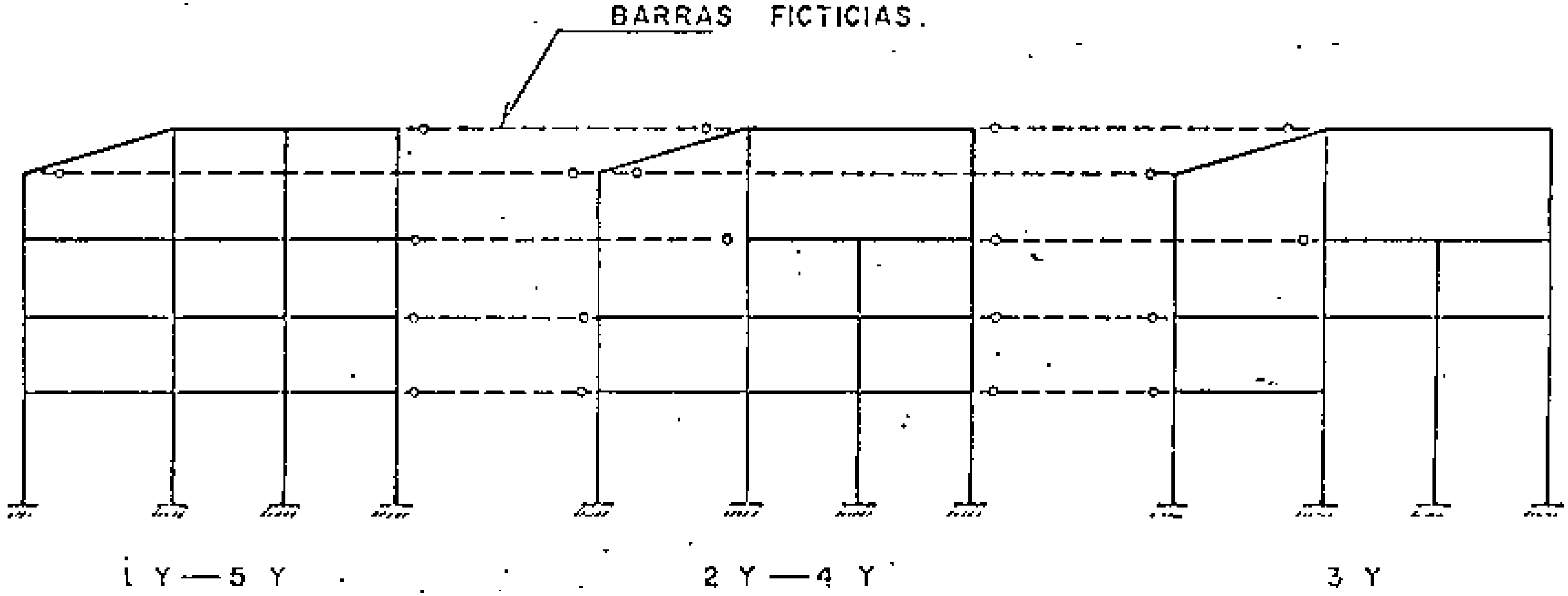
FIG. 2



DESPLAZAMIENTOS LATERALES POR CARGAS VERTICALES.
FIG. 3

CASO:

DIAFRAGMA RIGIDO EN CADA NIVEL Y SIN TORSION.
SIMETRIA DE CARGAS Y GEOMETRICA.



TIPOS DE MARCOS.

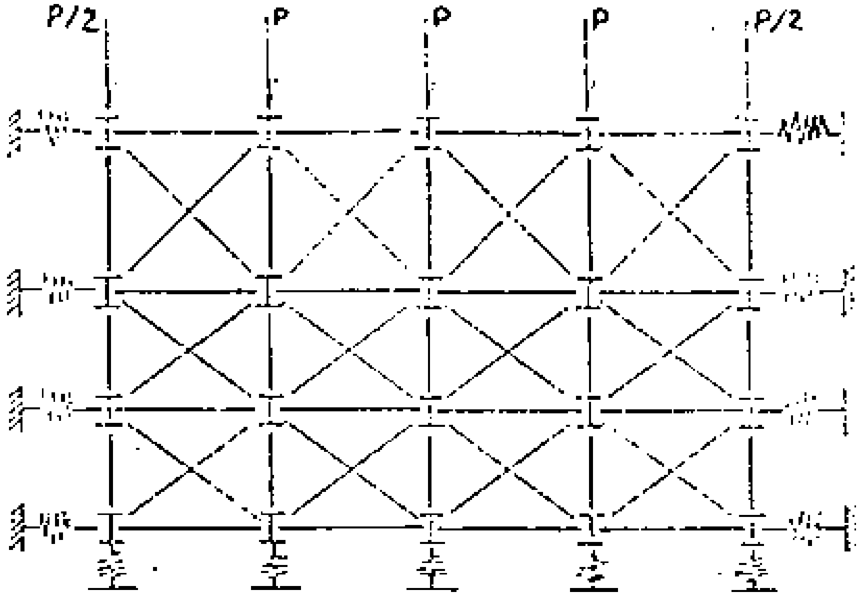
BARRAS FICTICIAS:

SIMULAN EL DIAFRAGMA RIGIDO

$$\left[EA/L\right]_{b.f.} = \pm\ 100 \text{ VECES EL } EA/L \text{ DE LAS VIGAS DEL NIVEL EN CONSIDERACION}$$

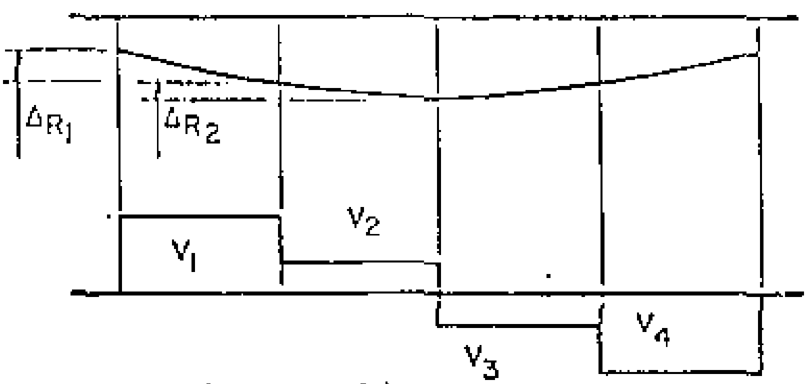
MODELO ESTRUCTURAL
PARA LOGRAR COMPATIBILIDAD DE DESPLAZAMIENTO HORIZONTALES.
DEBIDOS A CARGAS VERTICALES Y CARGAS HORIZONTALES

FIG. 4



CASO:

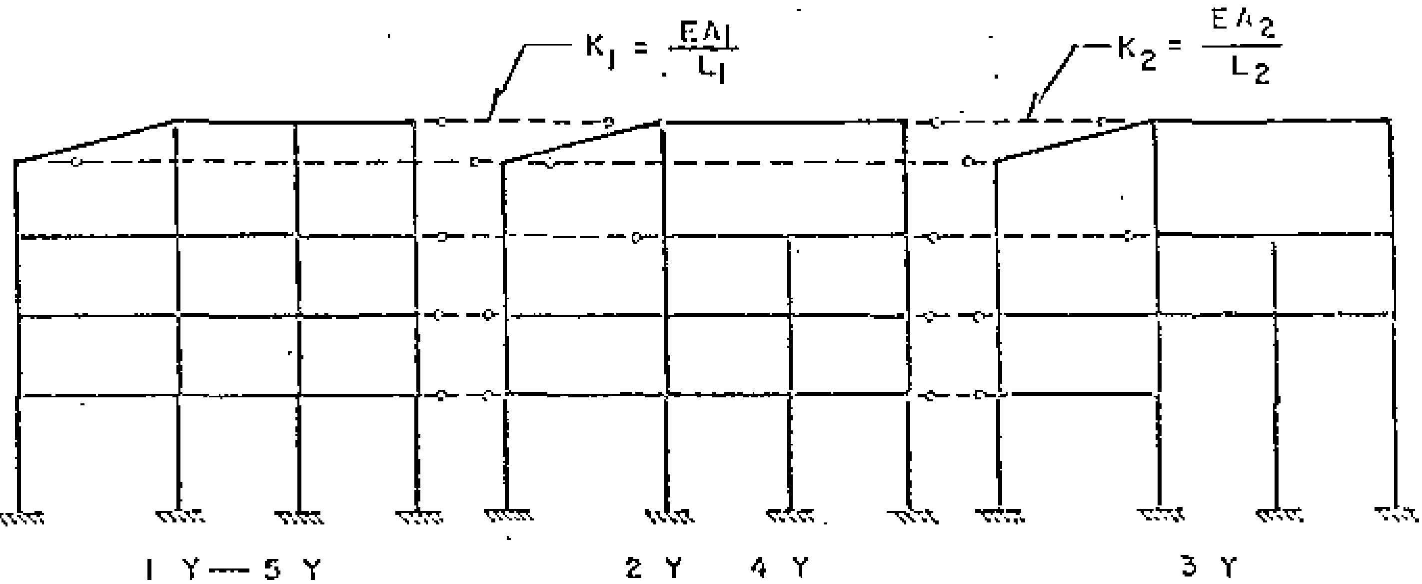
DIAPRAGMA FLEXIBLE EN CUBIERTA. SIN TORSION SIMETRIA DE CARGAS Y GEOMETRIA.



DESPLAZAMIENTOS. CUBIERTA.

CORTANTES. CUBIERTA.

$$K_1 = \frac{V_1}{\Delta_{R_1}} \quad K_2 = \frac{V_2}{\Delta_{R_2}}$$

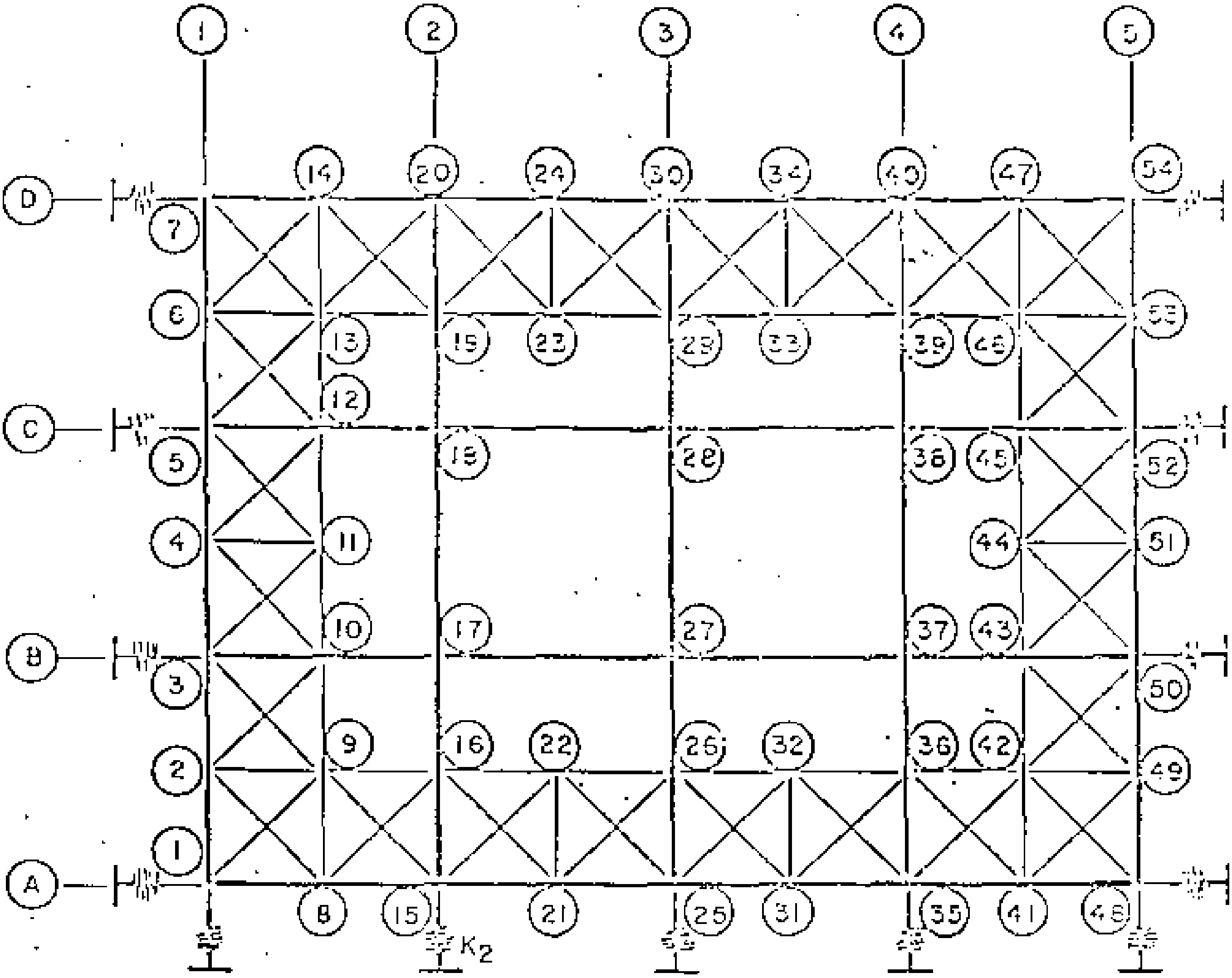


TIPOS DE MARCOS.

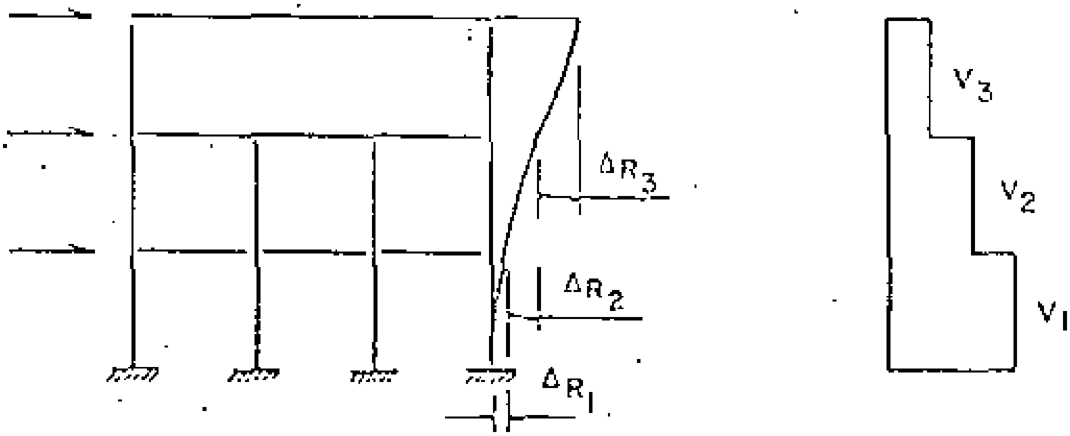
MODELO ESTRUCTURAL.

PARA LOGRAR COMPATIBILIDAD DE DESPLAZAMIENTO HORIZONTALES DEBIDOS A CARGAS VERTICALES Y CARGAS HORIZONTALES.

FIG. 5



2º NIVEL



$K_3 = V_3 / \Delta_{R_3}$

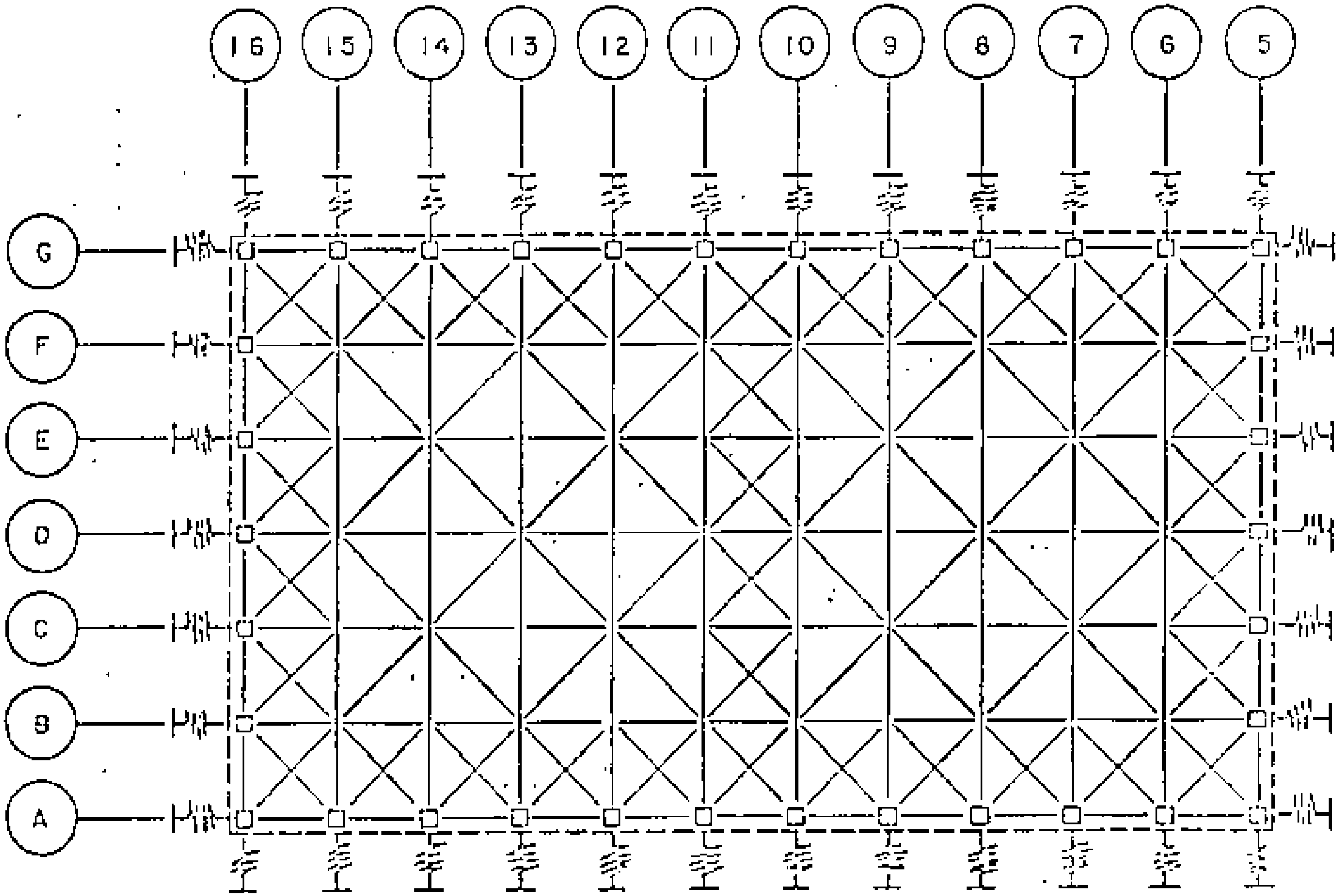
$K_2 = V_2 / \Delta_{R_2}$

$K_1 = V_1 / \Delta_{R_1}$

MARCO 2-2

MODELO ESTRUCTURAL PARA ANALISIS DE DIAFRAGMA

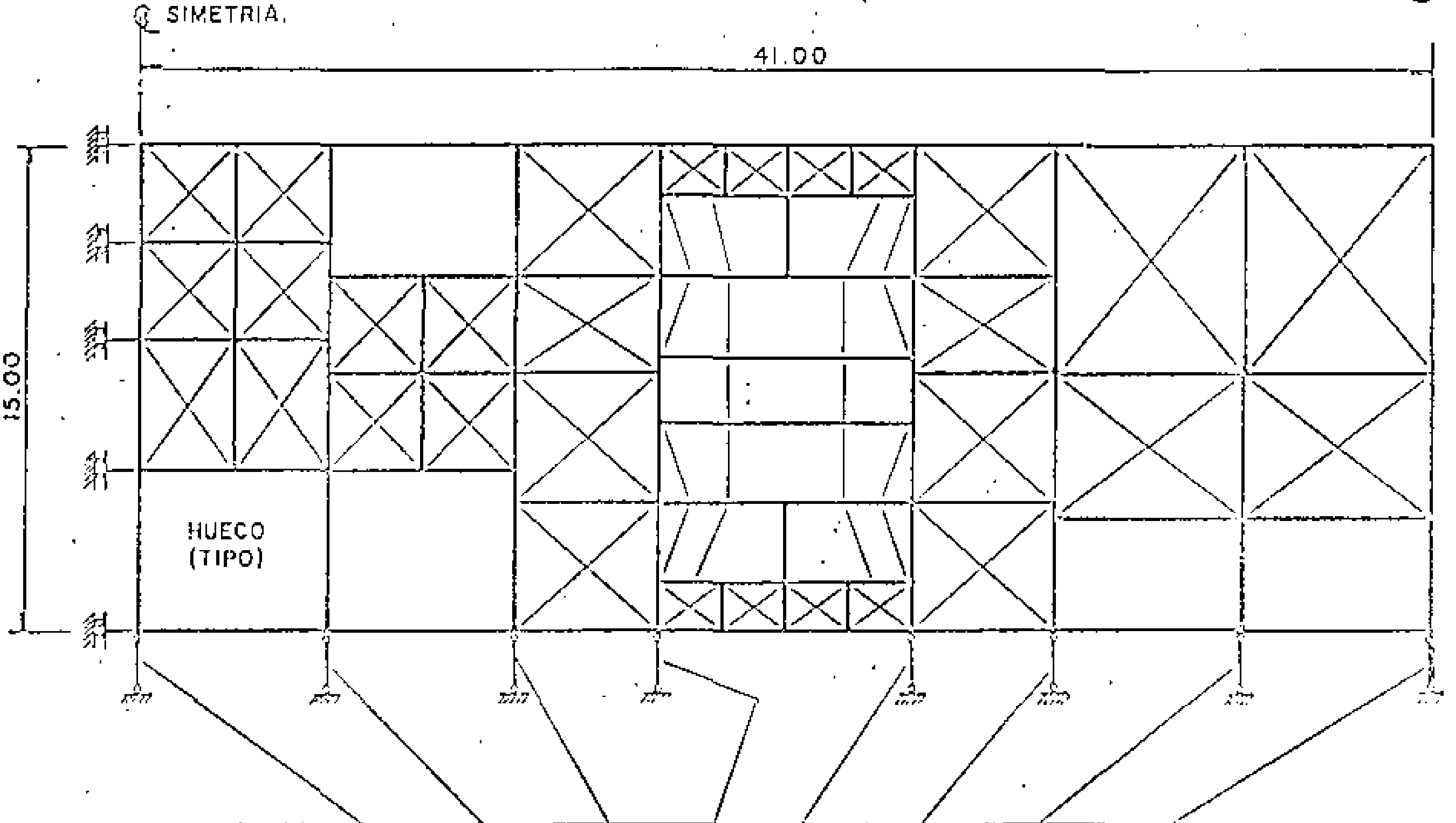
FIG. 6



% F DEL MODELO	38	2	2	2	3	3	3	3	2	2	2	38
% F DIAFRAGMA RIGIDO.	40	2	2	2	2	2	2	2	2	2	2	40

ANALISIS DE DIAFRAGMA NIVEL 31.00 (CUBIERTA)

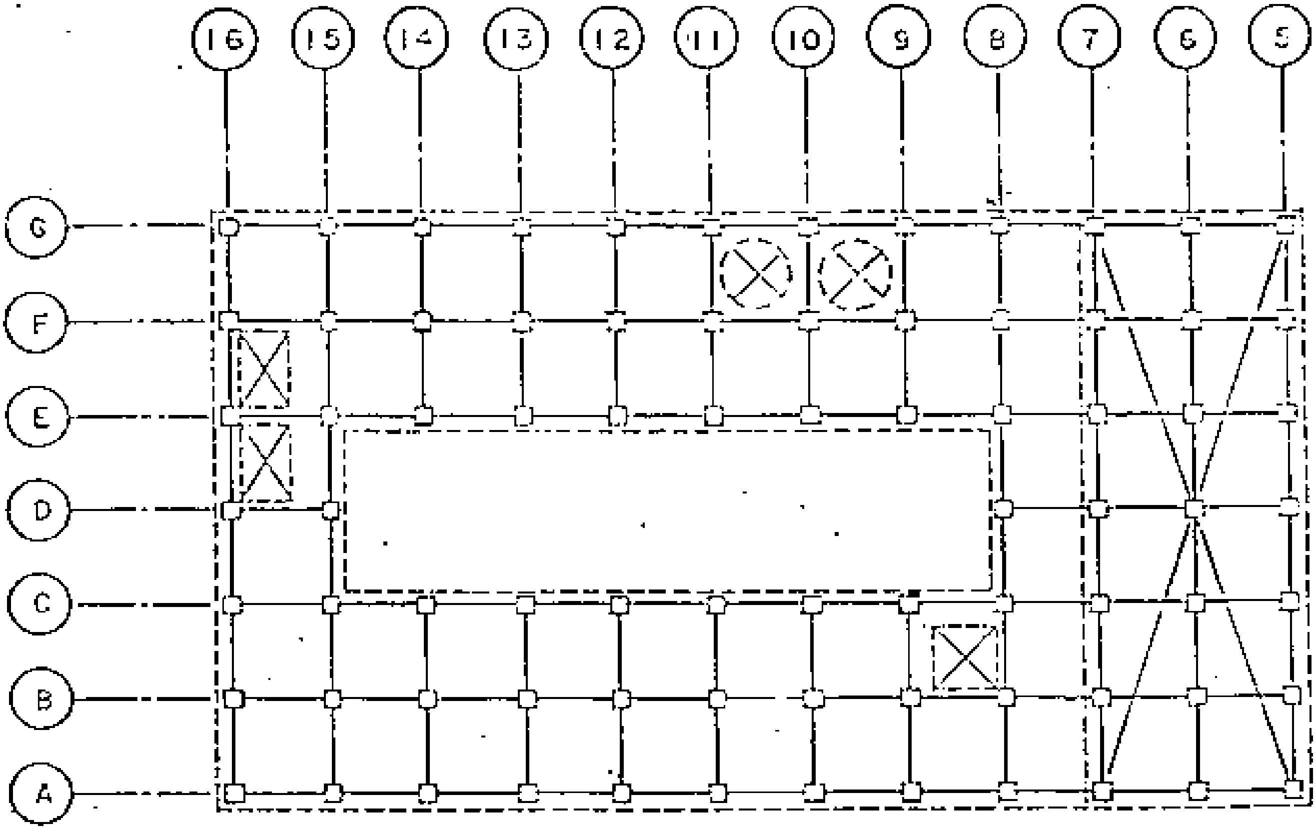
FIG. 7



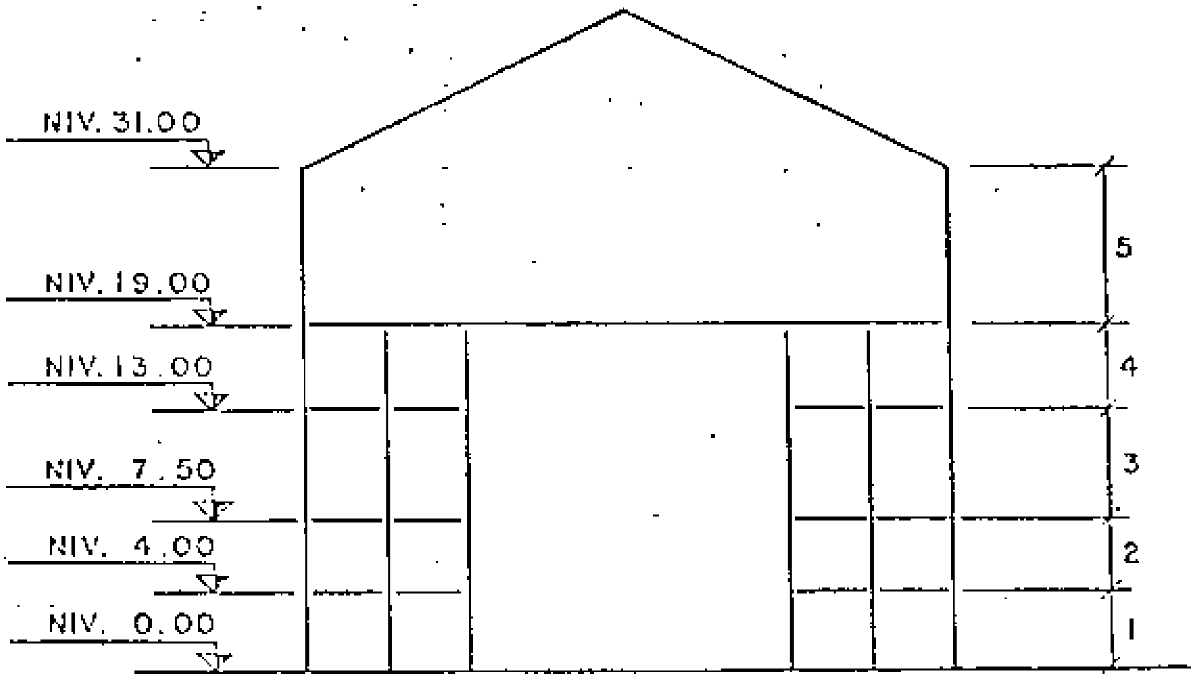
%F EN EL MODELO.	16.2	2.4	4.4	4.9	7.2	7.2	5.1	5.2
%F DIAFRAGMA RIGIDO.	24.1	3.6	3.4	5.1	5.1	3.5	3.7	3.2

ANALISIS DE DIAFRAGMA NIV. 5

FIG. 8

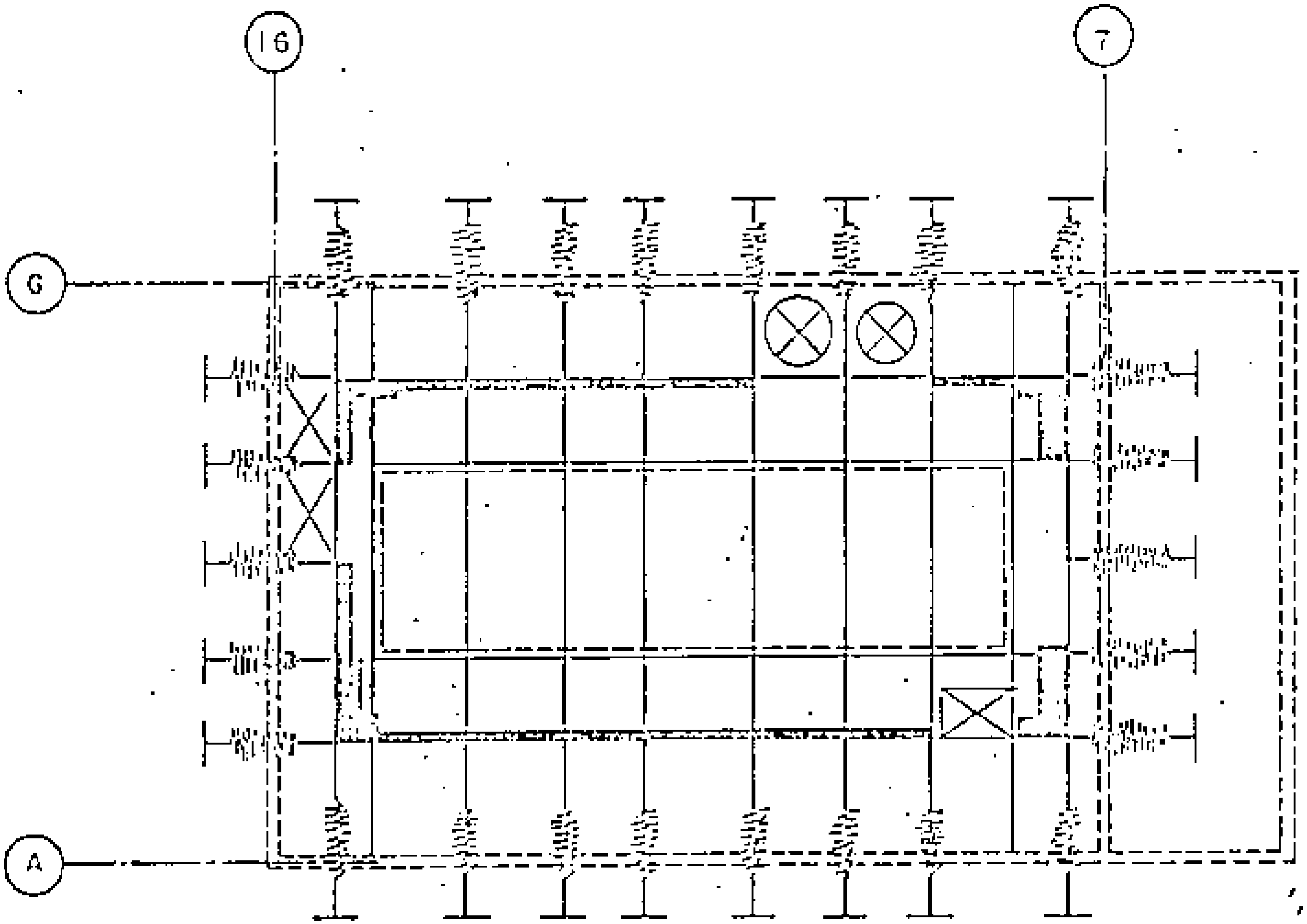


PLANTA



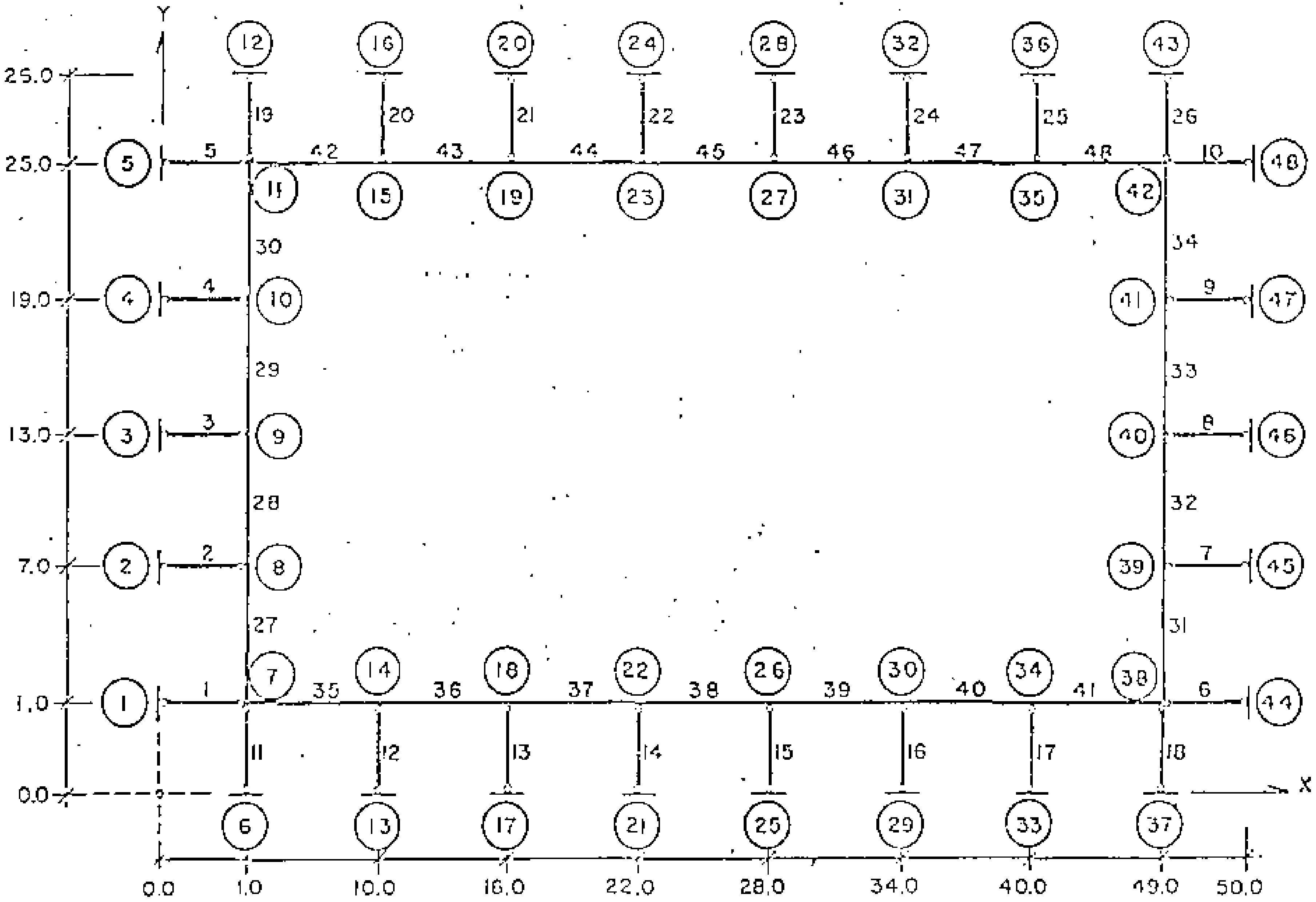
ELEVACION TRANSVERSAL.

FIG. 9

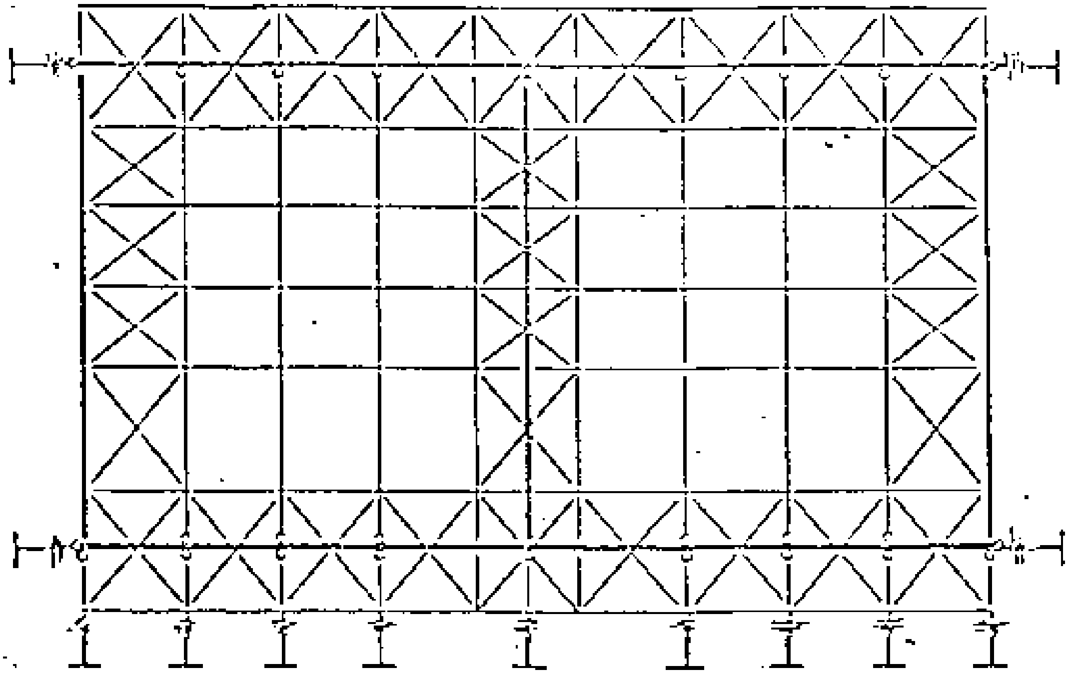


MODELO ESQUEMATICO.

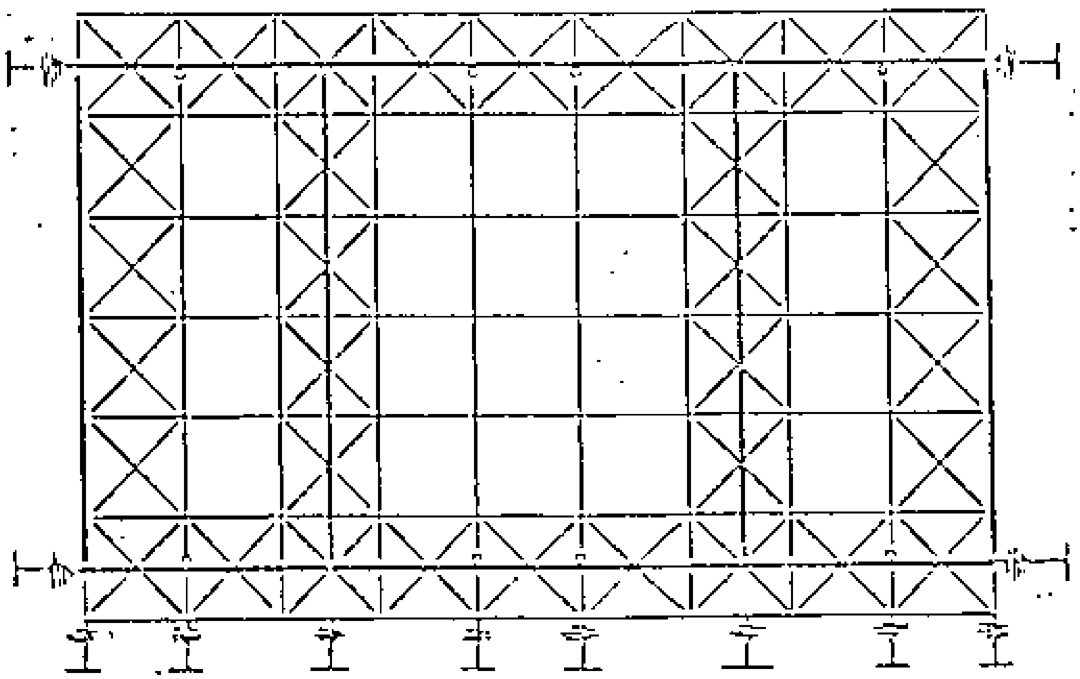
FIG. 10



MODELO DE ANALISIS.
FIG. II



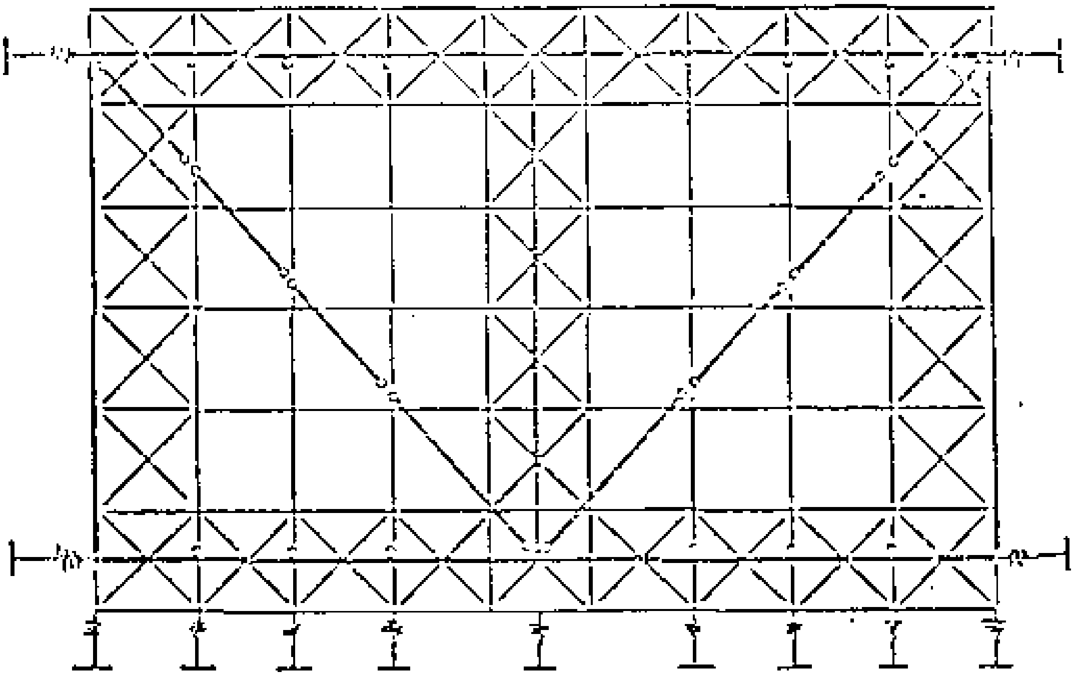
CONTRAVENTEO TIPO 1



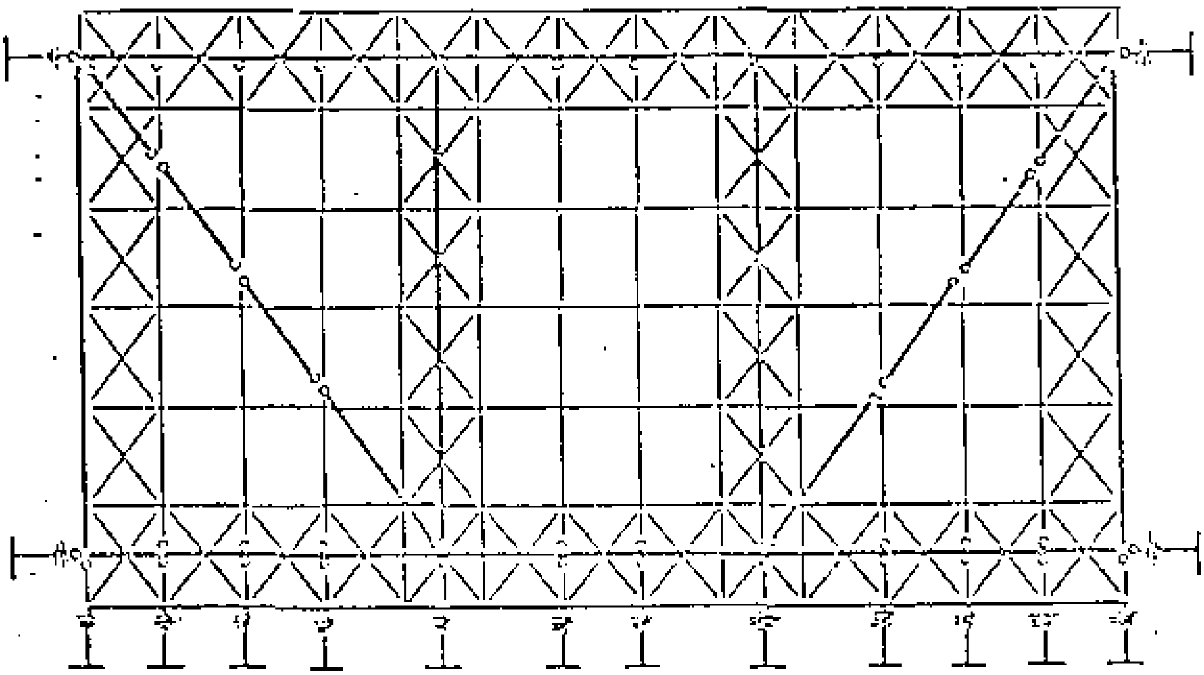
CONTRAVENTEO TIPO 2

TIPOS DE CONTRAVENTEOS

FIG. 12



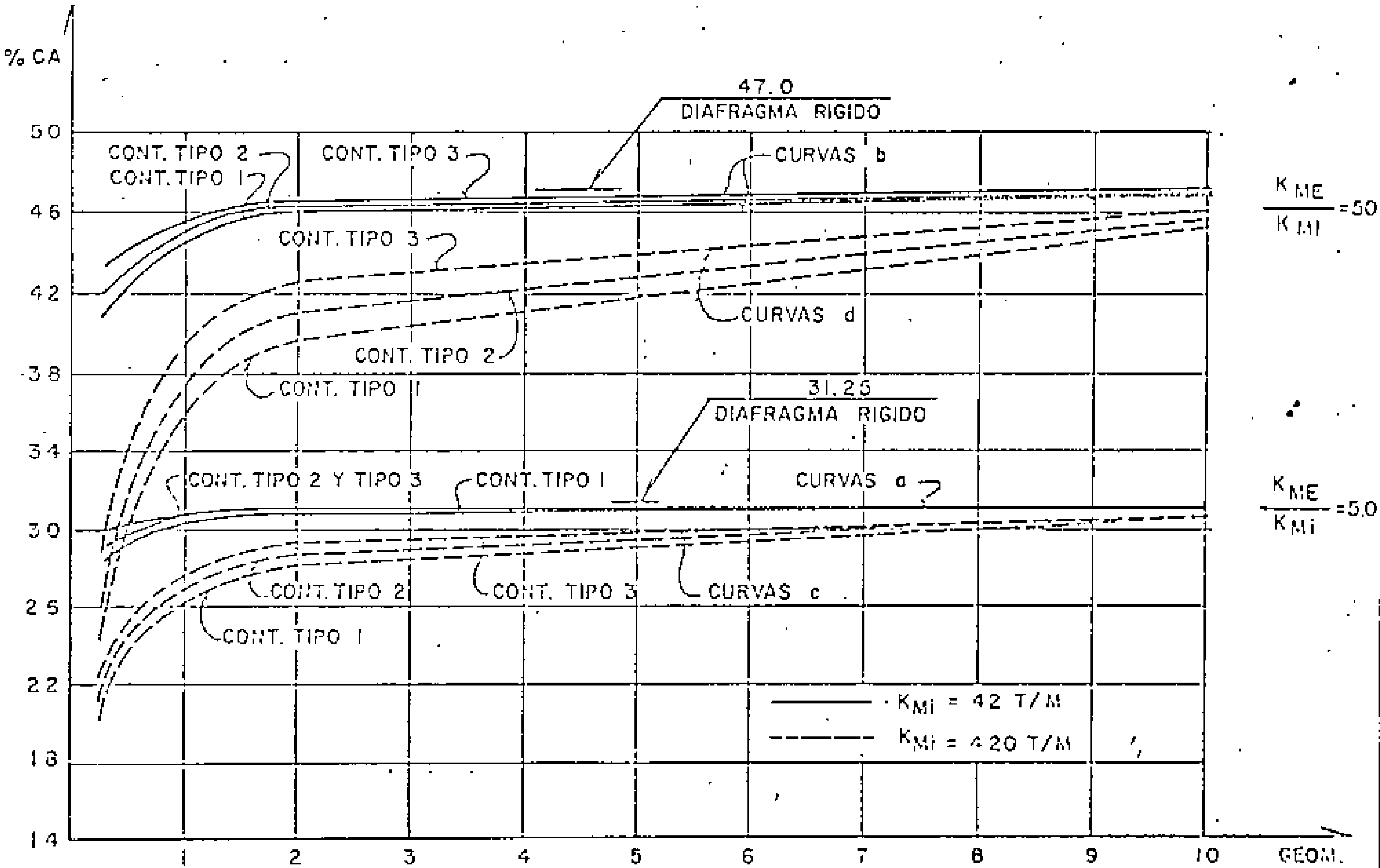
CONTRAVENTEO TIPO 3



CONTRAVENTEO TIPO 4

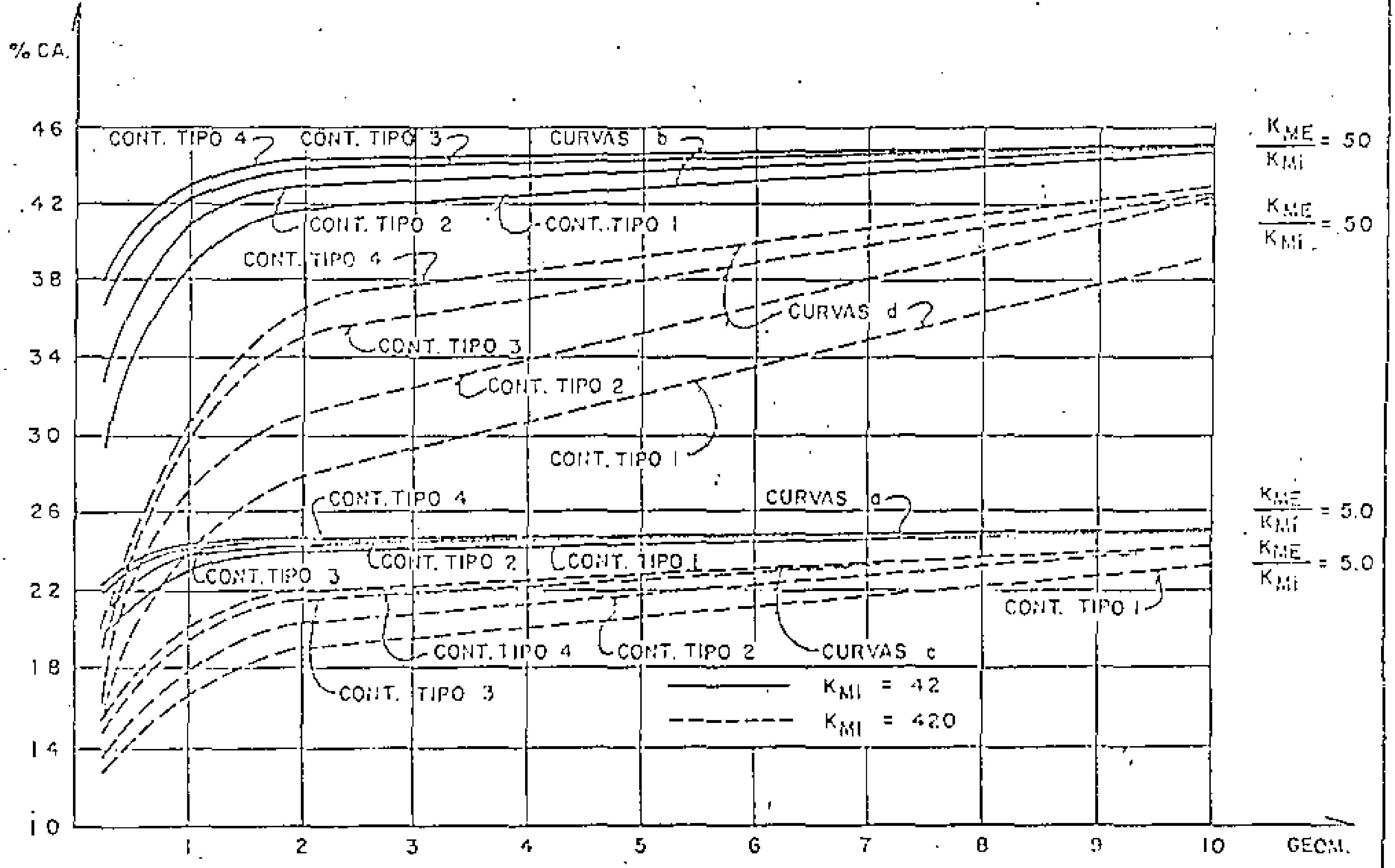
TIPOS DE CONTRAVENTEOS.

FIG. 13



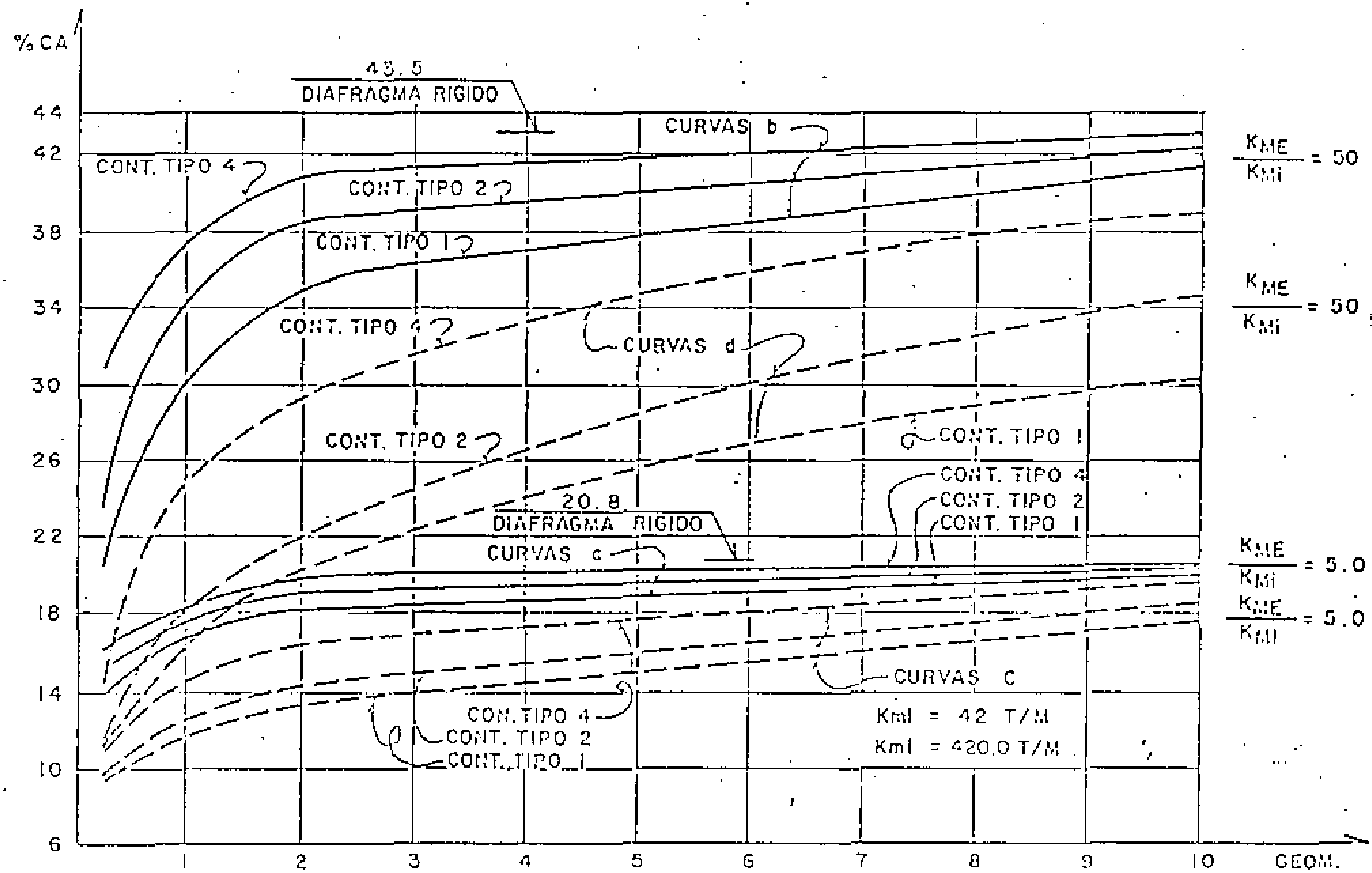
PORCENTAJE DE CARGA EN MARCO CABECERA DEBIDO A FUERZAS HORIZONTALES UNIFORMES PARA UNA RELACION A/B = 1.5

FIG. 14



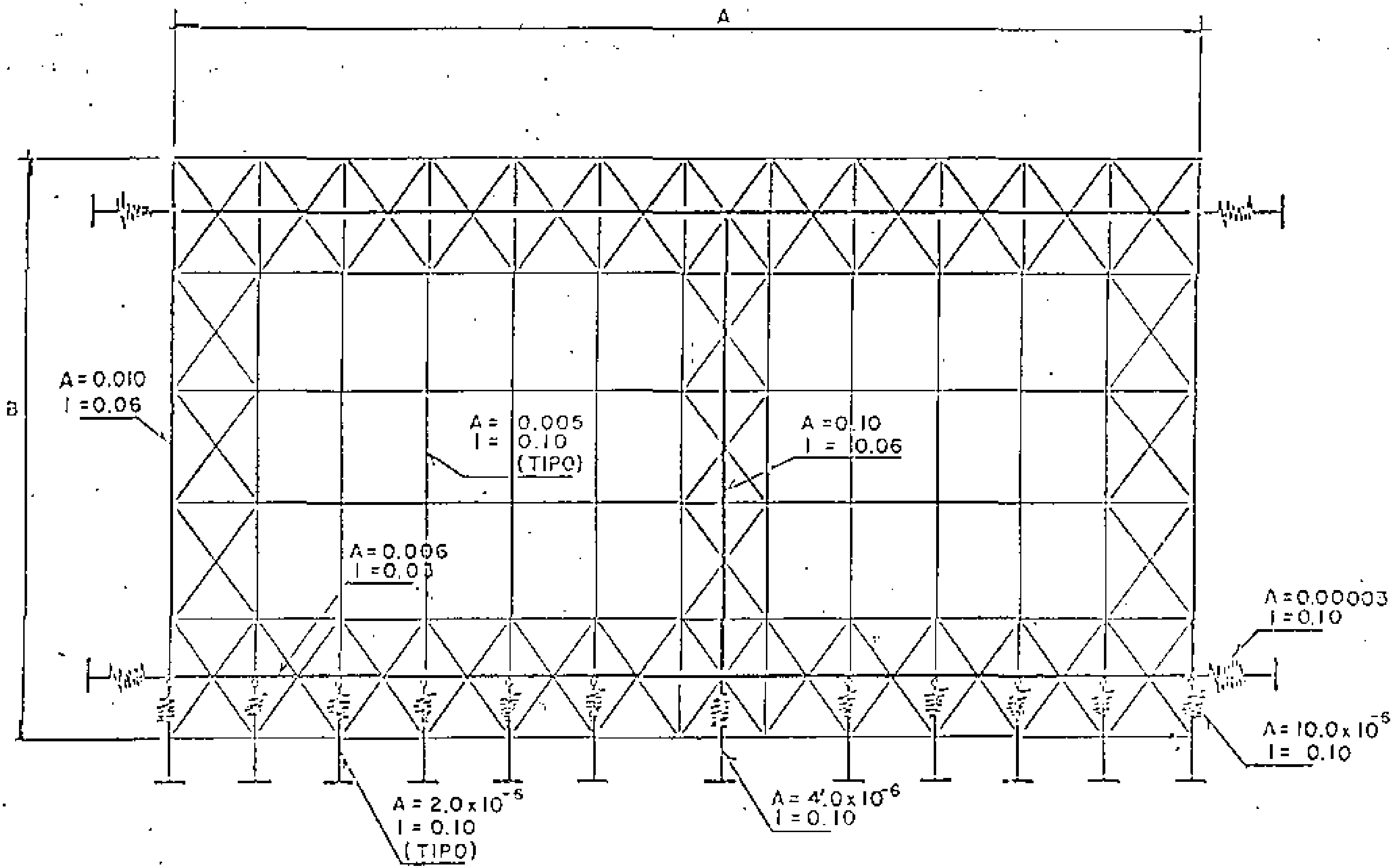
PORCENTAJE DE CARGA QUE ABSORBE EL MARCO CABECERA DEBIDO A FUERZAS HORIZONTALES UNIFORMES PARA UNA RELACION A/B=2.2

FIG. 15



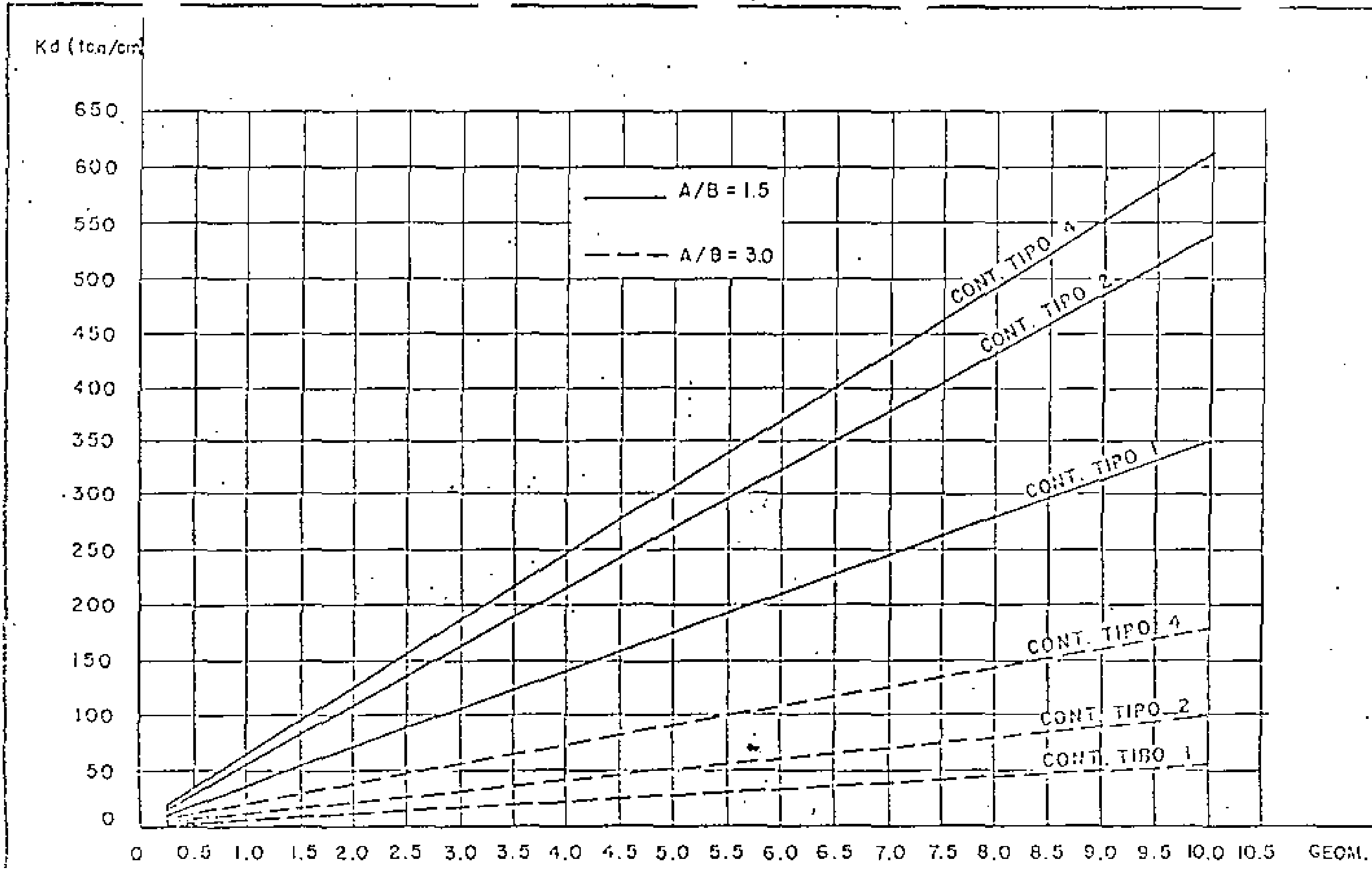
PORCENTAJE DE CARGA EN MARCO CABECERA DEBIDO A FUERZAS HORIZONTALES UNIFORMES PARA UNA RELACION A/B = 3.0

FIG 16

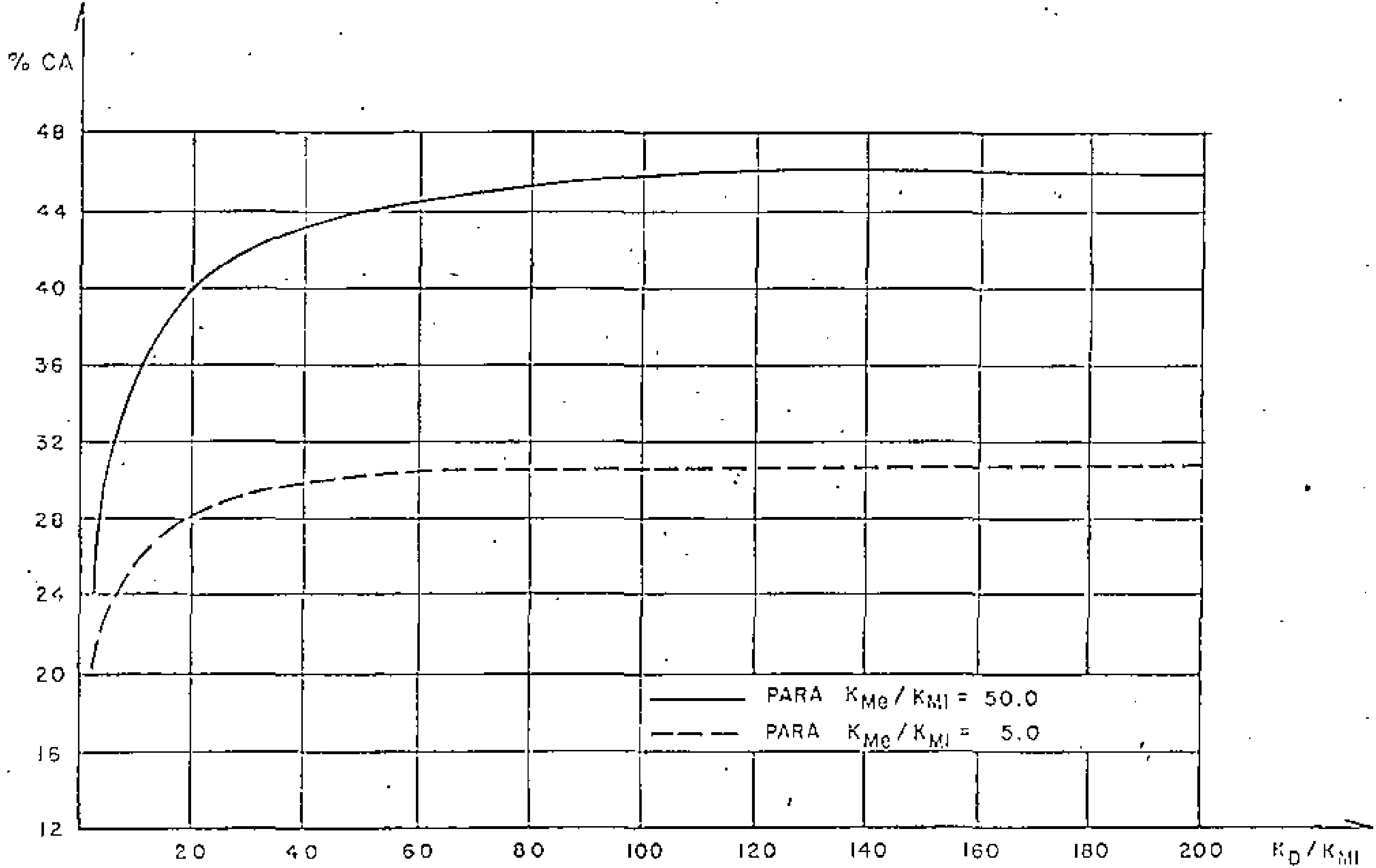


PROPIEDADES GEOMETRICAS DE PARTIDA DEL ESTUDIO PARAMETRICO.

FIG. 17

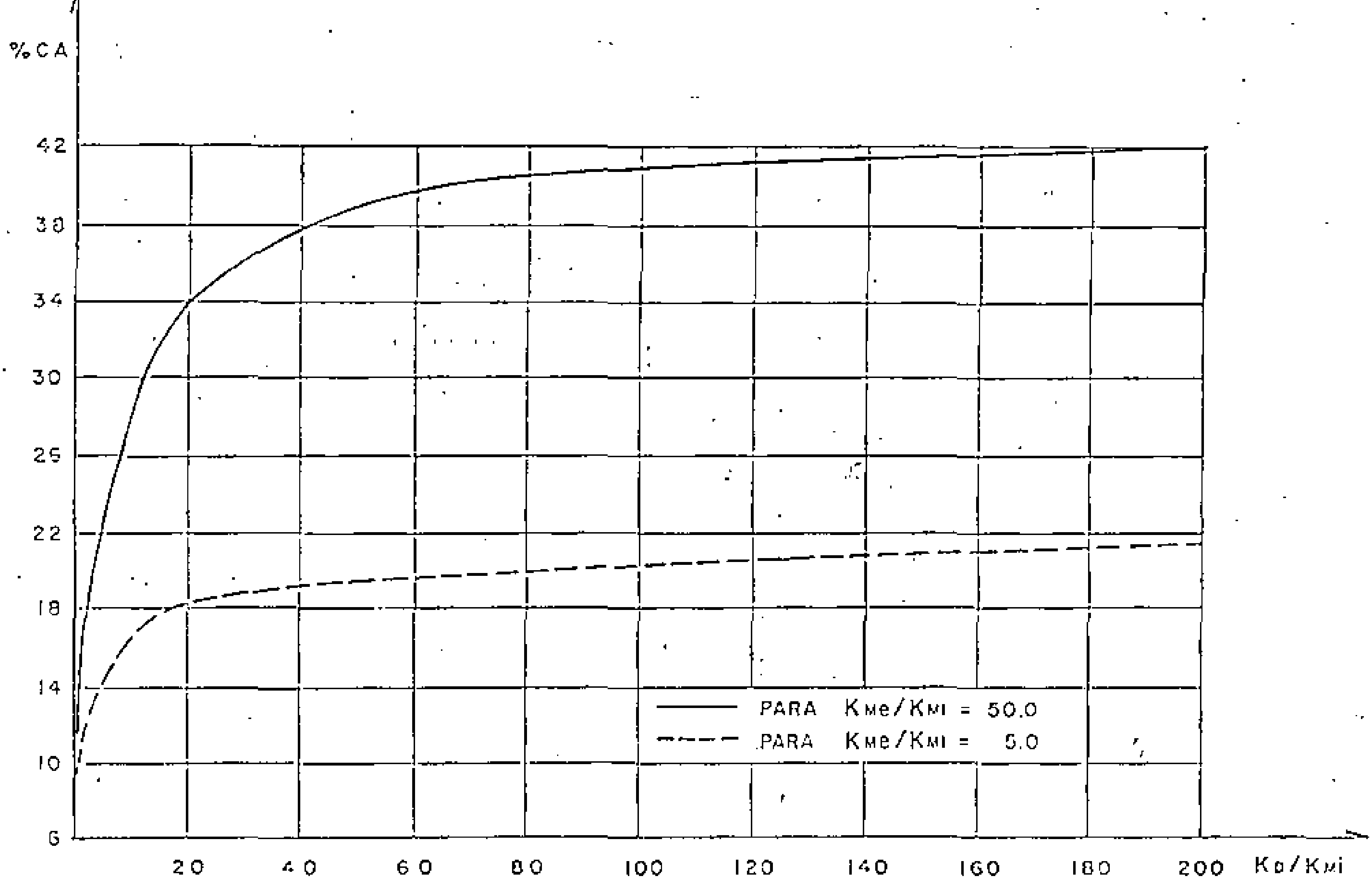


VARIACION DE LA RIGIDEZ DE DIAFRAGMA CON RESPECTO
A LA GEOMETRIA Y TIPO DE CONTRAVENTEO.
FIG. 18



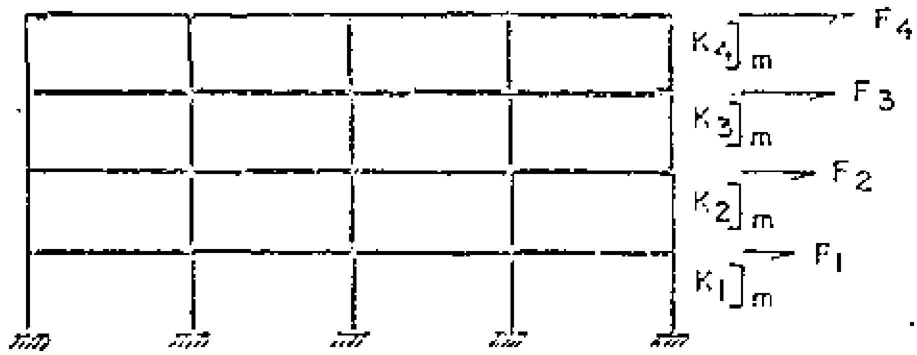
PORCENTAJE DE CARGA EN MARCO CABECERA PARA UNA RELACION
A/B = 1.5 SEGUN RELACION DE RIGIDECES.
FIG. 19

110

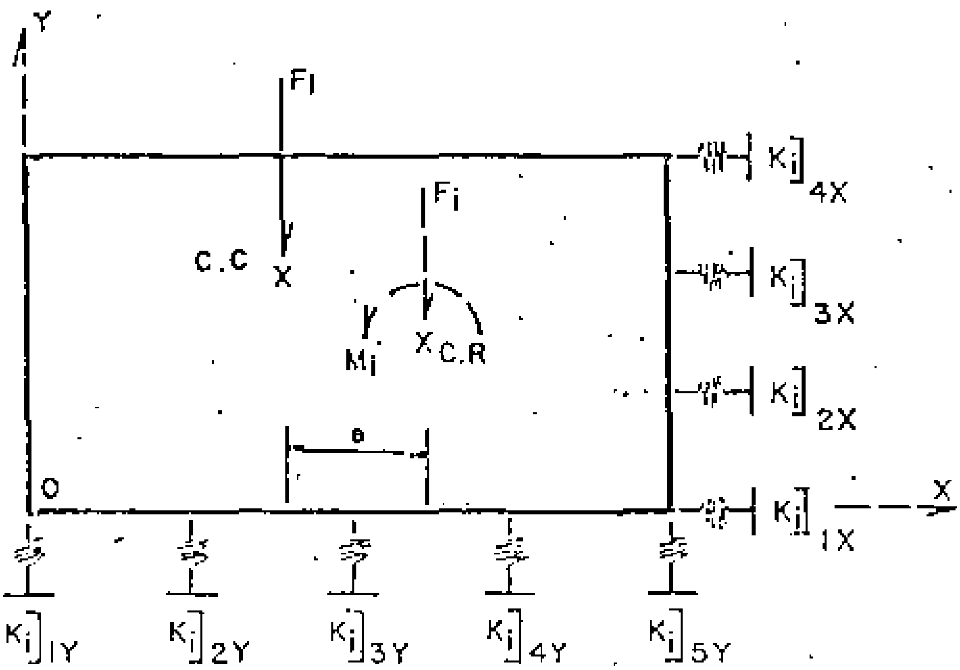


PORCENTAJE DE CARGA EN MARCOS CABECERA PARA UNA RELACION A/B = 3.0 SEGUN RELACION DE RIGIDECES.

FIG. 20



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

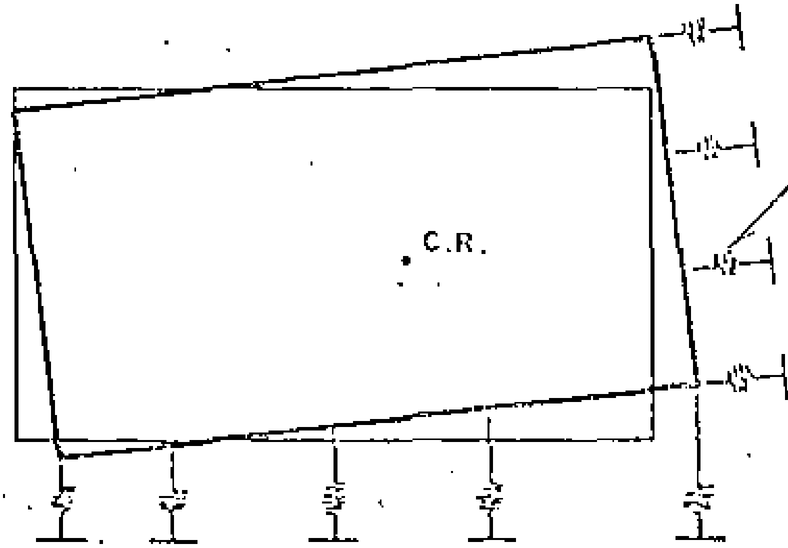


$$X_{CR} = \frac{\sum K_i]_y \cdot X_m}{\sum K_i]_y}$$

$$Y_{CR} = \frac{\sum K_i]_x \cdot Y_m}{\sum K_i]_x}$$

$$X_{CC} = \frac{\sum F_i]_y \cdot X}{V_x}$$

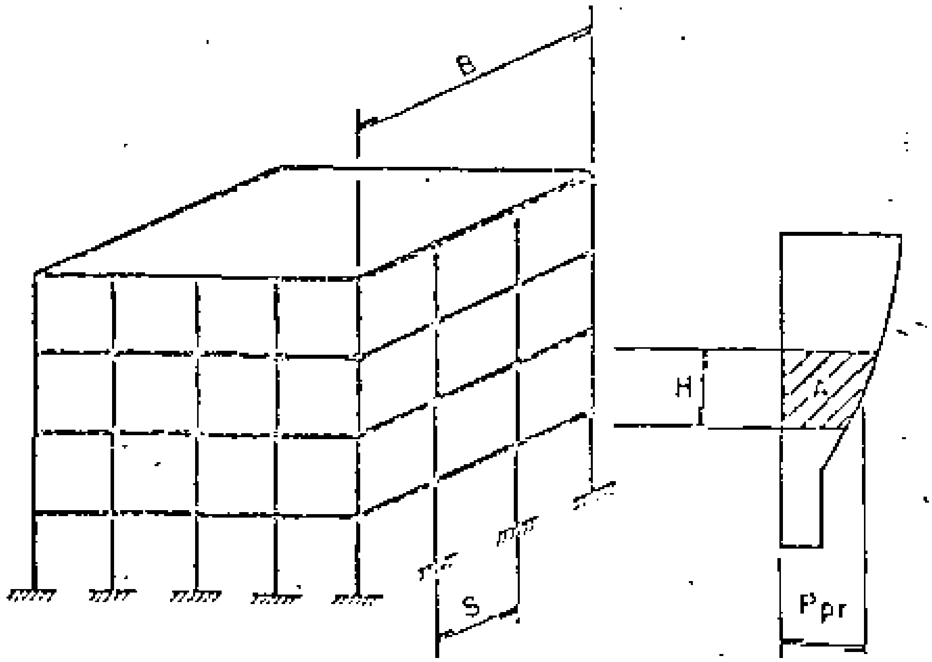
$$Y_{CC} = \frac{\sum F_i]_x \cdot Y}{V_y}$$



$$F_{M_x} = \frac{K_{i_x}}{\sum K_{i_x}} \cdot V_x \pm \frac{K_{i_x} \cdot Y_{i_1}}{\sum K_{i_x} Y_{i_1}^2 + K_{i_y} \cdot X_{i_1}^2} \cdot M_i$$

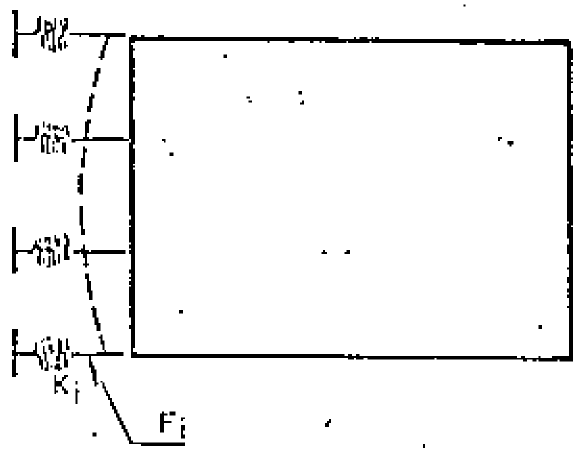
DISTRIBUCION DE FUERZAS SISMICAS
CON DIAFRAGMA RIGIDO.

FIG. 21

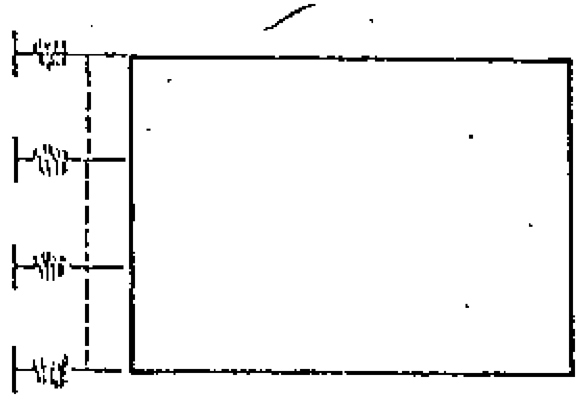


$$P = P_{pr} \cdot H \cdot B$$

$$P = A \cdot B$$



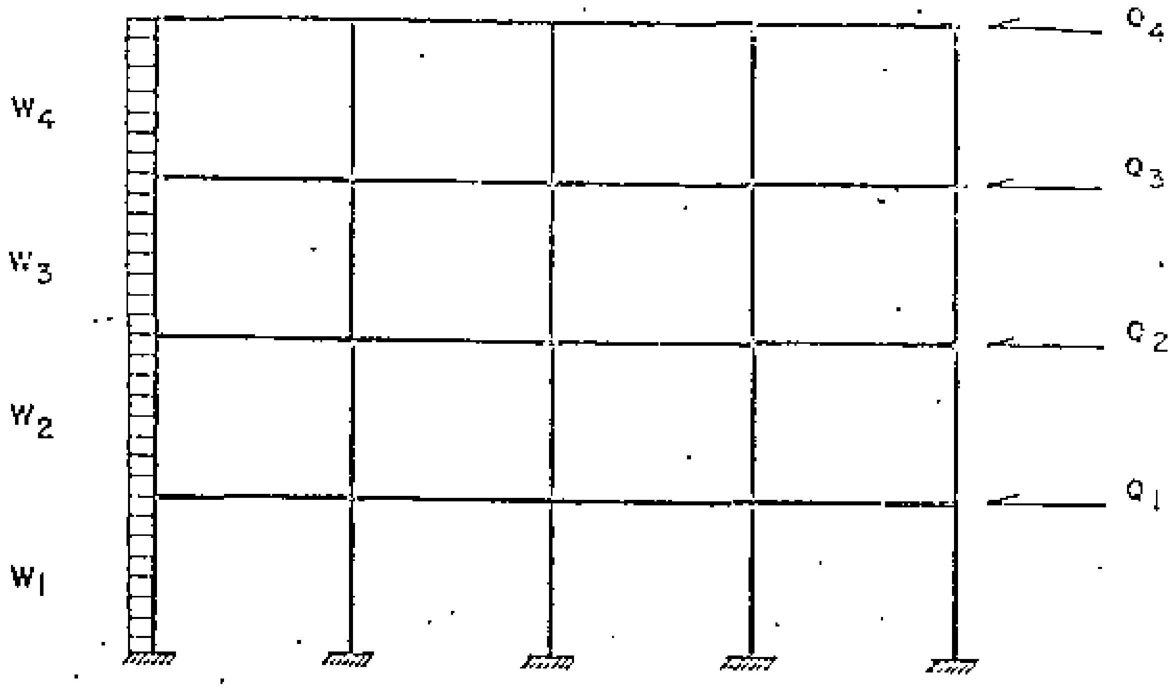
DIAFRAGMA
FLEXIBLE



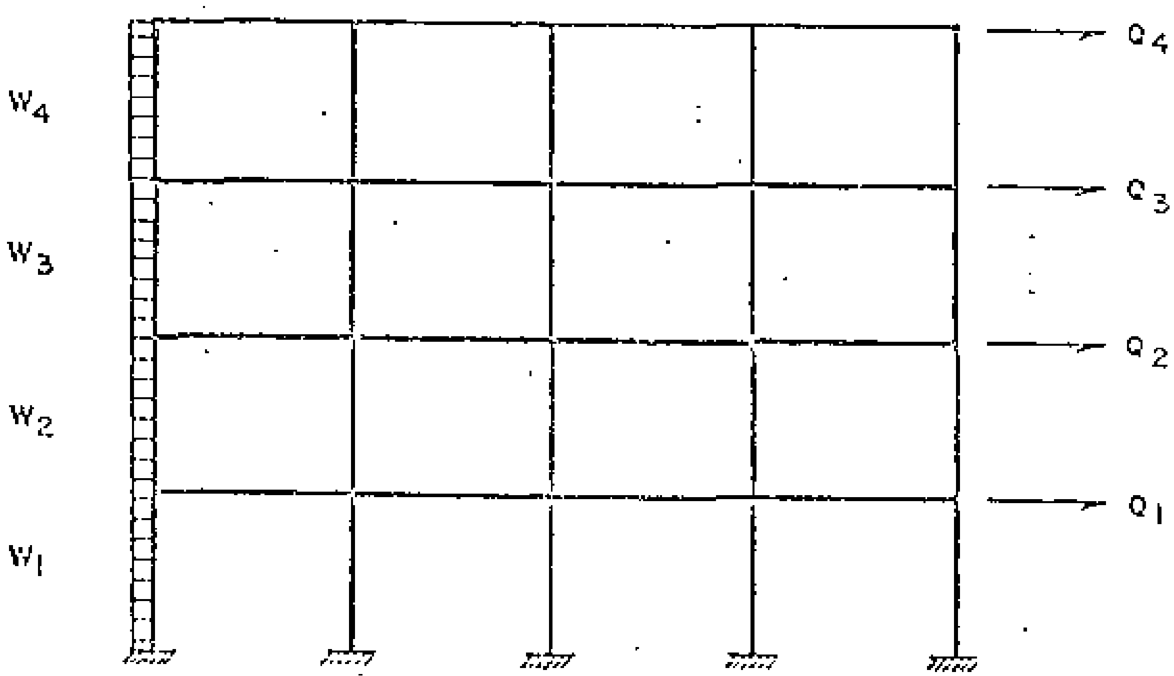
DIAFRAGMA
RIGIDO.

$$F_i = \frac{K_i}{\Sigma K_i} P$$

DISTRIBUCION DE FUERZAS DE VIENTO.
ANALISIS DE DIAFRAGMAS.
FIG. 22

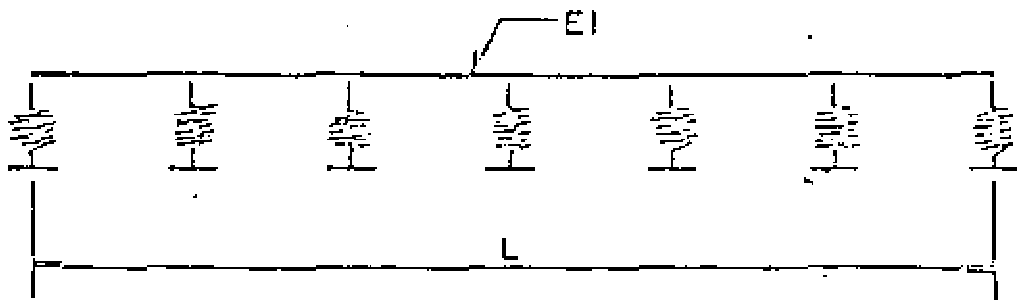


MARCO FLEXIBLE

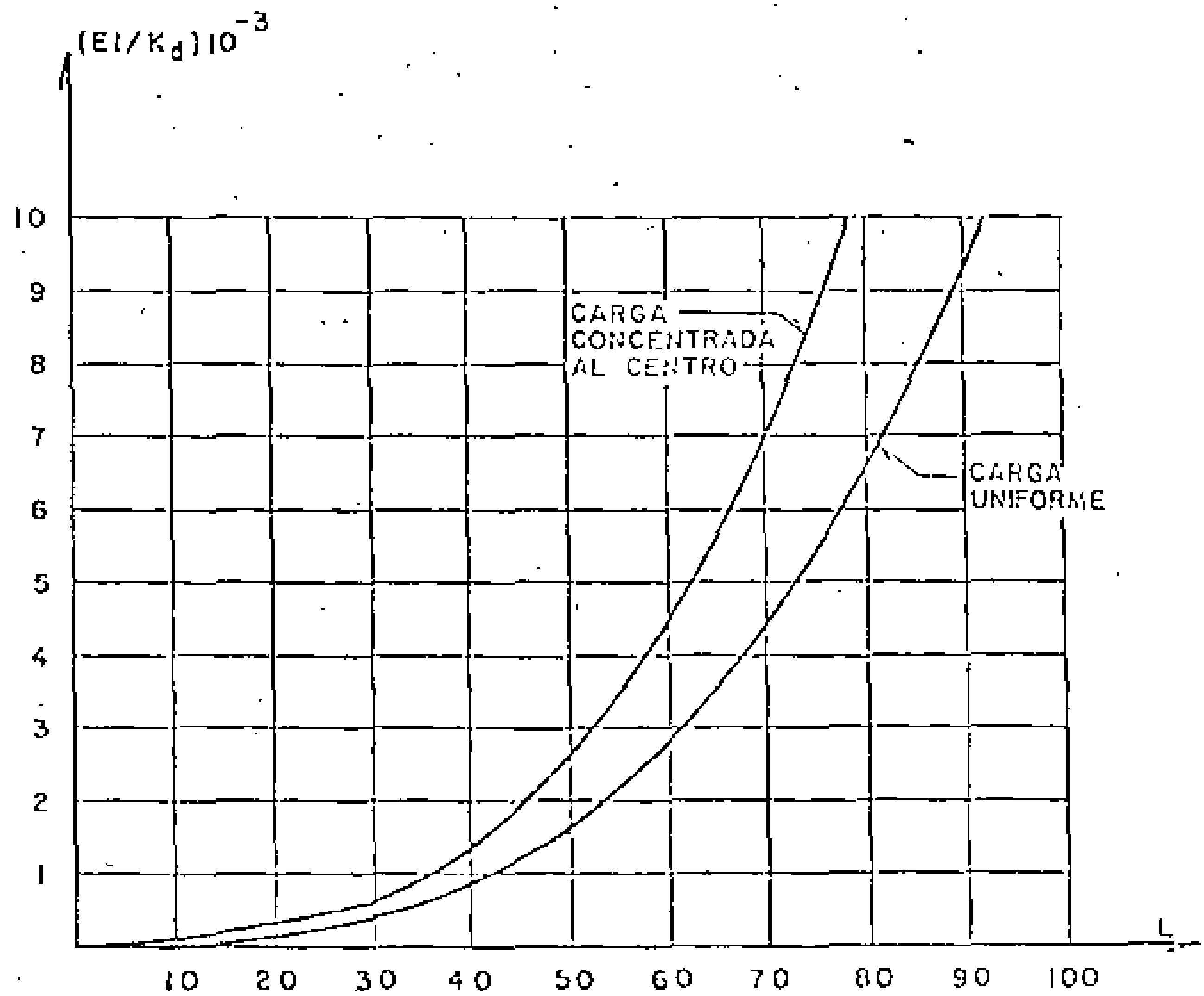


MARCO RIGIDO

DISTRIBUCION DE FUERZAS DE VIENTO
ANALISIS DE MARCOS
FIG. 23

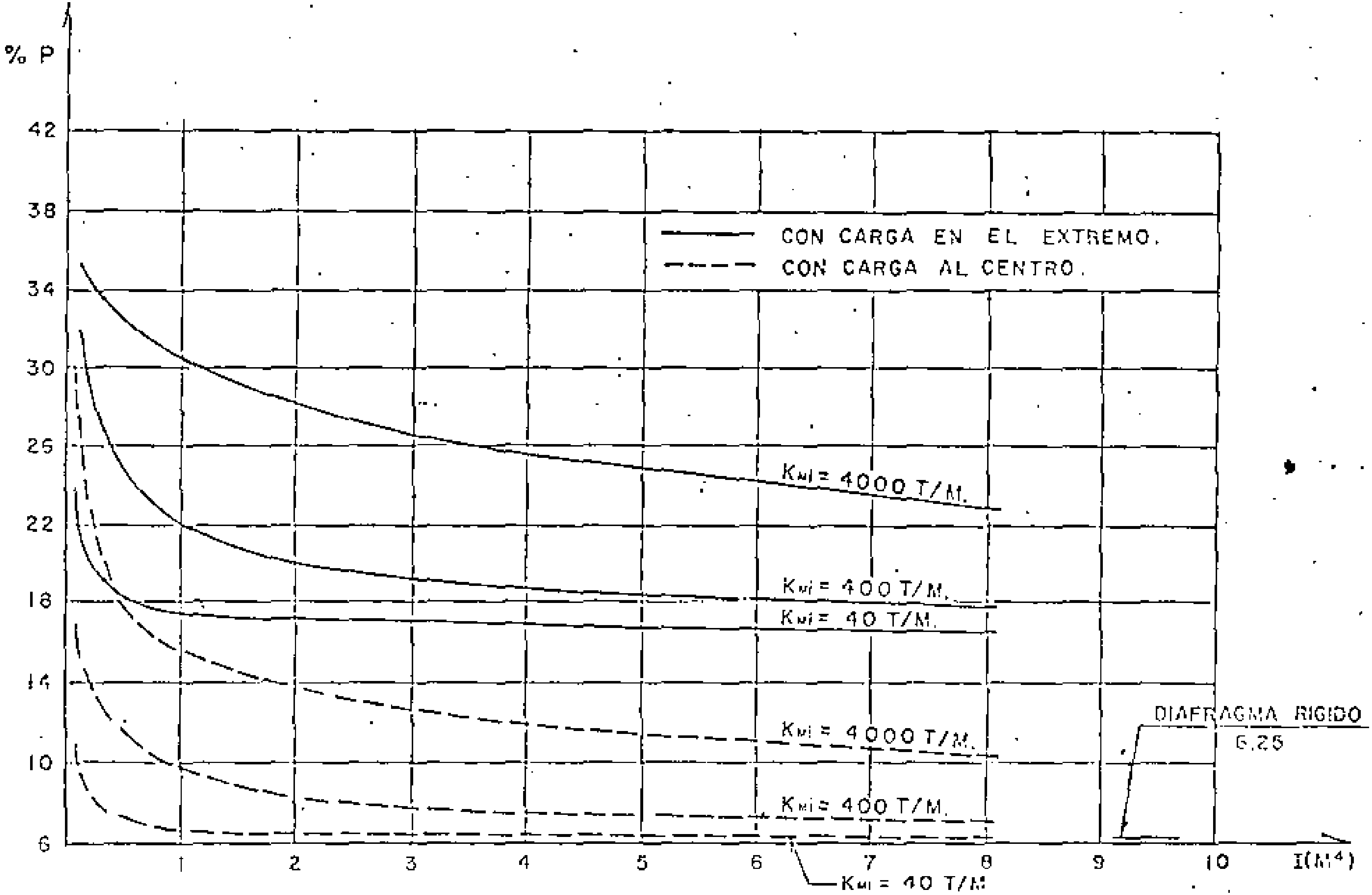


MODELO ESTRUCTURAL.

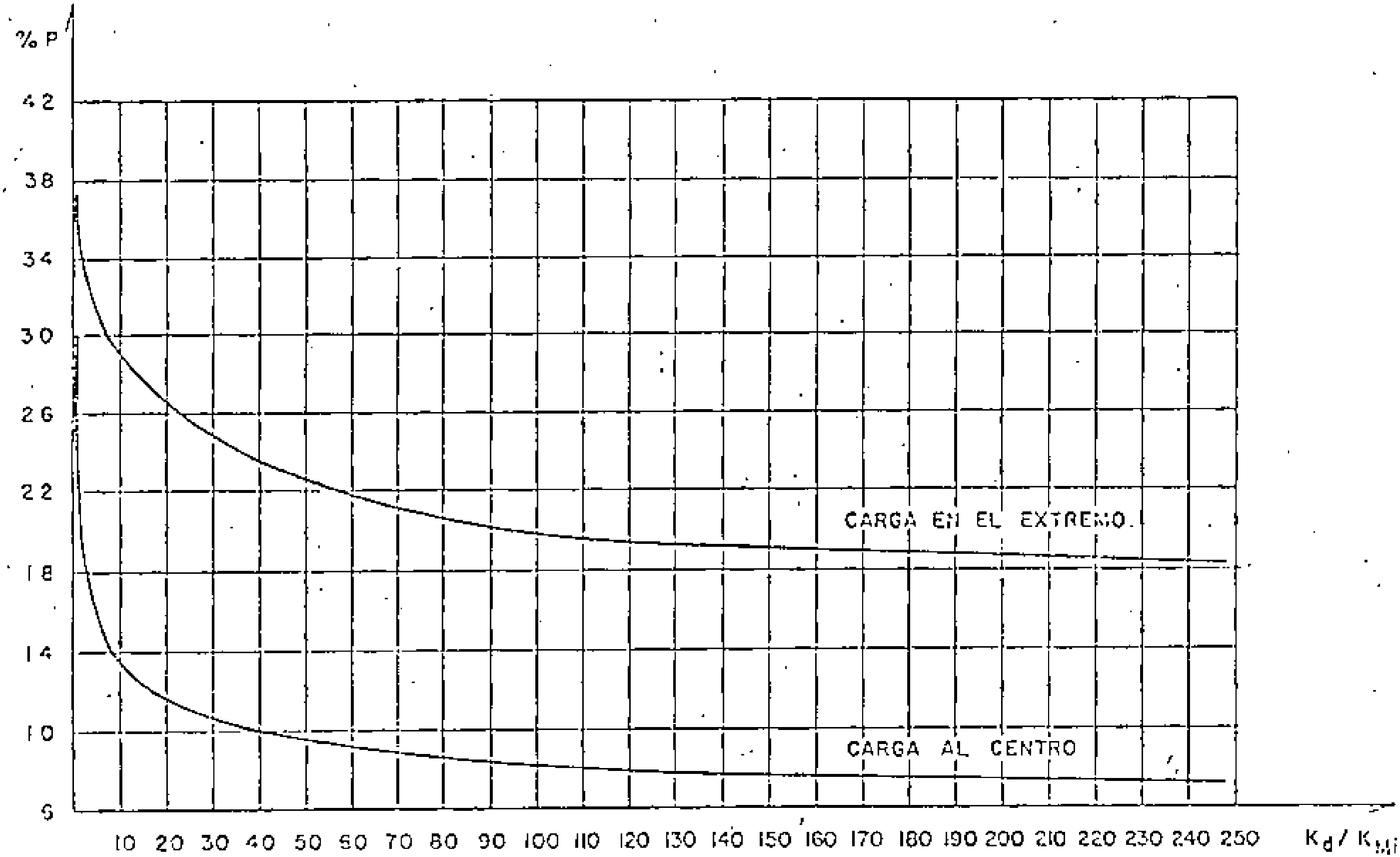


RELACION DE RIGIDECES VIGA EQUIVALENTE A DIAFRAGMA.

FIG. 24

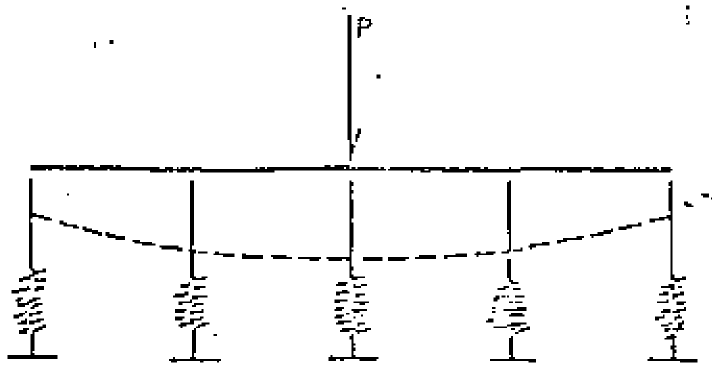


PORCENTAJE DE LA CARGA CONCENTRADA
QUE ABSORBE EL MARCO CARGADO.
FIG. 25

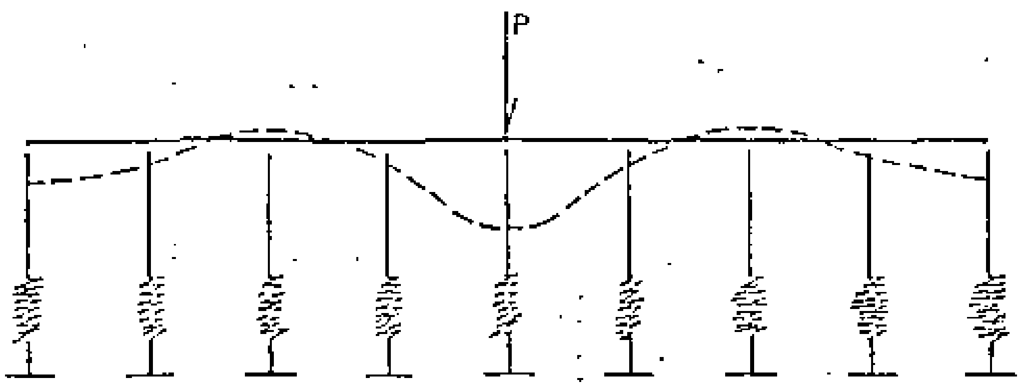


PORCENTAJE DE LA CARGA CONCENTRADA QUE ABSORBE
EL MARCO CARGADO FINAL

FIG. 26

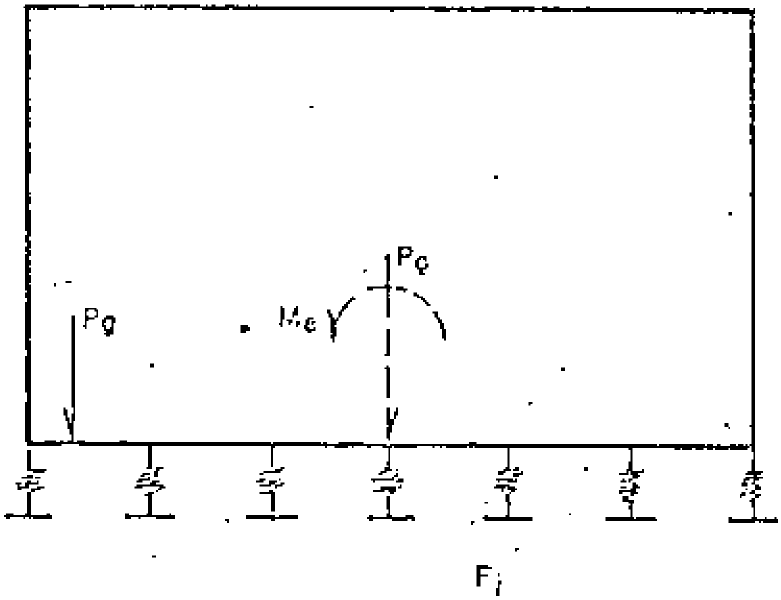


DIAFRAGMA SEMI—RIGIDO o FLEXIBLE
CON POCOS MARCOS



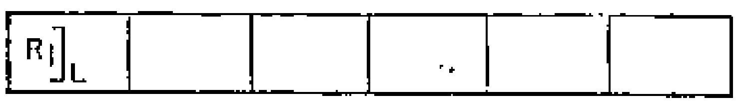
DIAFRAGMA FLEXIBLE

CONFIGURACION DE DESPLAZAMIENTOS
LATERALES DEL DIAFRAGMA
FIG. 27

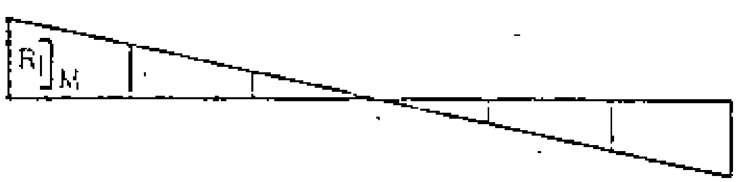


EFECTO LINEAL DE P_g

$$R_i = \frac{P_g}{N}$$



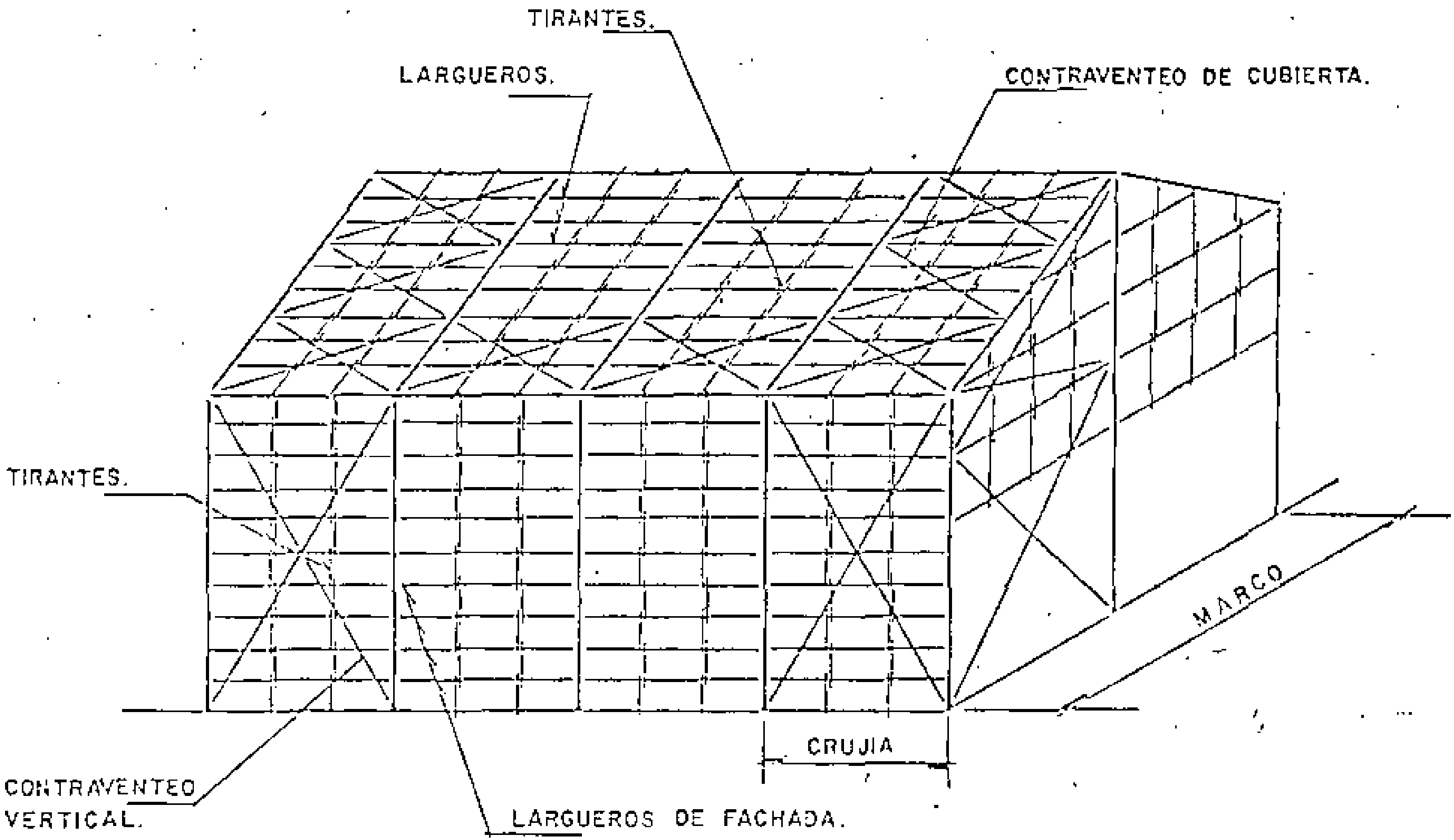
+



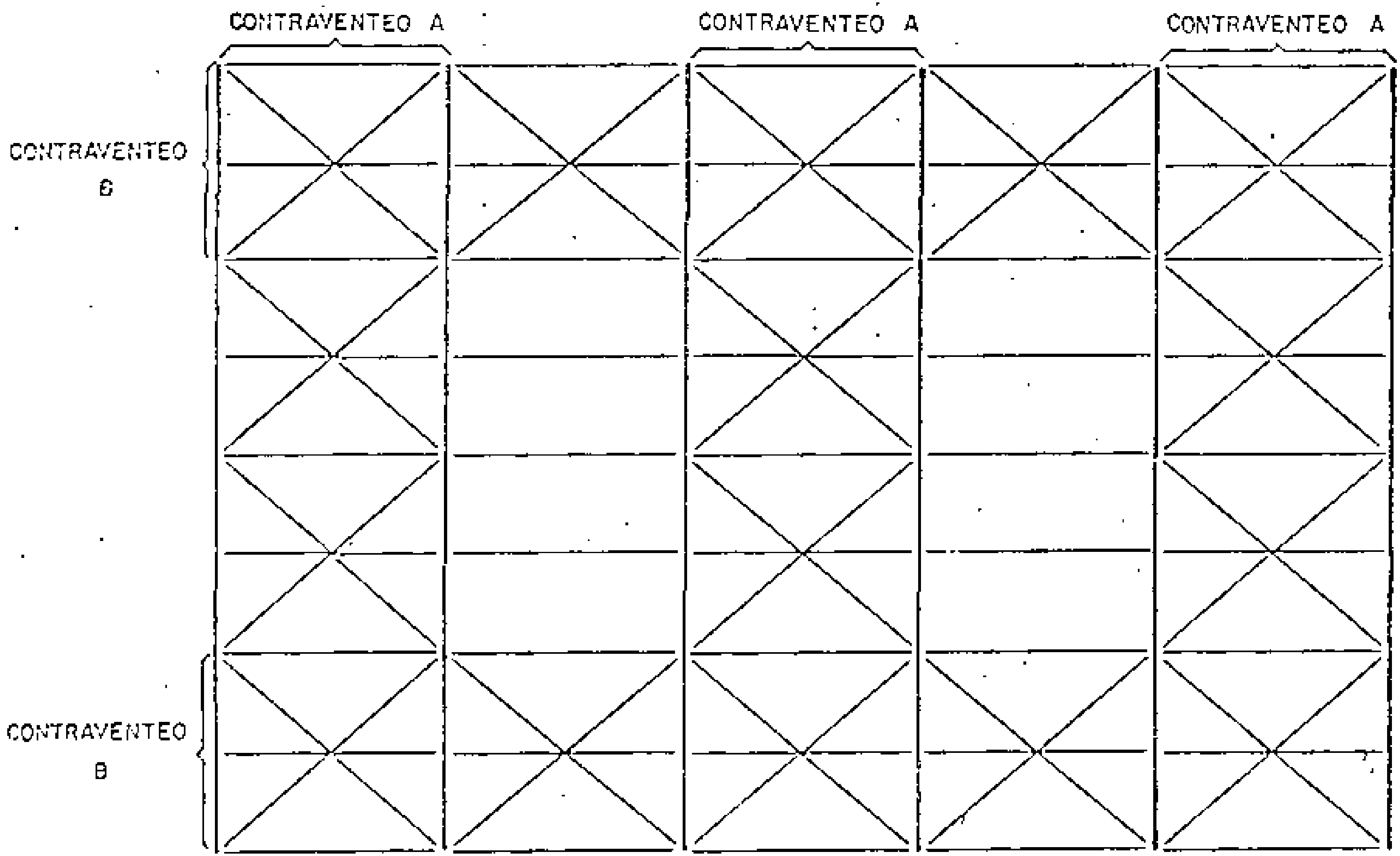
EFECTO DE M_g

VALORES DE % P EN MARCO EXTREMO			
No.	EFECTOS DE M	EFECTOS DE P	TOTAL
2	0.5 P	0.5 P	1.0 P
3	0.4 P	0.33 P	0.73 P
4	0.32 P	0.25 P	0.57 P
5	0.27 P	0.20 P	0.47 P

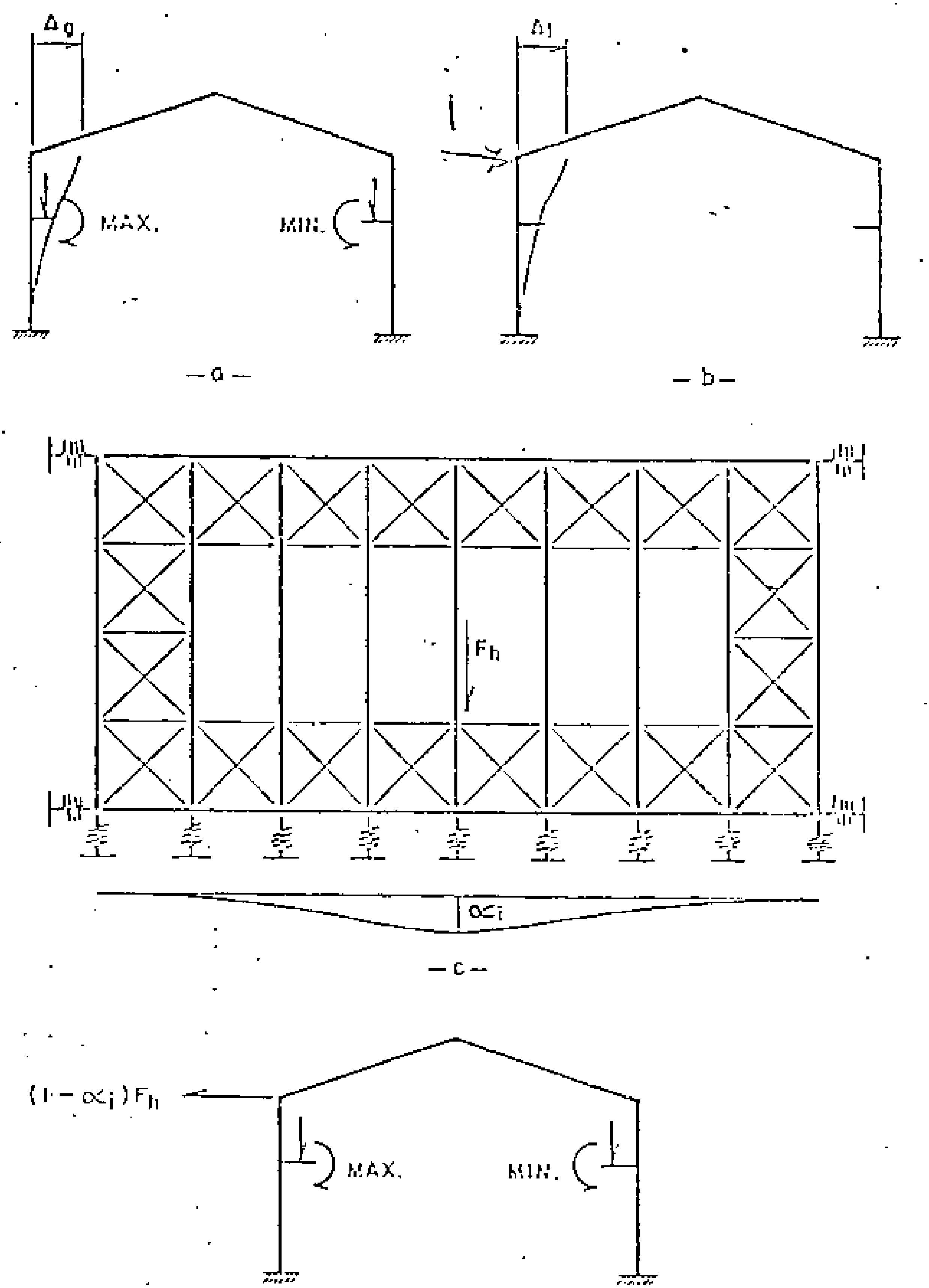
DISTRIBUCION DE FUERZA DE GRUA
DIAFRAGMA RIGIDO
FIG. 26



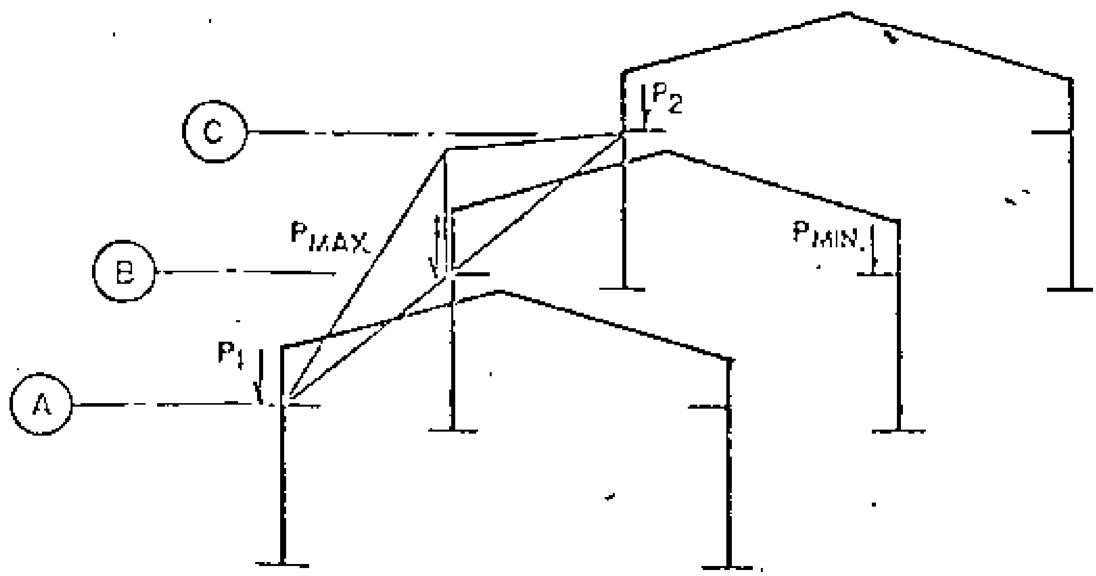
ESTRUCTURACION DE UNA NAVE INDUSTRIAL
FIG. 29



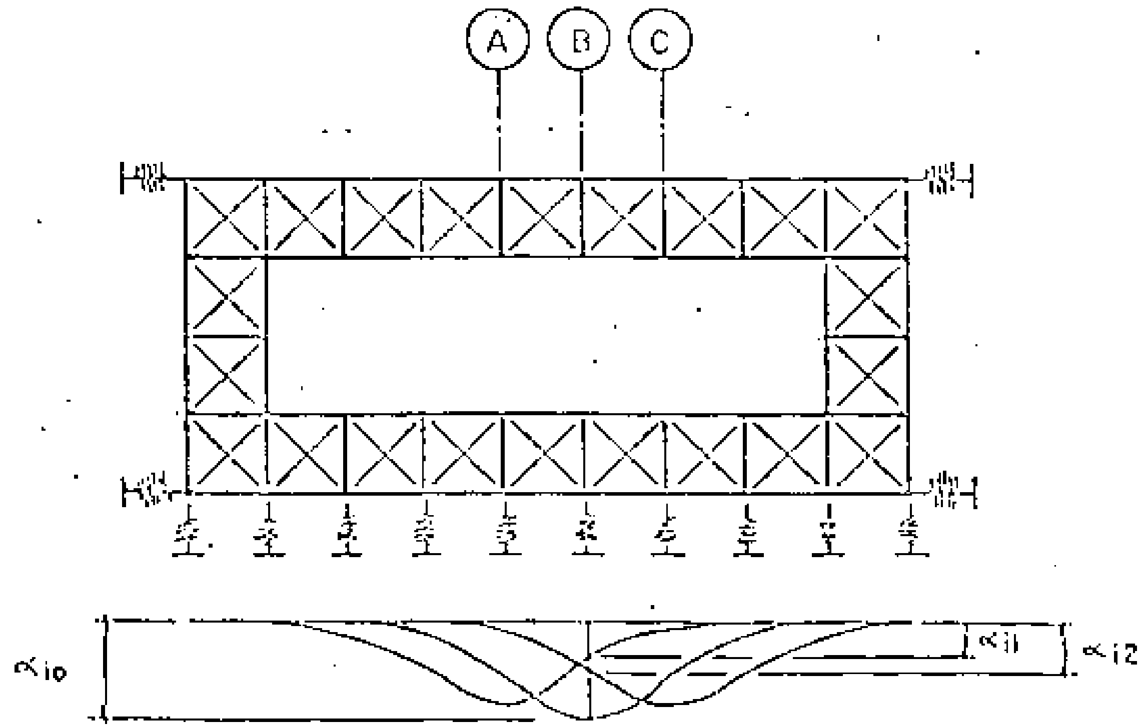
TIPOS DE CONTRAVENTEOS HORIZONTALES.
FIG. 30



COMPATIBILIDAD DE DESPLAMIENTOS LATERALES.
DEBIDO A CARGAS VERTICALES.
FIG. 31



— a —

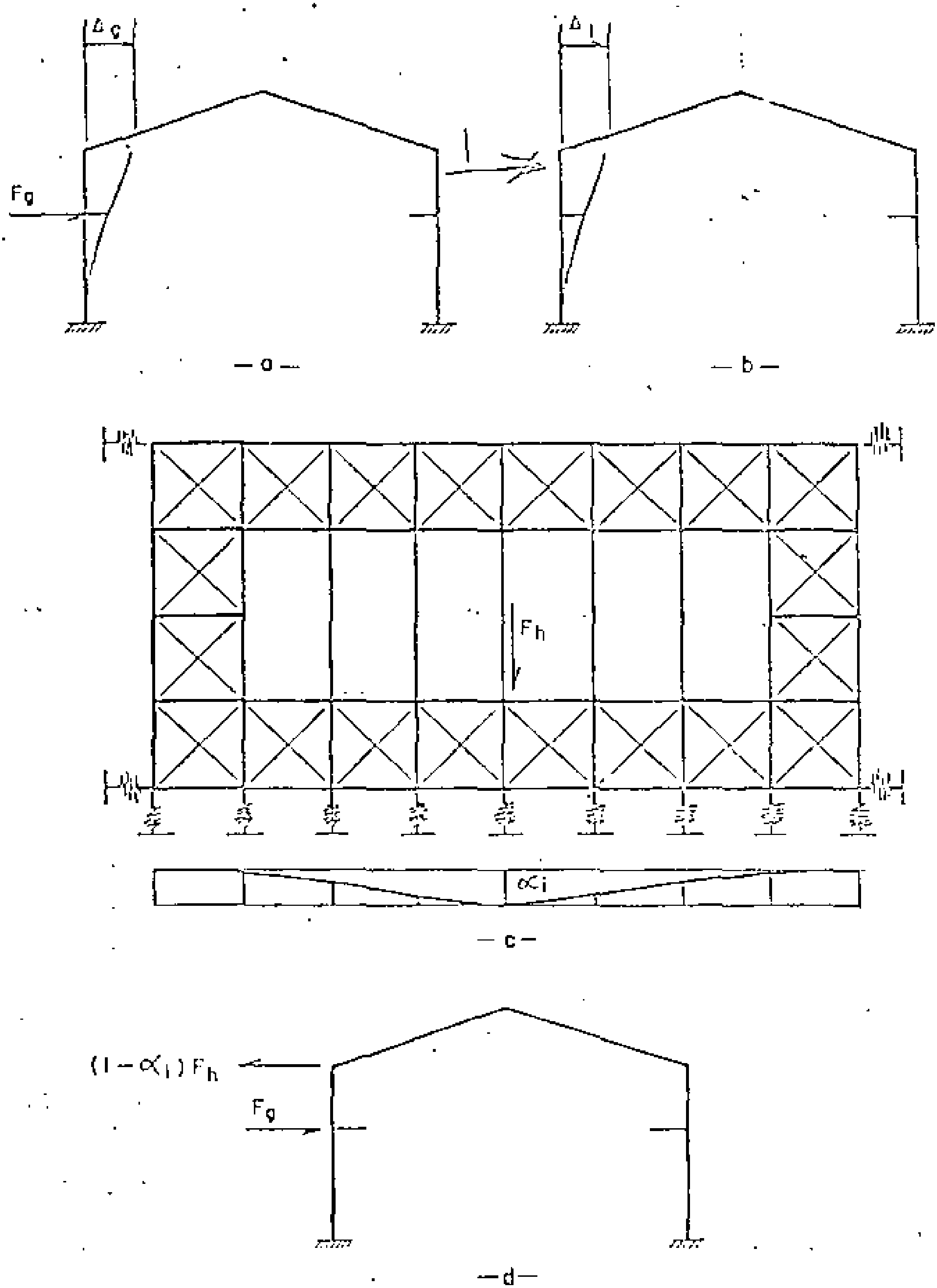


— b —

$\beta_i = P_i / P_{MAX.}$

$\alpha_i = \alpha_{io} + \beta_i \alpha_{i1} + \beta_2 \alpha_{i2}$

EFECTOS DE LAS DIMENSIONES REALES
DEL PUENTE DE GRUA
FIG. 32



COMPATIBILIDAD DE DESPLAZAMIENTOS LATERALES DEBIDO A CARGAS HORIZONTALES.

FIG. 33

CONSIDERACIONES PARA INCORPORAR LOS EFECTOS DE INTERACCION SUELO- ESTRUCTURA.

Mauricio Nanes G.

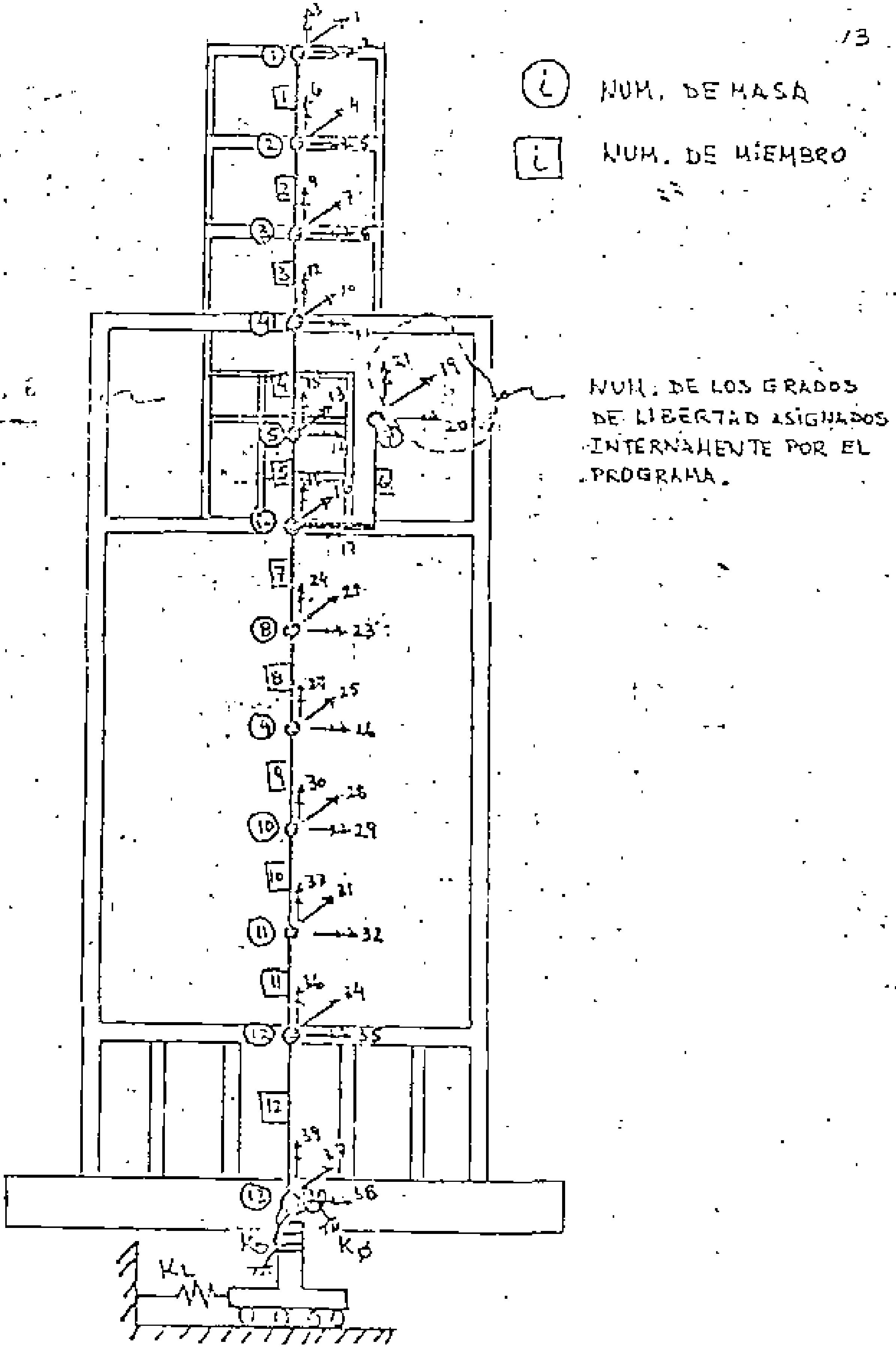

13
NUM. DE MASA
NUM. DE MIEMBRO
NUM. DE LOS GRADOS DE LIBERTAD ASIGNADOS INTERNAMENTE POR EL PROGRAMA.
K_L
K_ϕ

FIG. 2

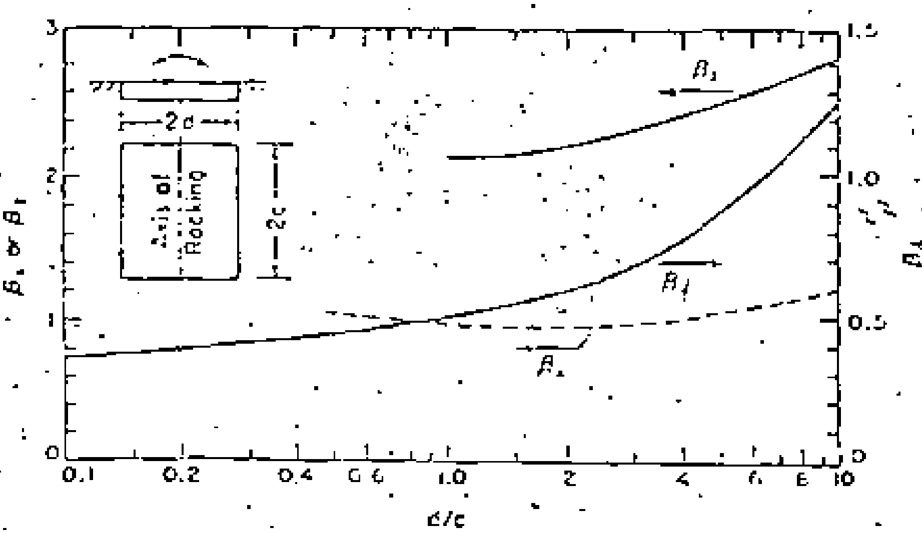
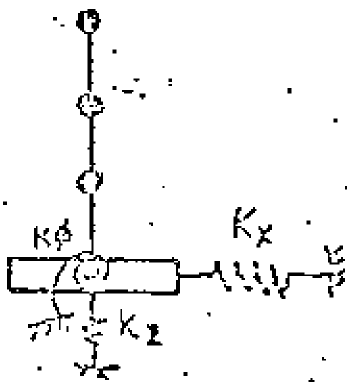


Figure 10-16. Coefficients β_x, β_z and β_ϕ for rectangular footings (after Whitman and Richart, 1967).



K_θ – TORSIÓN

$K_\phi = K_\psi$ CABECEO

$$K_x = 4(1+\mu)\, G\, \beta_x \sqrt{c \cdot d}$$

$$K_\phi = \frac{G}{1-\mu}\, \beta_\psi\, 8cd^2$$

$$K_\theta = \frac{16}{3}\, G\, r_0^3 \quad ; \quad r_0 = \sqrt[4]{\frac{16cd\,(c^2+d^2)}{6\pi}}$$

$$K_z = \frac{G}{1-\mu}\, \beta_z \sqrt{4cd}$$

G = MÓDULO DE ELASTICIDAD AL CORTANTE DINÁMICO

μ = RELACIÓN DE POISSON

FIG. 3

PARA CIMENTACIONES CIRCULARES

$$K_x = \frac{32(1-\mu)G r_o}{7-8\mu} \qquad \text{HORIZONTAL}$$

$$K_\phi = \frac{8 G r_o^3}{3(1-\mu)} \qquad \text{CABECEO}$$

$$K_\theta = \frac{16}{3} G r_o^3 \qquad \text{TORSION}$$

$$K_z = \frac{4 G r_o}{1-\mu} \qquad \text{VERTICAL}$$

FIG. 4

TORRE DE PRILADO

DESCRIPCION:

Cascarón cilindrico vertical de 14.5 m de diametro y de 71.6 m de altura ($H/D \approx 5$) y 0.30 m de espesor de concreto, con cuatro (4) losas de concreto de operación (Diafragmas rigidizantes.).

Sobre el remate del cascarón, se levanta una plataforma metalica de cuatro niveles de 17.9 m de altura.

Además, sobre una losa-Diafragma intermedia, se soporta otra plataforma metalica desligada del fuste.

La cimentación fué un tanto especial debido a las características del suelo soportante (Baja capacidad de carga - muy deformable). Se estudió la posibilidad de desplante por superficie, la cual se elimino de inmediato y se optó por una cimentación profunda a base de pilotes de punta.

LOCALIZACION:

Lázaro Cardenaz Michoacan - Zona Sísmica D

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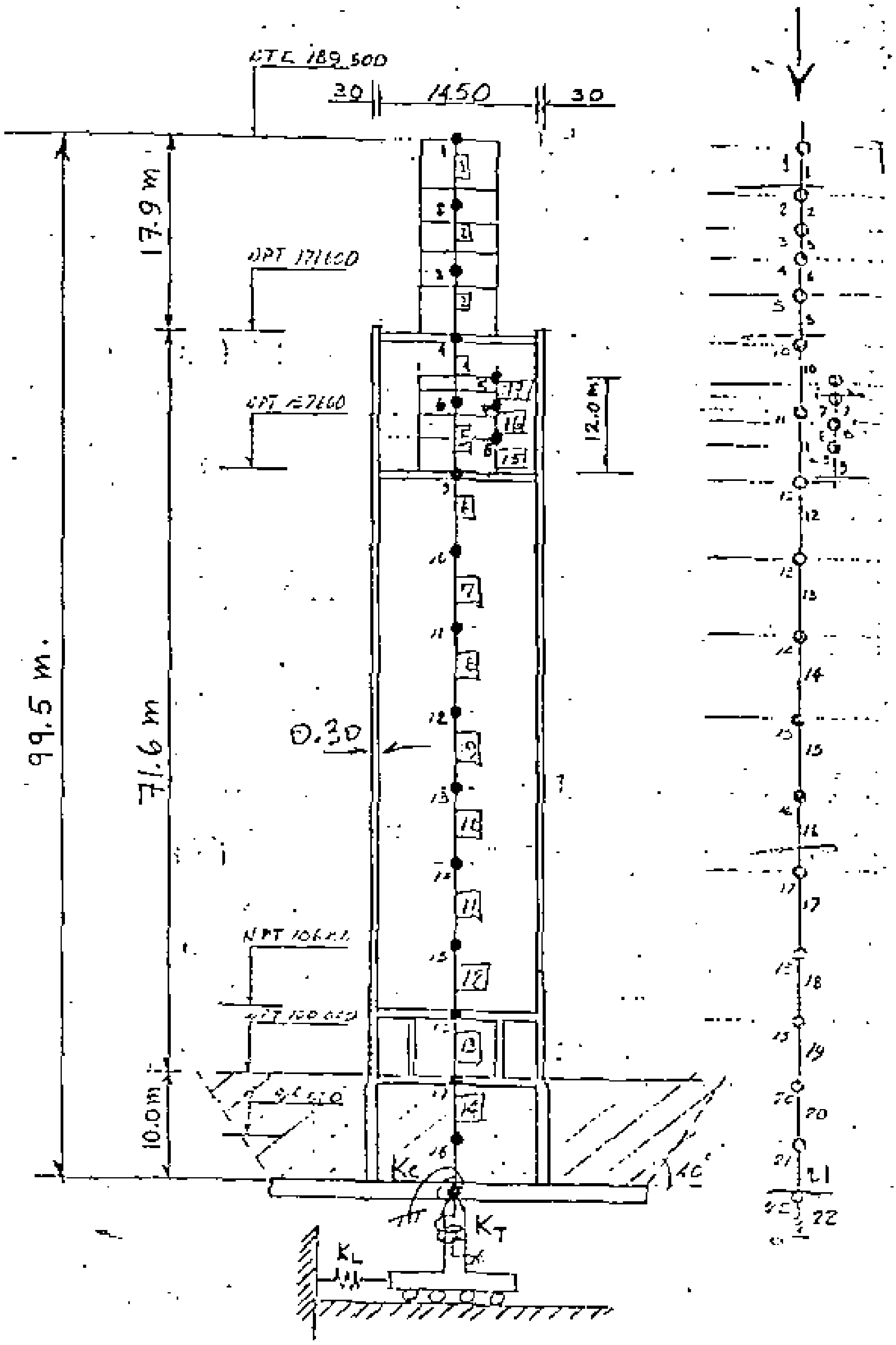
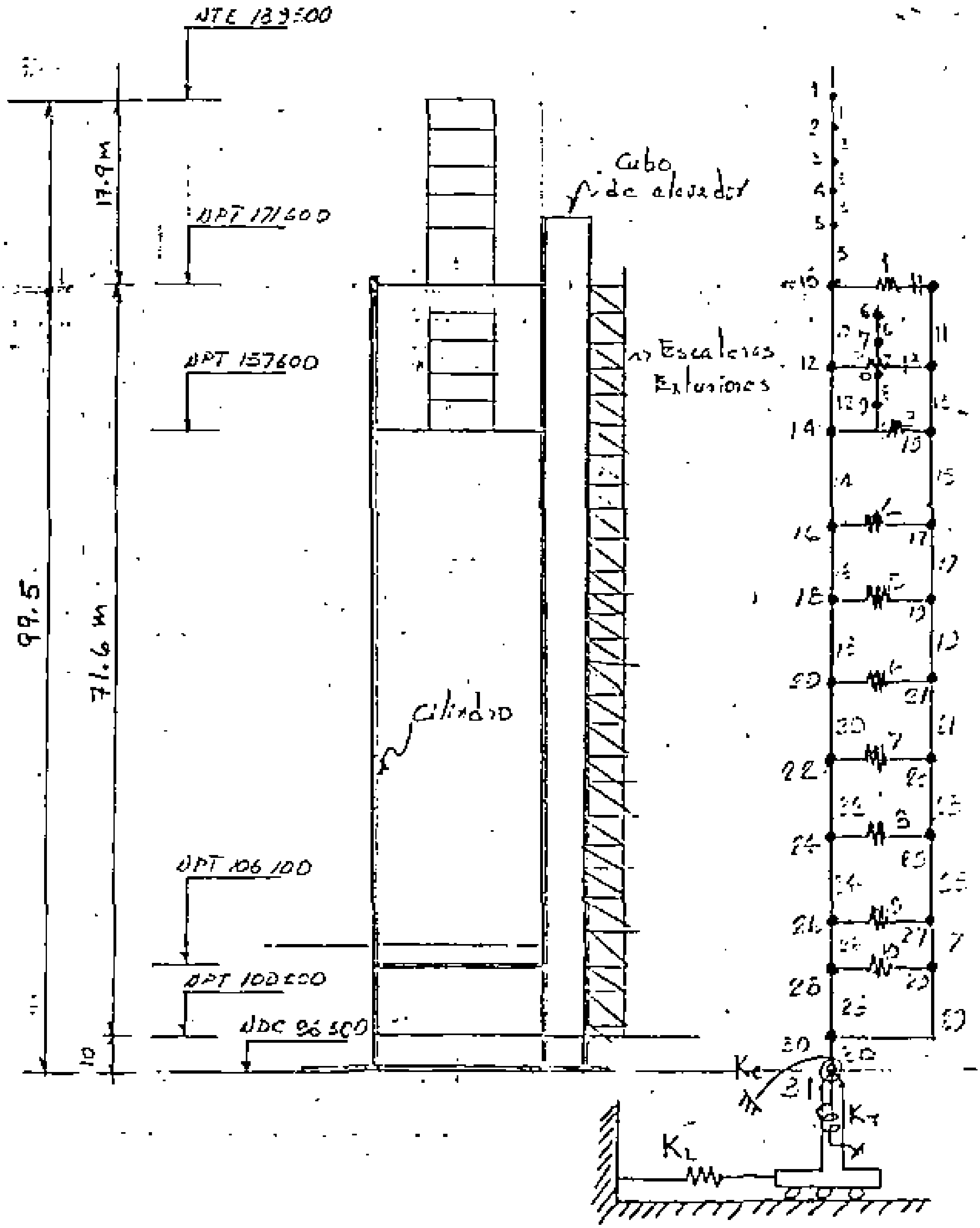


TORRE DE PRILADO

CASO 4


NTE 139500
NPT 171600
NPT 157600
NPT 106100
NPT 100500
NDC 96500
17.9 m
71.6 m
99.5
10
Cubo de elevador
Escaleras Exteriores
Cilindro
Ke
KL
KT

ANALISIS:

Se efectuó un análisis sísmico dinámico modal espectral y se consideró la interacción Suelo-Pilote-Estructura.

Debido al tipo de cimentación y a las características geométricas y de rígidez del fuste, la interacción es de primordial importancia.

Se procedió de la siguiente manera:

1. Se estimo el número de pilotes requeridos por carga vertical y su distribución aproximada. (8360 Ton.).

2. Se determinó el espesor preliminar de la losa-cimentación.

3. Calculó las rígideces de tres (3) resortes de interacción.

4. Utilizando estos valores de rigideces como mínimos, se efectuó un estudio paramétrico variando estas rígideces sin preocuparse por la distribución real de pilotes correspondientes.

5. Una vez estudiados los resultados del estudio paramétrico se eligió la "mejor" solución, se obtuvo la distribución equivalente de pilotes y se procedió con el diseño.

NOTA: El estudio paramétrico se efectuó con un modelo simplificado y el caso elegido se analizó con un modelo completo.

5

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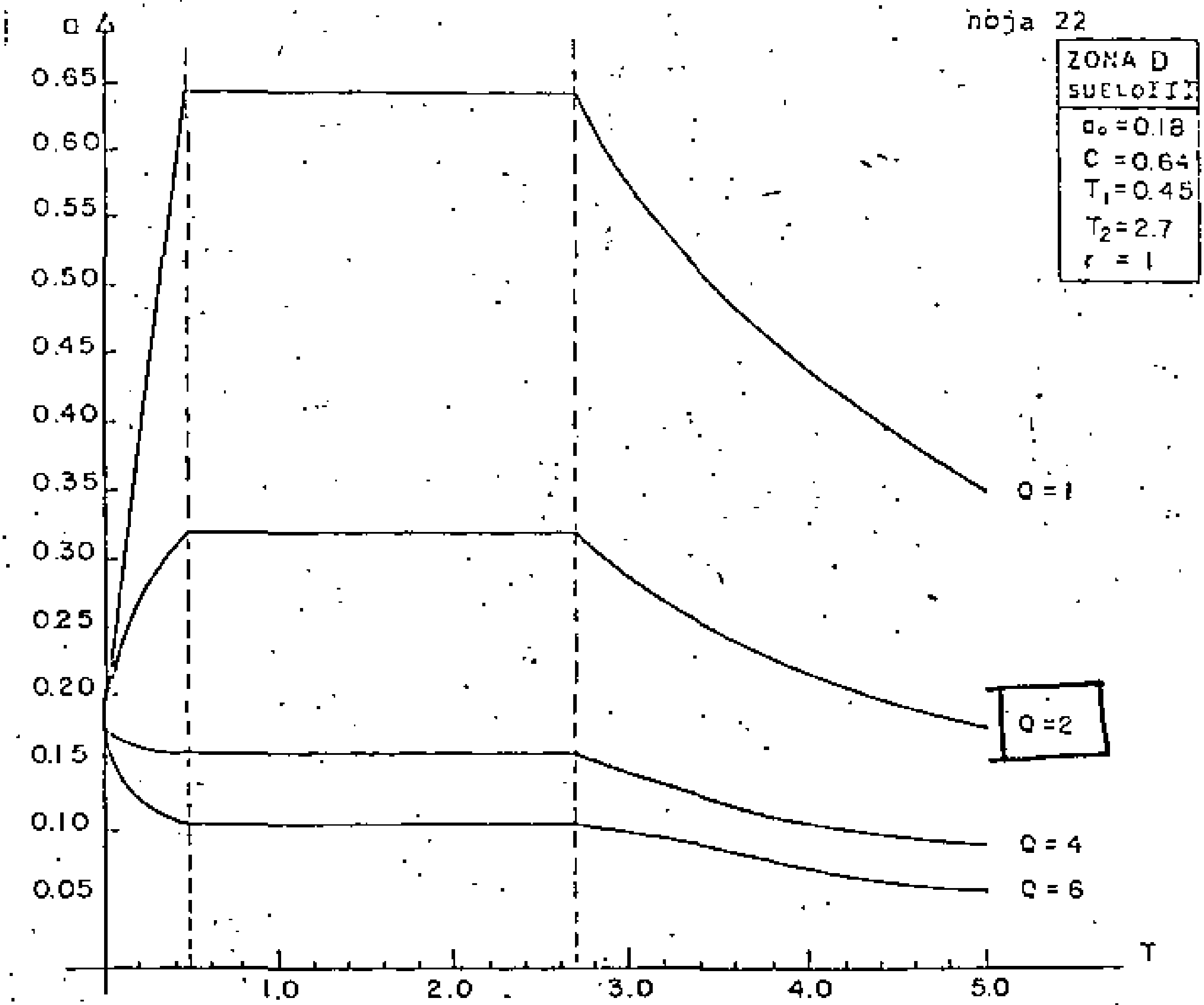
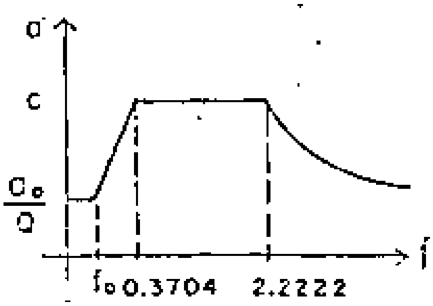
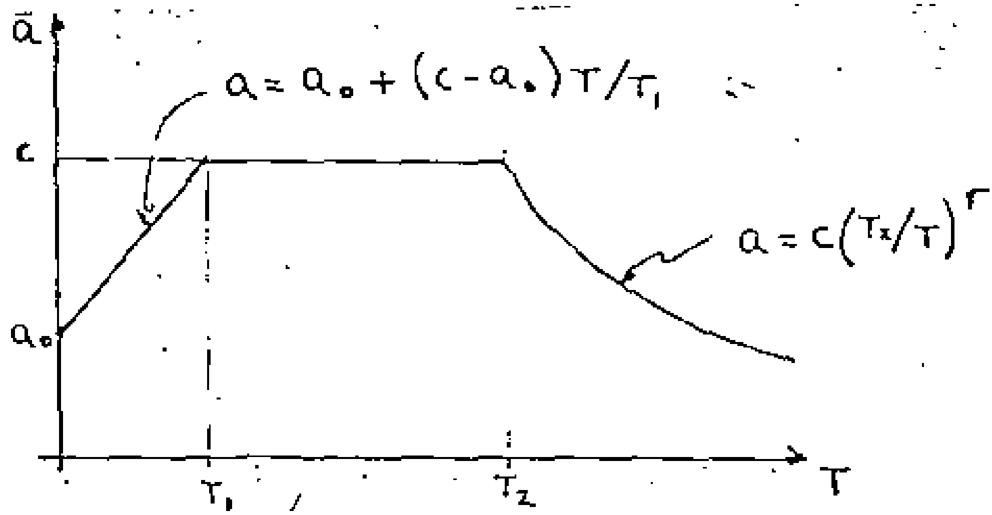


f	T	a			
		Q = 1	Q = 2	Q = 4	Q = 6
0.2	5.0	0.34560	0.17260	0.09640	0.05760
0.3704	2.7	0.64	0.32	0.16	0.10667
2.2222	0.45	0.64	0.32	0.16	0.10667
2.75	0.3636	0.55172	0.30514	0.16112	0.10946
3.50	0.2857	0.47206	0.28874	0.16251	0.11308
4.50	0.2222	0.40716	0.27256	0.16408	0.11737
5.50	0.1818	0.36586	0.26059	0.16539	0.12114
10.00	0.1000	0.28222	0.23091	0.16933	0.13368
50.00	0.0200	0.20044	0.19191	0.17686	0.16400



$f_0 = 0.1042$

OBTENCION DE UNA FAMILIA DE ESPECTROS A PARTIR DE UN ESPECTRO DEL RCDF



$a_o = 0.06$ $\quad T_1 = 0.8$ $\quad r = 1$

$c = 0.24$ $\quad T_2 = 3.3$

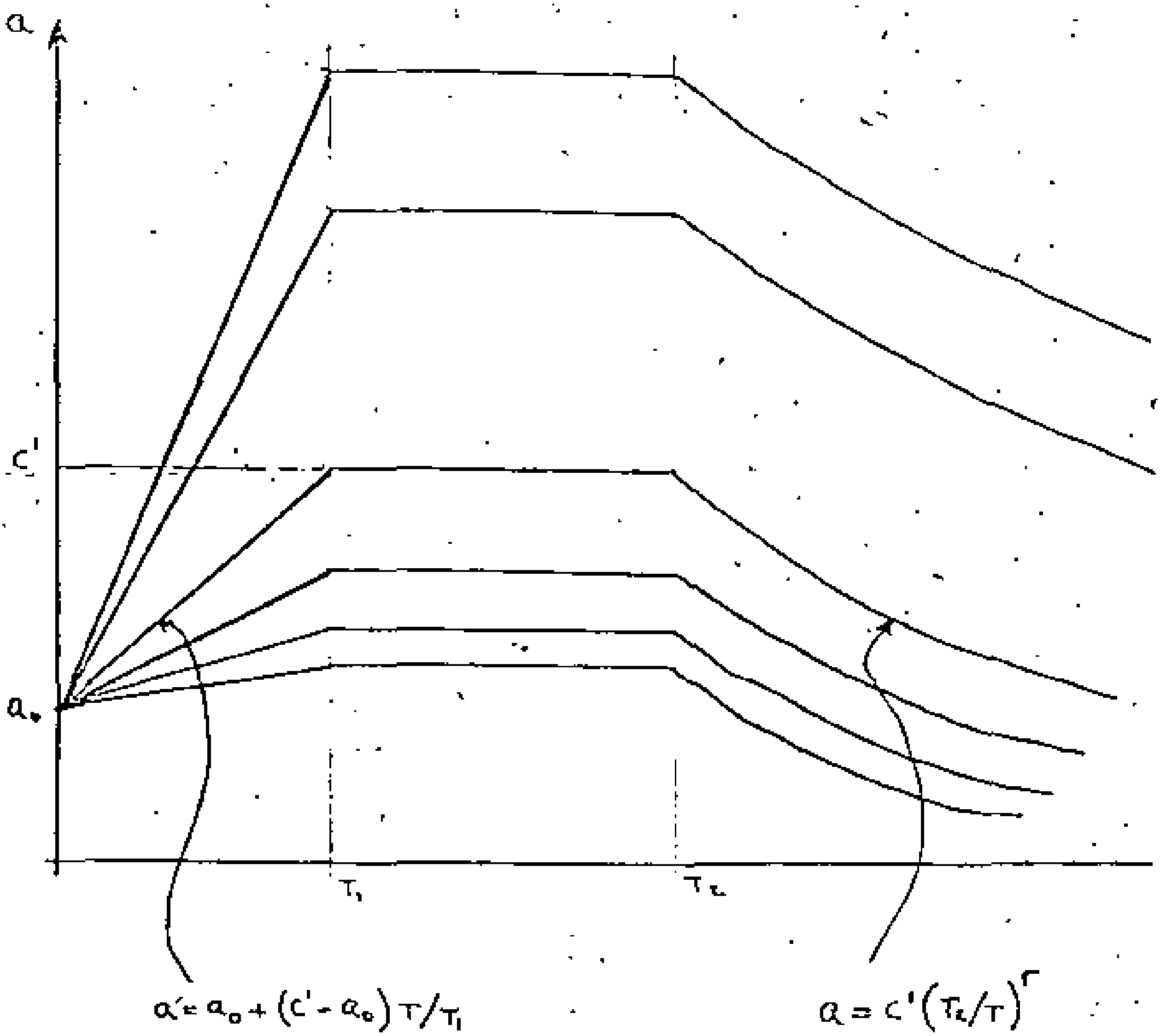
La normalización de los factores de amplificación de Newmark se indica a continuación

ξ %	FACTOR DE AMPLIFICACION	
	Newmark (F)	Normalizado (f)
0	6.4	2.46
0.5	5.8	2.23
1	5.2	2.00
2	4.3	1.65
5	2.6	1.00
7	1.9	0.73
10	1.5	0.58
20	1.2	0.46

Para el espectro correspondiente a un amortiguamiento ξ, la única variable será la aceleración espectral máxima c, que debe mutliplicarse por el factor de amplificación normalizado f.

$$c' = c \cdot f$$

Se obtiene así la familia de espectros:



donde $c' = c.f$

RIGIDECES DE RESORTES EQUIVALENTES DE INTERACCION PARA CIMENTACIONES PILOTEADAS

- Decisión de inclinar o no pilotes depende de varios factores:

  - ¿ Se puede tomar en cuenta la interacción Suelo-Pilote ? o sea, examinar relación de rigideces Suelo/Pilote.

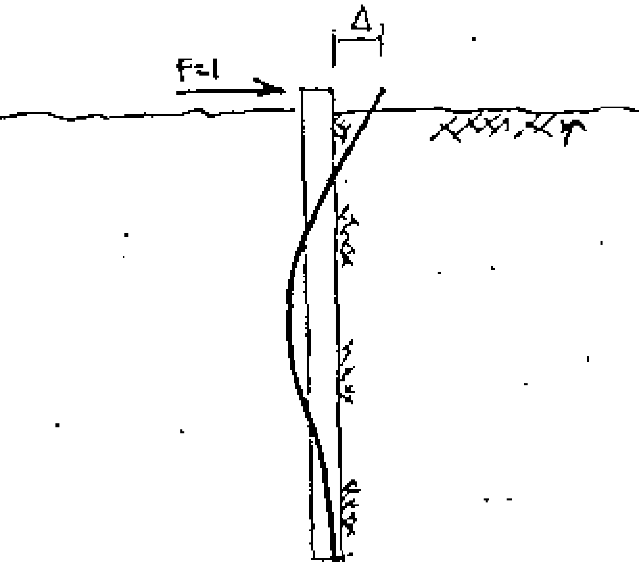
$$\lambda = \sqrt[4]{\frac{k}{4EI}} \qquad (\lambda L \geq \pi/4)$$

  - Valor requerido de la rigidez del resorte horizontal para optener un nivel aceptable de respuesta dinámica del conjunto Suelo-Estructura.

  - Se utilizarían pilotes inclinados unicamente cuando sean de punta (y no de fricción), para poder contar con la componente horizontal de la rigidez axial del pilote.

INTERACCION SUELO-PILOTE:

En este caso se consideró un comportamiento elastico del suelo, unque en general se puede tomar en cuenta el comportamiento inelastico (Curvas P-y Reese, Matlock y otros autores).

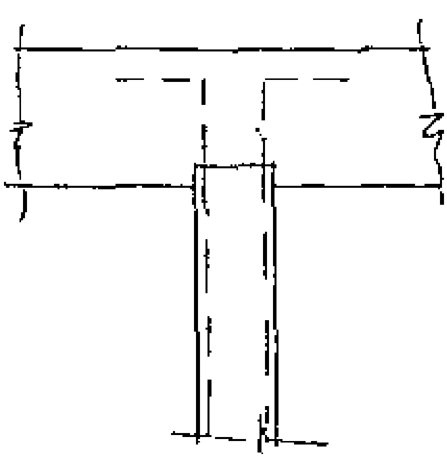


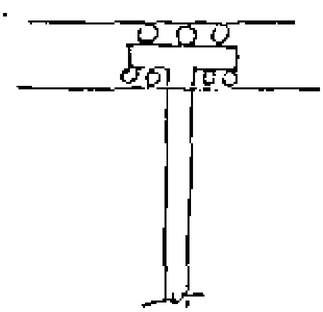
" Viga" sobre cimentación elástica:

$$K_x\Big]_u = \frac{1}{\Delta}$$

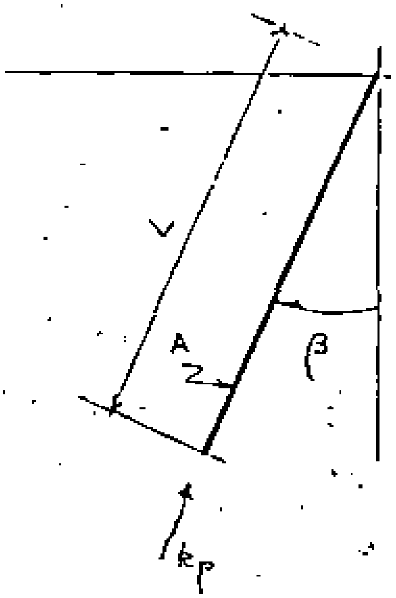
Rígidez horizontal unitaria por pilote-Suelo.

Se deben considerar las condiciones de frontera superior de manera que reflejen lo más cercanamente posible la realidad, por ejemplo:





RIGIDEZ HORIZONTAL DE PILOTES INCLINADOS:



"Punta" Indeformable:

$$K_x\Big]_u^i = \frac{EA}{L}\,\text{Sen}^2\beta$$

"Punta" Deformable:

$$K_x\Big]_u^i = \frac{1}{\left(\frac{1}{kp} + \frac{L}{EA}\right)}\,\text{Sen}^2\beta$$

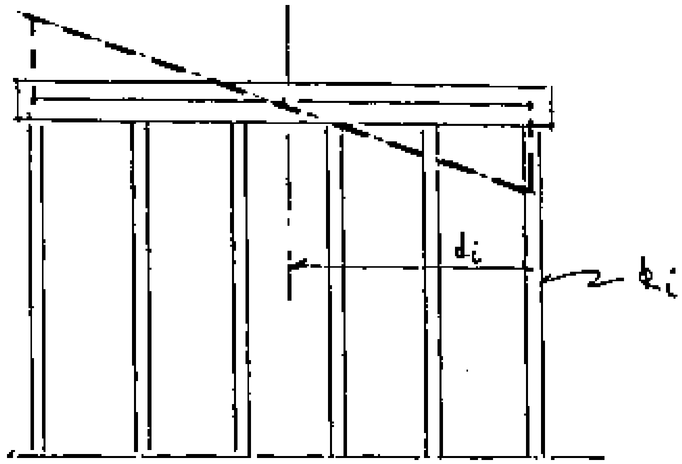
RIGIDEZ HORIZONTAL TOTAL:

$$K_x = ni \cdot K_x\Big]_u^i + n \cdot K_x\Big]_u$$

ni..... N° de pilotes inclinados

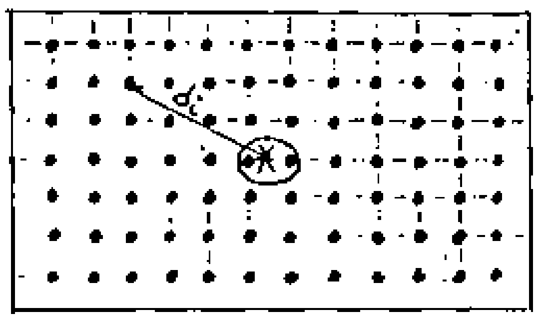
n ..... N° total de pilotes

RIGIDEZ DE CABECEO



$$K_\psi = \sum k_i\, d_i^2$$

RIGIDEZ DE TORSION:



$$K_\theta = \sum K_x]_u \cdot d_i^2$$

VALORES UTILIZADOS PARA:

— DUCTILIDAD: Q=2.0

— AMORTIGUAMIENTOS:

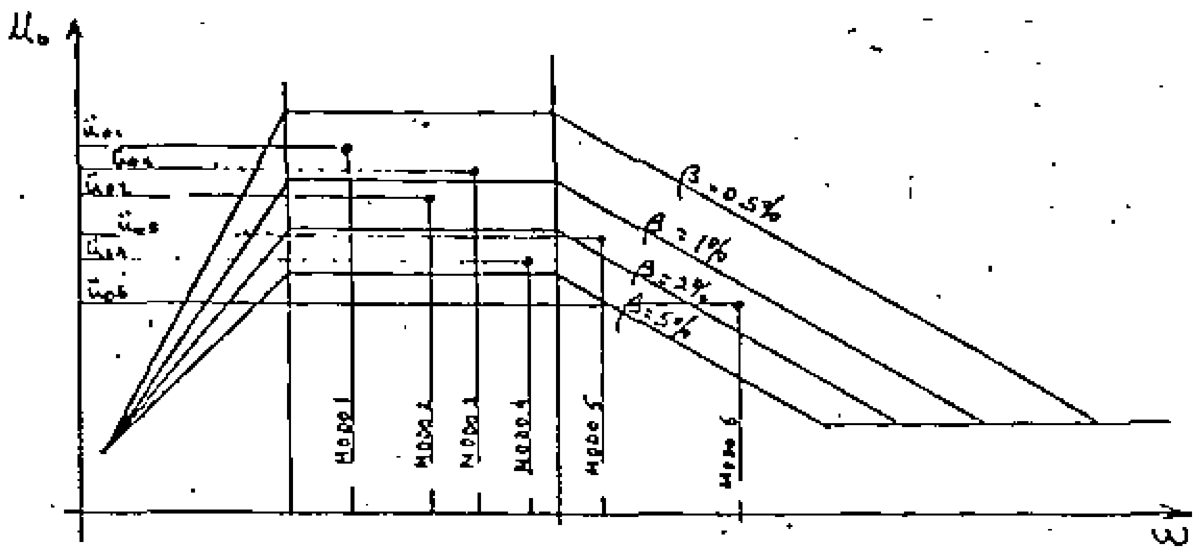
ESTRUCTURAL $\xi = 3\%$

INTERACCION

HORIZONTAL LINEAL ξ = 20%

CABECEO ξ = 6%

TORSION ξ = 20%



ACELERACIONES ESPECTRALES MODALES

CALCULO DEL AMORTIGUAMIENTO PROMEDIO PESADO

TABLA 13.1. *Valores típicos de amortiguamiento en las instalaciones de reactores nucleares.* Según Newmark y Hall (1969) y Newmark (1969b)

Nivel de esfuerzo	*Tipo y condición de la estructura*	*Porcentaje del amortiguamiento crítico*
1. Bajo, muy por abajo del límite de proporcionalidad, esfuerzos inferiores a $\frac{1}{4}$ del límite de fluencia	Tuberías vitales	0.5
	Acero, concreto reforzado o presforzado, madera; sin grietas; sin deslizamientos en las conexiones	0.5-1.0
2. Esfuerzos de trabajo, no mayores que aproximadamente $\frac{1}{2}$ del límite de fluencia	Tuberías vitales	0.5-1.0
	Acero soldado, concreto presforzado, concreto bien reforzado (sólo pequeños agrietamientos)	2
	Concreto reforzado muy agrietado	3-5
	Acero atornillado y/o remachado, estructuras de madera con juntas clavadas o atornilladas	5-7
3. Al límite de fluencia o justamente abajo de él	Tuberías vitales	2
	Acero soldado, concreto presforzado (sin pérdida total de presfuerzo)	5
	Concreto reforzado y concreto presforzado	7-10
	Acero atornillado y/o remachado, estructuras de madera con juntas atornilladas	10-15
	Estructuras de madera con juntas clavadas	15-20
4. Pasando el punto de fluencia, con deformación permanente mayor que la deformación al límite de fluencia	Tubería	5
	Acero soldado	7-10
	Concreto reforzado y concreto presforzado	10-15
	Acero atornillado y/o remachado, y estructuras de madera	20

CABECEO (N. Newmark)

DESCRIPCION DEL SUELO	VELOCIDAD ONDAS DE CORTANTE	PORCENTAJE DE AMORTIGUAMIENTO CRITICO
ROCA	$v_s > 1800$ m/seg	2 - 5
SUELO FIRME	$600 \leq v_s \leq 1800$ m/seg	5 - 7
SUELO COMPR.	$v_s < 600$ m/seg	7 - 10

HORIZONTAL Y TORSION (Preliminar)

DESCRPICION DEL SUELO	PROFUNIDAD DE LA ROCA (m)	PORCENTAJE DE AMORTIGUAMIENTO CRITICO
ROCA	0.0	5 - 7
SUELO FIRME	$\pm$ 5.0 m	7 - 10
SUELO COMPR.	5.0 m	10 - 20
SUELO COMPR.	10.0 m	20 - 40

AMORTIGUAMIENTO DEL SUELO

El amortiguamiento promedio pesado se calcula modo a modo de la siguiente manera:

1. Calcular el vector velocidades

$$v_{ij} = F_j \phi_{ij} \cdot g / \omega_j$$

donde: F_j = factor de participación del modo j

ϕ_{ij} = componente del vector característico en la dirección del grado de libertad i del modo j.

g = aceleración de la gravedad

ω_j = frecuencia natural del modo j.

las unidades correspondientes son:

$$\begin{array}{ccccccccc} v_{ij} & = & F_j & \cdot & \phi_{ij} & \cdot & g & / & \omega_j \\ [m/seg] & = & [1/m] & \cdot & [m] & \cdot & [m/seg^2] & / & [rad/seg] \end{array}$$

2. Calcular la energía total (cinética):

$$E_T]_j = \sum_{i=1}^{n} m_i \cdot v_{ij}^2 / 2.0$$

3. Se calcula la energía absorvida por los resortes de interacción:

$$E_{sh}]_j = K_{sh} \cdot \Delta_{sh}]_j^2 / 2.0 \qquad \text{(horizontal del modo j)}$$

$$E_{sc}]_j = K_{sc} \cdot \phi_{sc}]_j^2 / 2.0 \qquad \text{(cabeceo del modo j)}$$

$$E_{st}]_j = K_{st} \cdot \theta_{st}]_j^2 / 2.0 \qquad \text{(torsión del modo j)}$$

donde: K_{sh}, K_{sc}, K_{st} son las rigideces de los resortes de interacción.

Δsh, ϕsc, θst son los desplazamientos de la masa de la cimentación para el modo j.

4. La energía absorvida por la estructura es:

$$E_e]_j = E_T]_j - E_{sh}]_j - E_{sc}]_j - E_{st}]_j$$

5. Se calcula el amortiguamiento promedio pesado como:

$$\xi_p]_j = \frac{\xi_e \cdot E_e]_j + \xi_{sh} \cdot E_{sh}]_j + \xi_{sc} \cdot E_{sc}]_j + \xi_{st} \cdot E_{st}]_j}{E_T]_j}$$

donde: ξ son amortiguamientos, cuyo subindice identifica estructura, suelo horizontal, suelo cabeceo, etc.

y $\xi_p]_j$ es el amortiguamiento promedio pesado del modo j.

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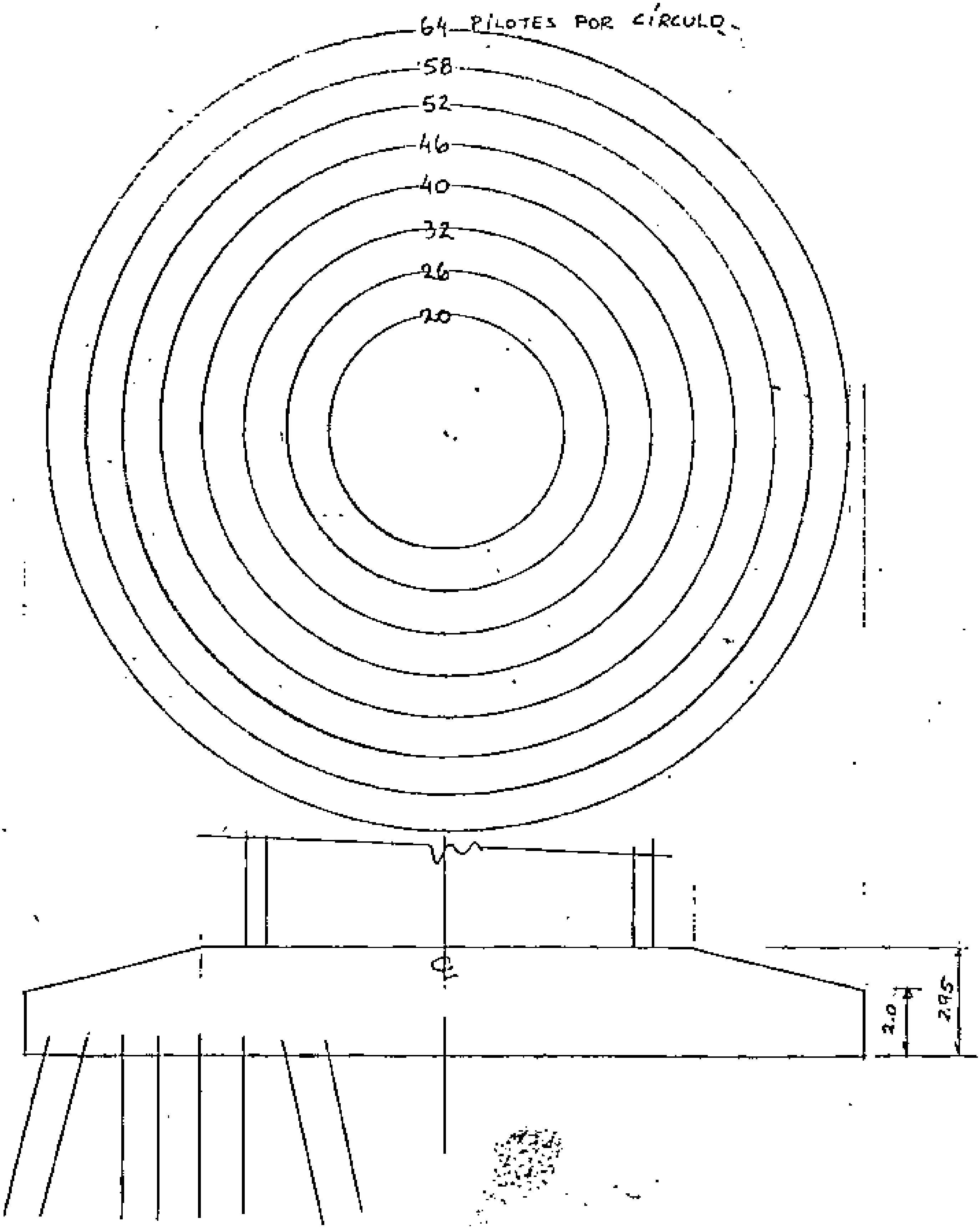

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