Zeitschrift: IABSE congress report = Rapport du congrès AIPC = IVBH Kongressbericht

Band: 8 (1968)

Artikel: New practices in concrete buildings

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DOI: https://doi.org/10.5169/seals-8847

New Practices in Concrete Buildings

Développements nouveaux relatifs aux bâtiments de grande hauteur en béton

Neue Entwicklungen bei Beton-Hochhäusern

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INTRODUCTION

The authors are to be commended for their concise presentation of the practices in concrete high-rise buildings. Their description is applicable where only wind forces act laterally against the buildings. In areas of high seismicity, both the design and construction is markedly different.

Professors N. M. Newmark and W. J. Hall are presenting, at this Congress, an extensive paper on the Dynamic Behavior of Reinforced Concrete Buildings. It is the philosophy expressed in their paper which led the Structural Engineers Association of California, (SEAOC) to initiate new procedures in the design of high-rise buildings.

Damaging earthquakes seem to indicate that, in order to safeguard life and property, the building frames have to resist lateral forces brought about by the earthquakes, and, furthermore, such building frames to be able to absorb energies without failures. The implementation of these criteria, however, is not a simple matter. Attempts to design within the elastic range would create buildings beyond economic feasibilities.

Forty years of evolution in producing a realistic design procedure has resulted in the latest SEAOC requirements which, for high-rise structures particularly, demand a ductile, energy absorbing rigid frame, with the following behavior:

a. Minor earthquake: No damage to the building.

b. Moderate earthquake: No structural damage, although minor damage to enclosing materials may be expected.

c. Major earthquake: Considerable non-structural damage, but the reinforced concrete frame absorbs the excess energy through ductile yielding and formation of elasto-plastic hinges.

EARTHQUAKE INTENSITIES. While no location may be guaranteed against earthquakes, past histories have been used for the production of a map of seismic probability.

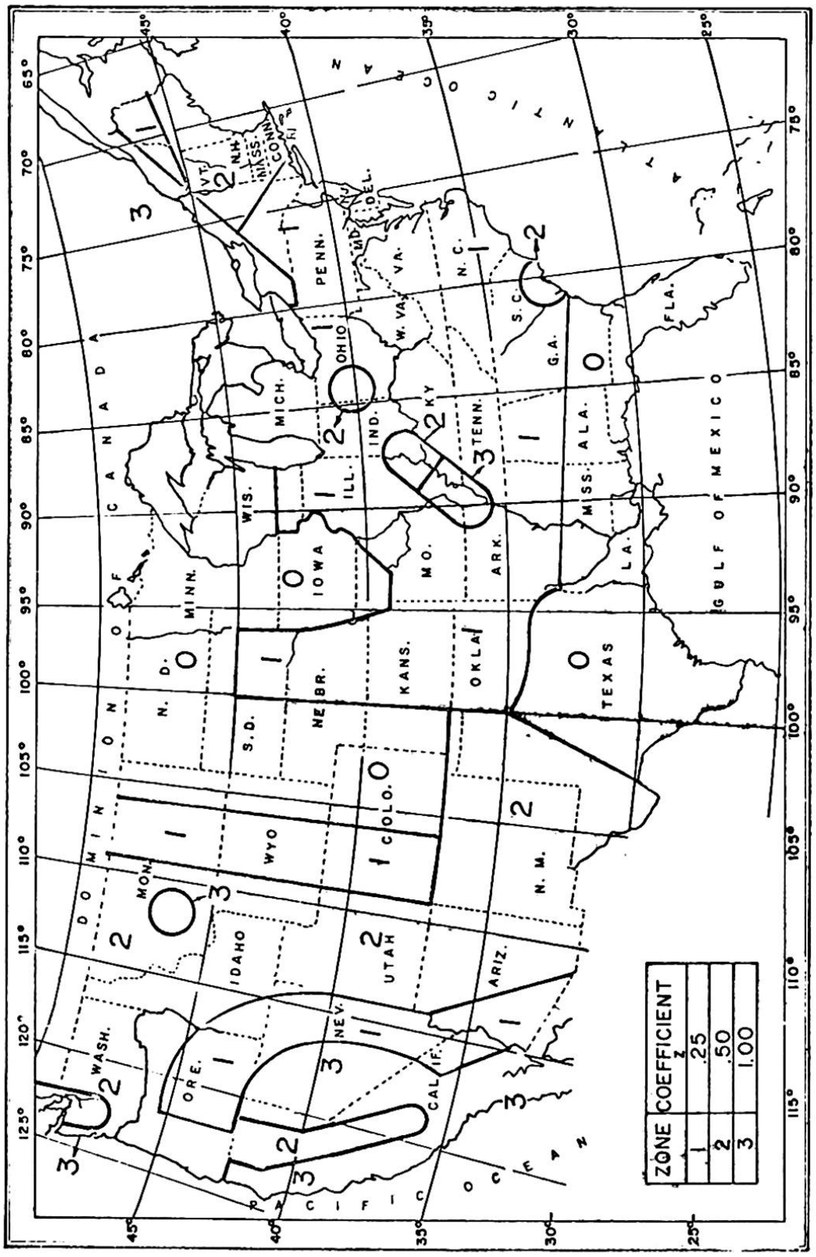


FIG. 1.
EARTHQUAKE ZONES AND COEFFICIENTS "Z"

This map is now in process of revision (e.g.: Substantial earthquakes were registered in Denver, Colorado, a zero intensity location on the map.) In the calculation of seismic lateral forces a coefficient "Z" to be employed. The value "Z" varies according to the probability zone, such as:

Zone 0: "Z"= 0; Zone 1: "Z"= 0.25; Zone 2: "Z"= 0.50;
Zone 3: "Z"= 1.00.

FORCE FACTOR "K". Dependent on the framing system employed, a coefficient "K" is to be used:

HORIZONTAL FORCE FACTOR "K" FOR BUILDINGS OR OTHER STRUCTURES(1)

TYPE OR ARRANGEMENTS OF RESISTING ELEMENTS	VALUE OF K(2)
All building framing systems except as hereinafter classified.	1.00
Buildings with a box system as defined in Section 2313(b).	1.33
Buildings with a dual bracing system consisting of a ductile moment resisting space frame and shear walls designed in accordance with the following criteria: 1. The frames and shear walls shall resist the total lateral force in accordance with their relative rigidities considering the interaction of the shear walls and frames. 2. The shear walls acting independently of the ductile moment resisting space frame shall resist the total required lateral force. 3. The ductile moment resisting space frame shall have the capacity to resist not less than 25 per cent of the required lateral force.	0.80
Buildings with a ductile moment resisting space frame designed in accordance with the following criteria: The ductile moment resisting space frame shall have the capacity to resist the total required lateral force.	0.67
Elevated tanks plus full contents, on four or more cross-braced legs and not supported by a building.(3), (4), (5).	3.00
Structures other than buildings and other than those set forth in Table 23-D.	2.00

(1) Where prescribed wind loads produce higher stresses, these loads shall be used in lieu of the loads resulting from earthquake forces.

(2) The coefficients determined here are for use in the State of California and in other areas of similar earthquake activity. For areas of different activity, the coefficient may be modified by the building official upon advice of seismologists and structural engineers specializing in aseismic design.

(3) The minimum value of KC shall be 0.12 and the maximum value of KC need not exceed 0.25.

(4) For overturning, the factor J as set forth in Section 2313(h) shall be 1.00.

(5) The torsional requirements of Section 2313(g) shall apply.

High-rise buildings in excess of 160 ft. ($\pm$ 49 m.) can only be of ductile system, with a "K" factor of 0.80 or 0.67. There is a penalty for use of shear walls, as stated above; however, shear walls may be necessary to restrict drift either against seismic or wind forces; or they are difficult to eliminate due to architectural layouts. (Enclosure walls around stairs and elevators. Solid cast-in-place stairs act as trusses similar to shear walls.)

The minimum seismic base shear, in the direction of each of the main axes is to be:

$V = ZKCW$, where

W= total dead load, (KIPS) $\quad W = \sum_{i=1}^{n} w_i$

$C = \dfrac{0.05}{\sqrt[3]{T}}$; ($C$= 0.10 for one and two-story buildings.)

$T = \dfrac{0.05 h_n}{\sqrt{D}}$; (T= 0.10 x number of stories above the base for rigid frame high-rises.)

= fundamental period of vibration in seconds in the direction under consideration. Properly substantiated data is also acceptable. (Such as computer calculations.)

h_n= total height of building above base. (ft.)

D= dimension of building in the direction of applied forces. (ft.)

In order to account for the higher modes of vibration and for whipping forces, the base shear "V" is to be distributed as follows:

F top level= $0.004V \left(\dfrac{h_n}{D_s}\right)^2 \leq 015V$, $\quad$ ($F_t = 0$ if $\left(\dfrac{h_n}{D_s}\right) \leq 3$)

F_x at level x= $\dfrac{(V-F_t)\, w_x h_x}{\sum_{i=1}^{n} w_i h_i}$, where

D_s= plan dimension of the vertical lateral force resisting system(ft)

wi, wx= that portion of "W" which is located at or is assigned to level "i" or "x" respectively.

TORSIONAL MOMENTS. As a rule, high-rise structures should be designed for a minimum torsional eccentricity of 5% of maximum building dimension.

OVERTURNING. Every high-rise building to resist the overturning effect caused either by wind or earthquake. For the latter there is a modified moment:

$M_{o.t.} = J\,(F_t h_n + \sum_{i=1}^{n} F_i h_i)$ $\quad$ where

$J = \dfrac{0.6}{\sqrt[3]{T}} \leq 1.$

SPECIAL PROVISIONS. Ductile frame buildings of 160 ft. or higher are subject to several restrictive provisions. The most important ones are enumerated as follows:

1. The main ductile moment resisting frame has to be cast-in-place monolithic reinforced concrete. Other members may be precast, prestressed, composite, etc.

2. The Ultimate Strength Design Method (USD) is specified for ductile frames.

3. The following load factors are to be used:

$U = 1.40\ (D + L + E)$ where D= Dead Load, L= Live Load, E= Earthquake Force

$U = .90\ D \pm 1.25E$

$= \text{(suggested)}\ .90\ D \pm 1.35E$

4. Under no conditions should plastic hinges be formed in columns but only in beams preferably near the columns.

5. In order to insure ductility both columns and beams at end near the joints shall be of confined concrete. Concrete may be confined by closely spaced ties, or stirrup-ties.

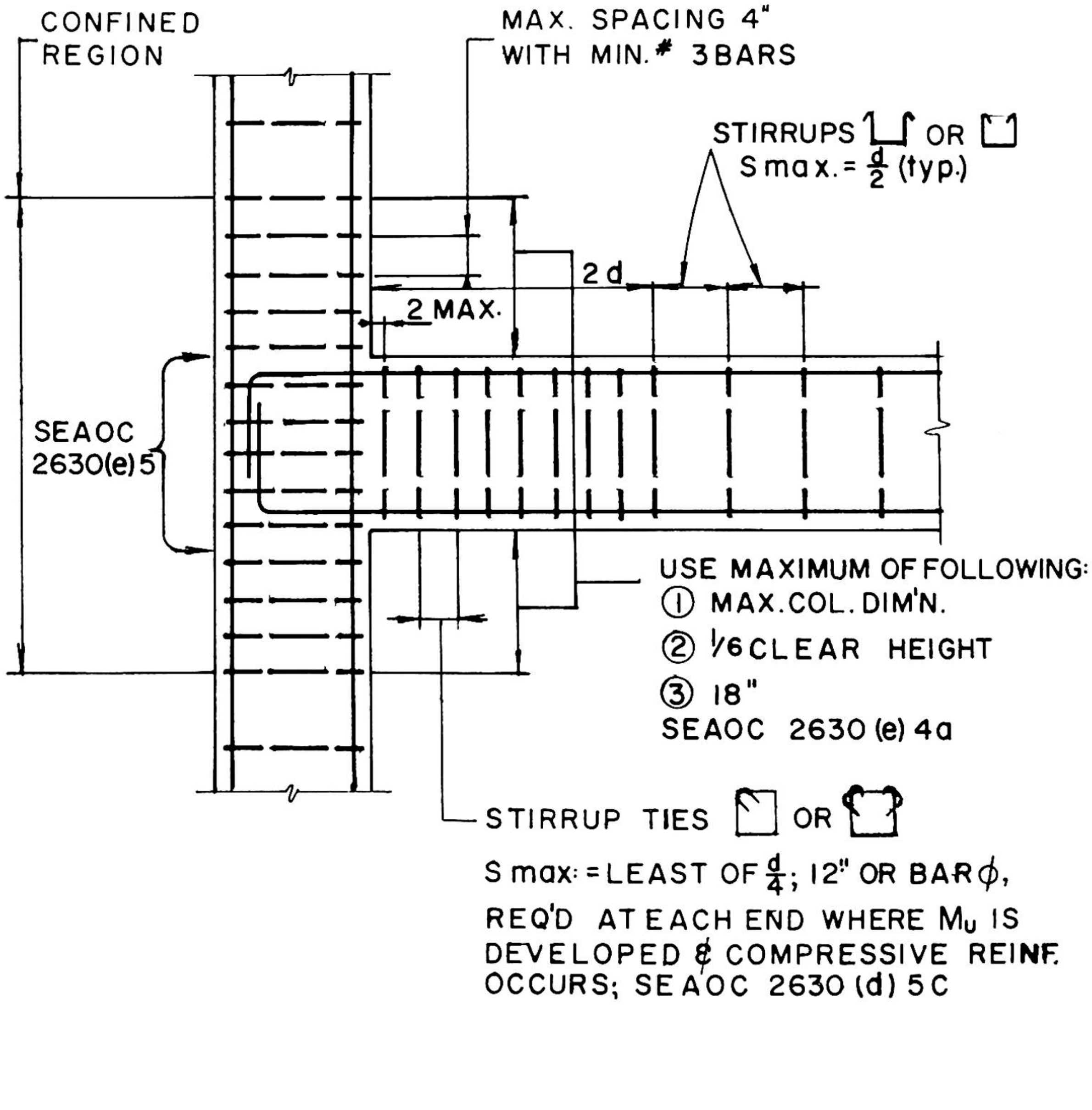
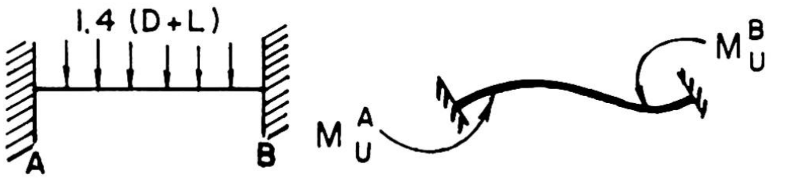


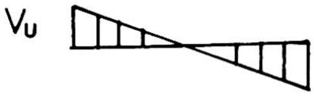
FIG. 2

COLUMN - BEAM JOINT REINFORCEMENT

WEB REINFORCEMENT MIN. # 3 BARS IN ACCORDANCE WITH ACI, CHAPTER 17 --- SEAOC 2630 (d) 5

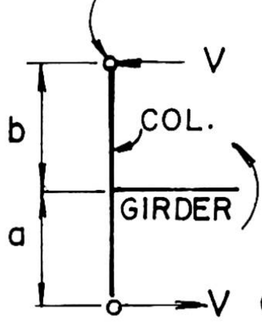


M_U^A & M_U^B = ULTIMATE MOMENT CAPACITY OF MEMBER OF OPPOSITE SENSE.



$$V_U \geqq \frac{M_U^A + M_U^B}{L} + 1.4\,V(D+L) \text{ --SEAOC 2630(d)5a(30-5)}$$

MID HEIGHT OF STORY AS DETERMINED BY RATIONAL ANALYSIS.



$M_U = V\,(a+b)$

OR $V = \dfrac{M_U}{a+b}$

$V_U = T - V$ ---------------------------- SEAOC 2630 (C) 5

DESIGN IN ACCORDANCE WITH ACI 318 CHAPTER 17, SPECIAL TRANSVERSE REINFORCEMENT REQ'T MAY GOVERN... SEAOC 2630 (C) 4

FOR COLUMNS WITH GIRDER FRAMING ON ALL FOUR SIDES ONE HALF OF THE SPECIAL TRANSVERSE REINFORCEMENT IS REQUIRED.

FIG. 3

GIRDER-COLUMN JOINT ANALYSIS

$$V_u = \frac{M_u^T + M_u^B}{h}$$ — — — — — — — — — — — — SEAOC 2630 C (6) 30 - 9

$$OR = \frac{M_u^B + \frac{1}{2} M_b}{h}$$ — — — — — — — — — — — — SEAOC 2630 C (6) 30 - 8

(OMIT FACTOR $\frac{1}{2}$ FOR ONLY ONE COLUMN IN TOP CONNECTION.

$V_u = V_c = A_v f_y \ \frac{d}{s}$ — — — — — — — — — — — — SEAOC 2630 (e) 6 30 - 7

WHERE $V_c = v_c bd$ — — — — — — — — — — — — ACI 318 CHAPTER 17

EXCEPT FOR $\frac{P}{A_g} \leq 0.12 f'_c$, $V_c = 0$.

A_v = TOTAL AREA OF TRANSVERSE REINF. IN TENSION WITHIN DISTANCE BETWEEN TIES

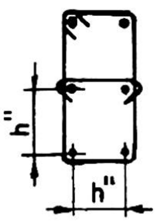
FOR SPIRALS USE $2/3\ A_v$ — — — — — — SEAOC 2630 (e) 6

FOR SPIRALS:

MIN. VOLUME SPIRAL $P_s = 0.45 \left(\frac{A_g}{A_c} - 1\right) \frac{f'_c}{f_y}$ - ACI 913 (b) (9-1)
BUT NOT LESS THAN REQUIRED BY — — SEAOC 2630 e (6)

FOR HOOPS:

MIN.: 2 P_s



SUPPLEMENTARY CROSS TIES; NOT TO EXCEED 25% OF TOTAL TIE VOLUME & SHALL HAVE STANDARD HOOKS AND ENGAGE EXTERIOR HOOP & VERTICAL BAR.

$$h'' \leq \frac{2A''_{sh} \ f''_y h}{p'' a f''_y}$$ — — — — — — — — — — SEAOC 2630 (e) 6 30 - 6

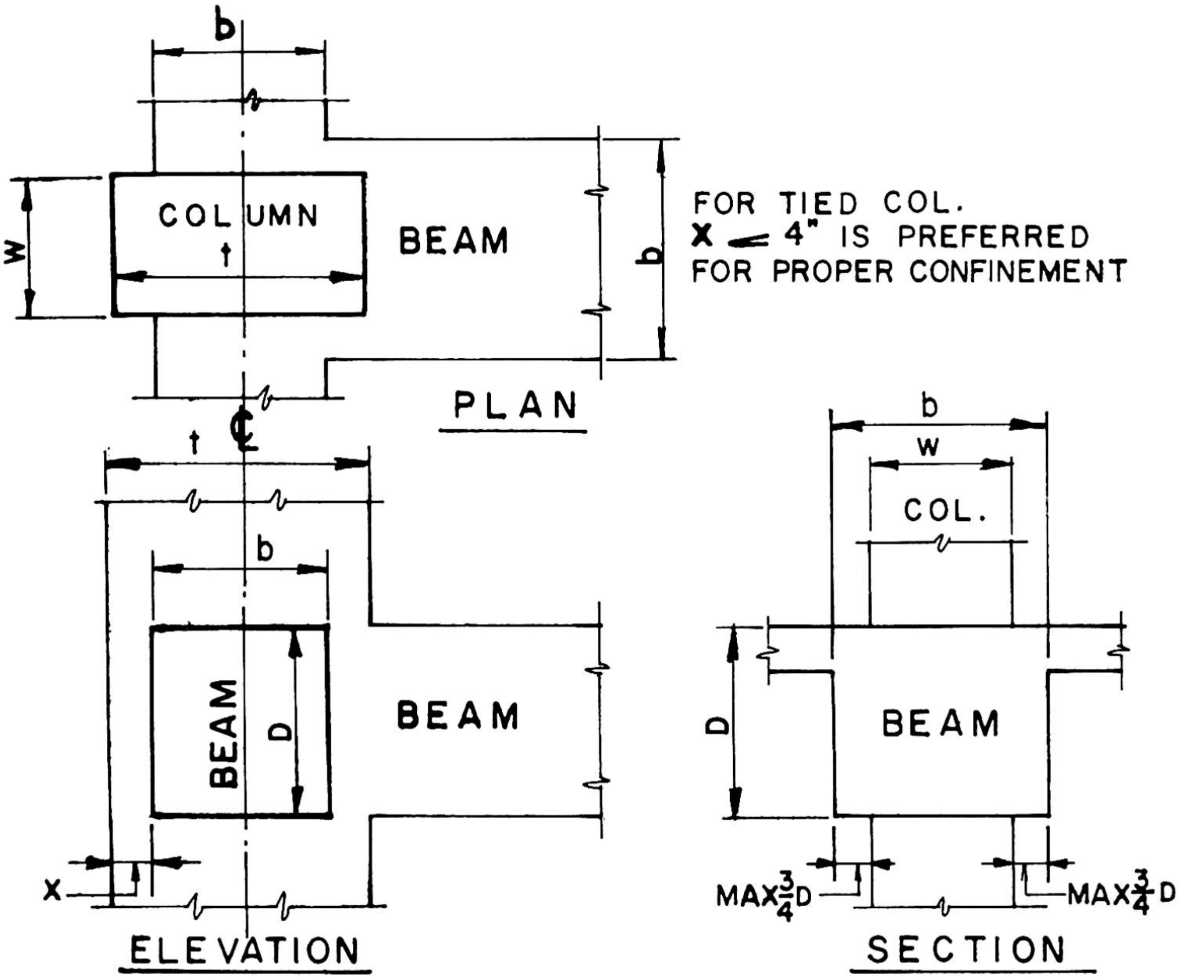
IF $h'' \leq$ MAX. COLUMN DIMENSION, PROVIDE NECESSARY OVERLAPPING HOOPS — — SEAOC 2630 (e) 4 C

FIG. 4

ANALYSIS FOR COLUMN TRANSVERSE REINFORCEMENT

6. Only intermediate grade reinforcing steel (A-15, 40,000 psi yield point) shall be used in flexural members. (This restriction will be lifted as soon as the ductility requirements of high strength steel will be guaranteed.) Higher strength steel (A-432, 60,000 psi yield point) may be used in columns.

7. Compact sections are prescribed both for columns and beams.



BEAMS: $\frac{b}{D} \geq 0.4$

OR 10" $\leq$ b $\leq$ W + MAX.($\frac{3}{4}$D) EACH SIDE OF COLUMN – – SEAOC 2630(d)1

b $\geq \frac{3}{4}$t OR $\frac{3}{4}$ W (RECOMMENDED) – – – – – – – – – – – – – SEAOC 2630(e)40

COLUMN: $\frac{W}{t} \geq 0.4$

W OR t OR ϕ OF ROUND COL. $\geq$ 12" – – – – – – – – – – – – SEAOC 2630(e)1

DESIGN: FOR $\frac{P}{Ag} \geq 0.12 f'_c$ WHERE Ag = (W) (t)

M_u OF COL. WITH P $\geq$ M_u OF BEAMS. FOR $\frac{P}{Ag} < 0.12 f'_c$ COLUMN SHALL CONFORM AS FLEXURAL MEMBER – – – – – SEAOC 2630(C)7

FIG. 5

SEAOC CODE LIMITATIONS ON FRAME DIMENSIONS.

8. The requirements under (7.) eliminate, until further studies are conducted, the use of flat slabs and flat plates as systems of unproven ductility.

9. In order to avoid brittle failures, not only minimum but also maximum reinforcements are prescribed.

10. Extreme attention is to be given to splices and anchorages.

11. Shear walls must be provided with strong boundary members designed to carry all vertical loads attributed to the shear walls including overturning axial forces. Thus, in case of failure of a shear wall during an earthquake, the boundary members would take over and insure safety against collapse.

CONCLUSIONS

The recently adopted Code provisions of the SEAOC permit now the construction of reinforced concrete high-rise buildings. (Previously only buildings less than 160 ft. in height could be built of reinforced concrete.) The writer has designed a 21-story medical building and a 26-story office building using the ductile frame reinforced concrete principles as described in this discussion.

SUMMARY

Reinforced concrete high-rise buildings in seismic areas have to be designed and constructed differently from the customary types described in the author's presentation. This discussion attempts to describe the present state of art for earthquake resisting high-rise buildings in the Western States of the United States.

RÉSUMÉ

Les bâtiments élevés en béton armé situés dans des zones sismiques doivent être projetés et construits différemment des types habituels décrits par l'auteur. La discussion essaye de faire le point sur les méthodes actuelles employées dans l'Ouest des Etats-Unis pour des bâtiments élevés résistants aux secousses sismiques.

ZUSAMMENFASSUNG

Hohe Stahlbetongebäude in Erdbebengebieten müssen anders als die üblichen, in des Verfassers Darstellung beschriebenen Typen entworfen und durchgeführt werden. Dieser Beitrag versucht, in den derzeitigen Stand der Bauweise erdbebensicherer, hoher Gebäude in den Weststaaten der USA Einblick zu geben.

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