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A study on the seismic requirements of steel building frames designed for wind

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STEEL BUILDING FRAMES DESIGNED FOR WIND

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

ABSTRACT

Several national and regional codes now require the consideration of earthquake ground excitations in the design of buildings in many parts in the United States, even in the eastern U.S., while a number of wind governed buildings exist in the U.S., especially in the eastern U.S.. These buildings would generally have an overstrength when compared with the seismic design loads and a greater ultimate strength than that implied in the seismic design codes. This suggests that it may not be necessary to provide the same structural details in order achieve the capacity or ductility for nonlinear deformations. Structural steel may become more competitive against reinforced concrete in building construction if the connections can achieve the desired ductility without additional fabrication and erection costs.

In this thesis, the load criteria which control structural design are first reviewed. Several codes in the U.S. are then compared and a discussion of the dominant criteria for various types of buildings is presented. Finally, ductility demands on wind governed buildings are examined.

1. INTRODUCTION

Several national and regional building codes now require the consideration of earthquake ground excitations in the design of buildings in many parts of the U.S., even in the eastern U.S.. Recent design experience has indicated that for low to medium rise buildings constructed in the eastern U.S. the governing lateral loading for design is sometimes the earthquake loading. Further, in some cases, although wind loading governs the proportioning of the structural members, the connection details have to meet certain ductility requirements because of the concern of energy absorption capacity.

These seismic codes adopt a static analysis and an equivalent lateral force procedure using force reduction factor (R) and ductility factor (C_d) without requiring any true nonlinear analysis. However, the R and C_d factors suitable for wind governed buildings are not specified in these codes.

In this thesis, the load criteria which control structural design will first be reviewed. Several codes in the U.S. will then be compared and a discussion of the dominant criteria for various heights and floor configurations will be presented. Finally, ductility requirements of building frames designed for wind will be examined.

2. OVERVIEW OF EXISTING CODES IN THE UNITED STATES

There are four major codes in use in the U.S. today:

1) "*Uniform Building Code*, 1988 *edition* (*UBC code*)", International Conference of Building Officials, Whittier, California.

2) "*The BOCA National Building Code*, 1987 *edition* (*BOCA code*)", Building Officials & Code Administrators International, Inc., Homewood, Illinois.

3) "*The Standard Building Code*", The Southern Building Code Congress, Birmingham, Alabama.

4) "*The National Building Code*", The American Insurance Association, New York.

Their use is somewhat regional: the UBC code is used most extensively in the West, BOCA in the Midwest, Standard in the South, and National in the Northeast. However, these codes do not have jurisdictional boundaries[2.1].

In addition to these codes, there is a set of proposed seismic regulations developed by the Applied Technology Council (ATC), a research and developement organization affiliated with the Structural Engineers Association of California (SEAOC):

5) ATC-3-06; "*Tentative Provisions for the Developement of Seismic Regulations for Buildings*", 1978.

Recently, the following provisions and acompanying commentary based on the above document were issued:

6) National Earthquake Hazards Reduction Program (NEHRP); "*Recommended Provisionss for the Development of Seismic Regulations for New Buildings*, 1985 *edition*", by Building Seismic Safety Council, 1985.

In this section, the code comparison will be done among the UBC code, the BOCA code, and the NEHRP provisions only, because the BOCA code, the Standard Building Code, and the National Building Code have similar approaches to determine seismic design loads and can be considered together.

2.1 Comparison of Seismic Design Loads

It is difficult to make a general comparison of these codes, so the following specific case is chosen. Consider a multi-story office building using a moment-resisting steel frame system. The building is to be located in a major city in California (CA), where the seismic requirement is the most severe, and in New York City (NY), a major city in the eastern U.S., where the wind loading is often the controlling loading.

2.1.1 Seismic Base Shear

The seismic base shear can be determined by the different formulas presented in the codes. Throughout this discussion, the following common symbols, as defined below, will be used:

- W: total gravity load of the building,
- S: coefficient related to the soil profile characteristics. The value of 1.2 is used in this study for deep stiff soil over rock.
- T: fundamental period of the building in second. $T = 0.035(h_n)^{3/4}$ for moment-resisting steel frames, where h_n is the overall building height in feet.

There are two types of moment-resisting frames: Special Moment Resisting Steel Frames (SMRSF) and Ordinary Moment Resisting Steel Frames (OMRSF). SMRSF's are the moment-resisting frames specially detailed to provide ductile behavior or to have the capability of significant nonlinear deformation during earthquakes. OMRSF's are the ones not meeting special detailing requirements and assumed to have a limited amount of nonlinear deformation capacity. SMRSF and OMRSF will be discussed in detail later because they are defined differently in each code.

(1) UBC code, 1988 edition

The seismic base shear, V, is given by the following formula:

$$V = (ZIC/R_w)\cdot W \qquad (2.1)$$

where Z: seismic zone factor

$Z = 0.40$ for CA (seismic zone 4)

$Z = 0.15$ for NY (seismic zone 2A)

I: importance factor

$I = 1.0$ for normal office buildings

R_w: numerical coefficients according to the building type

$R_w = 12$ for SMRSF

$R_w = 6$ for OMRSF

C: seismic design coefficient given by,

$$C = \frac{1.25S}{T^{2/3}} \quad (\leq 2.75) \qquad (2.2)$$

(2) BOCA code, 1987 edition

The seismic base shear, V, is given by,

$$V = (ZIKCS)\cdot W \qquad (2.3)$$

where Z: seismic zone factor

$Z = 1$ for CA (seismic zone 4)

$Z = 3/8$ for NY (seismic zone 2)

I: importance factor

$I = 1.0$ for normal office buildings

K: horizontal force factor

$K = 0.67$ for SMRSF

$K = 1.0$ for OMRSF

C: seismic design coefficient given by,

$$C = \frac{1}{15T^{1/2}} \quad (C \leq 0.12,\ CS \leq 0.14) \qquad (2.4)$$

(3) NEHRP provisions, 1985 edition

The seismic base shear, V, is given by,

$$V = C_s W \tag{2.5}$$

where C_s: seismic design coefficient given by,

$$C_s = \frac{1.2 A_v S}{R \cdot T^{2/3}} \quad (\leq \frac{2.5 A_a}{R}) \tag{2.6}$$

A_v: seismic coefficient representing the Effective Peak Velocity-Related Acceleration

A_v = 0.4 for CA (map area 7)

A_v = 0.1 for NY (map area 3)

A_a: seismic coefficient representing the Effective Peak Acceleration

A_a = 0.4 for CA (map area 7)

A_a = 0.1 for NY (map area 3)

R: response modification factor

R = 8 for SMRSF

R = 4.5 for OMRSF

Substituting the above specific values into Formulas (2.1) to (2.6) gives the expressions shown in Table 2-1 for OMRSF's and Table 2-2 for SMRSF's in CA and NY, respectively. The UBC and BOCA codes specify the working values of the seismic base shear, whereas the NEHRP provisions give the base shear for capacity design. Therefore, the code requirements for base shear capacity will be reduced to a working load level for comparison.

i) Ordinary Moment Resisting Steel Frames (OMRSF)

The UBC and BOCA codes state that OMRSF's shall confirm to the requirements of AISC Specification Part I (Allowable-Stress Design procedure)[2.2]. This specification permits a 1/3 increase of the allowable stresses in the presence of wind or seismic loading, either acting alone or in combination with the dead and live loads. Then, the base shear capacity requirement at the working load level is,

$$V_W = (3/4)\cdot V = 0.75V \tag{2.7}$$

According to the NEHRP provisions, the OMRSF's are to be designed and constructed in accordance with AISC Specification Part I (Allowable-Stress Design procedure) as modified by a modifier of 1.7 on the working stresses and a capacity reduction factor of Φ=0.9. Then the base shear capacity requirement at the working load level is,

$$V_W = \frac{V}{0.9 \times 1.7} = 0.654V \tag{2.8}$$

The working base shear coefficient for OMRSF's shown in Table 2-1 were obtained by simply dividing the seismic base shear, V, by the total gravity load, W, and then multiplying this result by 0.75 for the UBC and BOCA codes and 0.654 for the NEHRP provisions, respectively. Fig. 2-1 shows a comparison of the design base shear coefficients at the working load level for OMRSF's among the UBC code, the BOCA code, and the NEHRP provisions.

For OMRSF's located in CA, the NEHRP requirement is the greatest, UBC is the second, and BOCA is the smallest. But in NY, the UBC requirement exceeds those of the NEHRP and BOCA. Also, for buildings with periods of 0.7 second or more, the NEHRP requirement is the smallest.

ii) Special Moment Resisting Steel Frames (SMRSF)

The UBC code defines a SMRSF as a moment-resisting frame specially detailed to provide ductile behavior satisfying the code requirements. The BOCA code requires the SMRSF's to be designed to satisfy the requirements of AISC Specification Part II (Plastic Design procedure)[2.2] Sections 2.7, 2.8 and 2.9, which are related to the width-thickness ratio, connections, and lateral bracing, respectively. These two codes contain a seismic overload factor of 4/3 by allowing 1/3 overstress in the presence of earthquake forces. Thus, the base shear capacity requirement at the working load level is,

$$V_w = (3/4)\cdot V = 0.75V \tag{2.9}$$

The NEHRP provisions calls for steel member strength to be evaluated by the plastic design procedures, using 1.7 times the bending stress allowed for in the conventional elastic design. It also imposes a capacity reduction factor of Φ=0.90 for steel members and connections that develop the strength of the members. Combining these and taking the ratio of plastic moment to yield moment to be M_p/M_y= 1.14 for rolled steel beams, the base shear capacity requirement, reduced to the working load level, becomes:

$$V_w = \frac{V}{0.9\times1.7\times1.14} = 0.573V \tag{2.10}$$

The tabulated values given in Table 2-2 were obtained by multiplying the base shear coefficients, C_s, calculated using Formulas (2.1) to (2.6) by 0.75 for the UBC and BOCA codes, and 0.573 for the NEHRP provisions, respectively.

Fig. 2-2 shows the comparison of capacity requirements of SMRSF's in CA and NY. For SMRSF's in CA there are only slight differences among the three, while for the SMRSF's in NY the NEHRP requirement is the smallest. Their absolute values are almost half of those of the OMRSF's shown in Fig. 2-1.

2.1.2 Vertical Distribution of Seismic Forces

(1) UBC code, 1988 edition

(2) BOCA code, 1987 edition

The UBC and BOCA codes have a similar approach to determine the vertical distribution of seismic forces. Therefore, they are to be considered together. The total lateral force, V, shall be distributed over the height of the building in accordance with the following formula:

$$V = F_t + \sum_{i=1}^{n} F_i \tag{2.11}$$

where F_t: the portion of V concentrated at the top of the structure, but it is considered to be zero when $T \leq 0.7$sec.

$$F_t = 0.07TV \qquad (\leq 0.25V) \tag{2.12}$$

The lateral force, F_x, at level x is given by,

$$F_x = (V - F_t) \cdot w_x h_x \;/\; \sum_{i=1}^{n} w_i h_i \tag{2.13}$$

where w_i or w_x is the portion of W located at level i or x and h_i or h_x, the height above the base to level i or x.

(3) NEHRP provisions, 1985

The vertical distribution of seismic forces is given by,

$$F_x = C_{vx} V \tag{2.14}$$

$$C_{vx} = w_x \cdot (h_x)^k / \sum_{i=1}^{n} w_i \cdot (h_i)^k$$

where w_i or w_x is the portion of W located at level i or x, h_i or h_x, the height above the base to level i or x, and k, a power coefficient given by,

$k = 1$	for $T \leq 0.5$ sec.
$k = 0.75 + 0.5T$	for $0.5 < T < 2.5$ sec.
$k = 2$	for $T \geq 2.5$ sec.

Under the assumption that each story has a constant gravity load, w, and a constant story height, h, Formula (2.11) can be rewritten for an n-story building as

$$F_n/V = 0.07T + (1-0.07T)\cdot \frac{2}{n+1} \quad (2.15.a)$$

$$F_n/V = (1-0.07T)\cdot \frac{2x}{n(n+1)} \qquad (x=1,2,\cdots,n\text{-}1) \quad (2.15.b)$$

Also, Formula (2.14) can be rewritten as

$$F_x/V = x^k / \sum_{i=1}^{n} i^k \qquad (x=1,2,\cdots,n) \quad (2.16)$$

Similar expressions can also be written using the base shear coefficient, C_s, as the normalizing parameter.

For the UBC and BOCA codes,

$$F_n/w = [0.07T + (1-0.07T)\cdot \frac{2}{n+1}]\cdot nC_s \quad (2.17.a)$$

$$F_x/w = [(1-0.07T)\cdot \frac{2x}{n(n+1)}]\cdot nC_s \qquad (x=1,2,\cdots,n\text{-}1) \quad (2.17.b)$$

For the NEHRP provisions,

$$F_x/w = \left(x^k / \sum_{i=1}^{n} i^k\right)\cdot C_s \qquad (x=1,2,\cdots,n) \quad (2.18)$$

The vertical distribution seismic coefficient at the working load level is defined as $k_{xw} = F_{xw}/w$ where F_{xw} is the lateral force at the working load level. Figs. 2-3(a) and (b) show the k_{xw} values for 3-, 9-, and 15-story OMRSF's and SMRSF's located in CA designed by the UBC, BOCA and NEHRP provisions. The distribution over the height of the 3-story frame is almost same for all the codes, while the distributions of the 9- and 15-story frames show a large difference between the UBC, BOCA codes and the NEHRP provisions. This difference occurs because the concentrated load at the top level in the UBC and BOCA codes exists in the 9- and 15-story frames, and also because the NEHRP provisions have an exponential distribution over the height of buildings, whereas the UBC and BOCA codes have a triangular distribution.

2.2 Comparison of Wind Design Loads

Since the NEHRP provisions are not regulations for wind loads, the code comparison will be done only between the UBC code and the BOCA code.

(1) UBC code, 1988 edition

The design wind pressure for structures is determined for any height in accordance with the following formula:

$$P = C_e \cdot C_q \cdot q_s \cdot I \tag{2.19}$$

where C_e: combined height, exposure, and gust factor coefficient as shown in Fig.2-4.

C_q: pressure coefficient, for flat roof structures,

$C_q = 1.3$ for 40ft. or less in height

$C_q = 1.4$ for over 40ft. in height

q_s: wind stagnation pressure at standard height of 30ft.

$q_s = 13$ psf for CA (basic wind speed 70mph)

$q_s = 21$ psf for NY (basic wind speed 90mph)

I: importance factor,

$I = 1.0$ for normal office buildings

(2) BOCA code, 1987 edition

The design wind pressure is determined as follows:

$$P = P_e \cdot I^2 \cdot C_p \tag{2.20}$$

where P_e: effective velocity pressure which is the same as the product of $C_e \cdot q_s$ in the UBC code as shown in Fig.2-4.

I: importance factor, equal to 1.0 for normal office buildings.

C_p: external pressure coefficient with consideration of the building plan configuration as indicated in Fig. 2-5.

For buildings with length-to-width ratio, or L/B, equal to or less than 1.0, the design wind forces specified by the BOCA code have the same value as those by the UBC code except for buildings over 40ft. in height whose pressure coefficient is C_p=1.4. This difference is very small and negligible, so the BOCA code will be used for wind design loads in this study.

3. CRITERIA THAT CONTROL BUILDING DESIGN

In this chapter, the horizontal loads that govern building design will be first investigated for buildings with different configurations, heights, and gravity loads. This will be followed by discussions of the member design moments and axial forces due to wind loads, seismic loads, and combined dead and live loads.

3.1 Analysis Models

(1) Building Configuration

Building configurations and dimensions used in this analysis are shown in Figs. 3-1(a), (b), and (c). They are considered to be typical low to medium rise office buildings with moment-resisting frames. All buildings have a constant story height, h=12 ft., and a constant span length, l=24 ft.. The variables included in this study are as follows:

F (number of story) = 3, 6, 9, 12, 15
L/B (length-to-width ratio) = 0.4, 1.0, 2.5

Note that the buildings with L/B ratio of 0.4, 1.0, and 2.5 have 2, 3, and 5 bays, respectively.

(2) Gravity Loads

The dead load is determined with consideration of design experience and the live load is selected according to the UBC and BOCA codes. They are as follows:

Dead load = 50, 75, 100 psf
Live load = 50 psf

The live load is excluded when estimating the total gravity load, W, for seismic force calculations.

(3) Cases Analyzed

The buildings are assumed to be located in California (CA) and New York (NY) and also designed as Ordinary Moment Resisting Steel Frames (OMRSF) and Special Moment Resisting Steel Frames (SMRSF). The applicable codes for design are the UBC code, the BOCA code and the NEHRP provisions for seismic loads and the BOCA code only for wind loads, because the difference of the wind design loads between the UBC and BOCA is very small and negligible, as mentioned in Section 2.2. Therefore, the total number of cases studied is 300:

3(code)x2(location)x2(building system)x5(story)x3(dead load)=180 for L/B=1.0
3(code)x2(location)x2(building system)x5(story)x1(dead load, kept at 75 psf)=60 for L/B=0.4, 2.5

3.2 Horizontal Loads Governing Building Design

The horizontal loads that govern the building design will be examined for buildings with various configurations, heights, and gravity loads. The analysis will be done using the models and parameters described in Section 3.1. The code requirement will be reduced to the working load level and story shear will be used for the purpose of comparison. The story shear is defined as follows:

$$Q_j = \sum_{i=j}^{n} F_i \qquad (j=1,2,\cdots,n) \tag{3.1}$$

where Q_j is the story shear at level j and F_i is the lateral force at level i. All the results obtained from this analysis are summarized in Figs. 3-2 to 3-8.

Fig. 3-2 shows that in CA the seismic design load is double to triple or more of the wind design load for the OMRSF's. However, for the 15-story SMRSF buildings, the seismic load and the wind load are fairly close. This difference occurs because not only the seismic design loads for the OMRSF's are almost double of those for the SMRSF's, but also the wind load increases with the building height, while the seismic load does not increase as much as the wind load. Fig. 3-2 also shows that the story shear distribution due to the seismic forces is different from that due to the wind, because the seismic design forces have a triangular distribution along the height of the building while the wind design force is almost constant over the height as explained in Sections 2.1 and 2.2.

Fig. 3-3 shows that in NY the wind load governs all the designs except for the 3-story SMRSF, because the wind design force in NY is 1.6 times that in CA and the seismic design force in NY is 1/3 to 1/4 of that in CA.

The information presented in Fig. 3-2 and Fig. 3-3 are rearranged in Fig. 3-4 in order to show, at a glance, which load, wind or seismic, dominates the design. In this comparison the seismic loads are determined by the NEHRP provisions. The effect of gravity load is presented in Figs. 3-4, 3-5, and 3-6. In general, the greater the gravity load, the more important is the seismic load, because the wind load is constant and independent of the gravity loads. For a gravity load of 100 psf, the seismic load governs considerable parts of low to medium rise buildings using the OMRSF's even in NY (Fig. 3-6).

The effect of building configuration is shown in Figs. 3-4, 3-7, and 3-8. The seismic load becomes more governing, as the ratio of L/B, or the number of bays increases, because the seismic load increases in proportion to the number of bays, while the area subjected to wind pressure is always constant. In the case of the 5-bay OMRSF in NY, the seismic load governs all the stories for the 3- and 6-story low rise buildings and several stories for the 9-, 12-, and 15-story medium rise buildings.

3.3 Governing Bending Moment for Member Design

The member design moment due to the horizontal load and the combined dead load and live loads is now discussed under the same conditions as in Section 3.2. The seismic design loads are determined in accordance with the NEHRP provisions only, because the differences among the UBC, BOCA, and NEHRP, are so small that they can be considered together and the NEHRP has the most detail regulations for seismic design.

It is necessary to assume the trial member sizes at the beginning of member design to determine the bending moment distribution. However, this procedure is too complicated. In this study the moment distributions both for the vertical and horizontal loadings are assumed as shown in Figs. 3-9(a) and (b), respectively, based on previous design experiences.

As in the NEHRP provisions, the load combinations of the working gravity load, DL+LL, wind load, WL, and seismic load, EL, can be written as follows:

$$\mathbf{DL + LL} \tag{3.2}$$

$$\mathbf{k_W \cdot (DL + LL + WL)} \tag{3.3}$$

$$\mathbf{k_E \cdot (k \cdot DL + LL + EL)} \tag{3.4}$$

where k_W and k_E are the coefficients to reduce to the working load level as explained in Section 2.1,

$$k_W = 0.75$$

$$k_E = 0.654 \text{ (for OMRSF)}, 0.573 \text{ (for SMRSF)}$$

and k is defined by

$$k = 1.1 + 0.5A_V$$

$$A_V = 0.4 \text{ (for CA)}, 0.15 \text{ (for NY)}$$

The live load reduction is be determined according to the UBC and BOCA codes, because there is no description concerning the live load reduction in the NEHRP provisions. In the UBC code the formula for live load reduction, R, in percent, is given by,

$$R = r \cdot (A - 150) \qquad (\leq 40\%) \tag{3.5}$$

where r is the rate of reduction equal to 0.08 for floors and A is the area of floor supported by the member. In the BOCA code, the live load reduction is defined by,

$$L = L_0 \cdot \left(0.25 + \frac{15}{(A_i)^{1/2}} \right) \tag{3.6}$$

where L is the reduced design live load (psf), L_0, the unreduced design live load (psf), and A_i, the influence area (ft^2) taken as two times the tributary area for a beam. In this analysis, $A=24\times24=576(ft^2)$ for the UBC code and $A_i=2\times(24\times24)=1152(ft^2)$ for the BOCA code. Substituting these values into the above formulas gives the following:

$$R = 34.1(\%),\ \ L = 0.692L_0 \tag{3.7}$$

Considering these results, the live load reduction is assumed to have a constant value of 30% for this study.

Under the above assumptions, the design moments and axial forces for columns and beams can be derived[3.1]. The fixed end moment of beams for gravity loads, M_F, is given as follows:

$$\begin{aligned} M_F &= (w_D+w_L)\cdot L^2/12 \\ {}_WM_F &= M_F \\ {}_EM_F &= (k\cdot w_D+w_L)\cdot L^2/12 \end{aligned}$$

where ${}_WM_F$ and ${}_EM_F$ are the fixed end moments of beams for gravity loads used in the combination of the wind and the seismic loading, respectively. N_j is the axial force caused by gravity loads at the working load level at level j as given by the following:

$$\begin{aligned} N_j &= (w_D+w_L)\cdot l\cdot(n-j+1) && (j=1,2,\text{---},n) \\ {}_WN_j &= N_j && (j=1,2,\text{---},n) \\ {}_EN_j &= (k\cdot w_D+w_L)\cdot l\cdot(n-j+1) && (j=1,2,\text{---},n) \end{aligned}$$

where ${}_WN_j$ and ${}_EN_j$ are the axial force caused by gravity loads used in the combination of the wind and the seismic loading, respectively. M_{Tj} and M_{Bj} are the moments at the top of columns at level j and at the bottom of columns as shown in Fig. 3-9.

$$\begin{aligned} M_{Tj} &= (1-y_j)\cdot Q_j\cdot h/n_s && (j=1,2,\text{---},n) \\ M_{Bj} &= y_j\cdot Q_j\cdot h/n_s && (j=1,2,\text{---},n) \end{aligned}$$

where w_D: dead load (kips/ft),

w_L: reduced live load (kips/ft),

y_j: ratio defining location of inflection point in column at level j as shown in Fig. 3-9,

Q_j: design story shear at level j determined by wind or seismic load,

h: story height,

n_s: number of bays.

i) Interior columns

The design moments, C_{ij}, and axial forces, N_{ij}, for the interior columns are determined by (subscript i denoting interior column),

$$_LC_{ij} = 0$$
$$_LN_{ij} = N_j$$

$$_SC_{ij} = k_S \cdot \mathrm{Max.}[_SM_{Tj},\ _SM_{Bj}]$$
$$_SN_{ij} = k_S \cdot {_SN_j}$$

where the subscript L stands for vertical loading condition and S stands for W in the case of wind loading or E in the case of seismic loading.

ii) Exterior columns

The design moments, C_{ej}, and axial forces, N_{ej}, for the exterior columns are (subscript e denoting exterior column),

$$_LC_{ej} = 0.6M_F \qquad (j=n)$$
$$= 0.4M_F \qquad (j=1,2,\text{---},n\text{-}1)$$

$$_LN_{ej} = 0.5N_j$$

$$_SC_{ej} = k_S \cdot (0.5 \cdot {_SM_{Tj}} + 0.6 \cdot {_SM_F}) \qquad (j=n)$$
$$= k_S \cdot (0.5 \cdot {_SM_{Tj}} + 0.4 \cdot {_SM_F}) \qquad (j=2,3,\text{---},n\text{-}1)$$
$$= k_S \cdot \mathrm{Max.}[0.5 \cdot {_SM_{Tj}} + 0.4 \cdot {_SM_F},\ 0.5 \cdot {_SM_{Bj}} + 0.2 \cdot {_SM_F}] \qquad (j=1)$$

$$_SN_{ej} = k_S \cdot ({_LN_{ej}} + 0.5 \cdot {_SM_{Tj}}/l) \qquad (j=n)$$

$$= k_S \cdot [{_LN_{ej}} + 0.5 \sum_{i=j}^{n-1} ({_SM_{Ti}} + {_SM_{Bi+1}})/l + 0.5 \cdot {_SM_{Tn}}/l] \qquad (j=1,2,\text{---},n\text{-}1)$$

iii) **Beams**

The beam design moments, B_j, are given by,

$${}_L B_j = M_F$$

$${}_S B_j = k_S \cdot (0.5 \cdot {}_S M_{Tj} + {}_S M_F) \qquad (j=n)$$
$$= k_S \cdot [0.5 \cdot ({}_S M_{Bj+1} + {}_S M_{Tj}) + {}_S M_F] \qquad (j=1,2,\cdots,n\text{-}1)$$

Fig. 3-10 gives comparisons of the beam design momonts of the 3-, 9-, and 15-story OMRSF's and SMRSF's located in CA and NY. The dead load is 75 psf and the L/B ratio is 1.0. The bending moment that governs the beam design are indicated by the shaded area. It can be recognized that the top two or three stories may be governed by the gravity loads both in CA and NY and also that the criterion controlling most of the beam design is to seismic in CA and wind in NY. For the 15-story buildings in CA, because the wind and seismic design loads for the SMRSF's are fairly close, the buildings shall be designed as SMRSF rather than OMSRF from the viewpoint of economy and efficient member use, even though the connection details have to meet more strict requirements. However, for the 15-story buildings in NY, the wind design loads exceed the seismic design loads both for the OMRSF's and SMRSF's. There is no need to design these buildings as SMRSF's. Moreover, there is a possibility to choose a smaller R-factor, the response modification factor defined in Formula (2.6), in the design procedure, which results in a smaller ductility demand, because the beams determined by wind load have an overstrength against the reduced seismic design loads.

Figs. 3-11 to 3-15 show the governing criteria for beam design for the various building configurations with different dead load values. For the buildings selected in this study, the beam design is usually governed by wind loading if they are located in NY and by seismic if located in CA.

4. SEISMIC REQUIREMENTS OF WIND GOVERNED BUILDINGS

4.1 Strength and Ductility Requirements

Buildings are usually designed to behave elastically under working wind load and the elastic limit of strength may be the most important consideration. In seismic-resistant design, dual criteria are generally used: the buildings are designed to resist moderate earthquakes without structural damage and to resist major earthquakes without collapse. It would be uneconomical to design buildings to withstand major earthquakes that might occur once or a few times during the life of the building without damage. Therefore, the ductility of the structures may be the most important factor, because the post-elastic deformation is generally depended on for the energy absorption capacity of the structure.

The two recently introduced seismic codes: the NEHRP provisions and the Japanese seismic code[4.1], adopted the static analysis method using force reduction factor (R) and ductility factor (C_d) without actually carrying out a nonlinear inelastic analysis. The R and C_d factors are determined as follows:

$$R = Q_{el}/Q_{pl}, \quad C_d = u_{pl}/u_y \tag{4.1}$$

where Q_{el}: maximum internal force of an elastic system,

Q_{pl}: maximum internal force of an elastplastic system,

u_{pl}: maximum lateral deflection of an elastplastic system,

u_y: yield lateral deflection.

Using this method the plastic behavior of a structure under a severe earthquake can be predicted based on two well-known concepts (assumptions): equal maximum deflection response and equal maximum energy response. These concepts are illustrated in Figs. 4-1(a) and (b)[4.2].

The assumption of equal maximum deflections is based on the observation from dynamic analyses that the maximum deflections reached by an elastic system and an elastoplastic system may be approximately the same. Referring to Fig. 4-1(a), this assumption would give the following relationship:

$$\mathbf{R = C_d} \tag{4.2}$$

In the NEHRP provisions, R and C_d are nebulously called, respectively, the "Response Modification Coefficient" and the "Deflection Amplification Factor", without providing detailed explanations. It is evident, however, that the NEHRP provisions are based on the assumption of equal maximum deflections when the tabulated R and C_d values are plotted as shown in Fig. 4-2.

Some dynamic analyses have indicated that the equal maximum deflection assumption may be unconservative. Blume[4.2] has shown that a probable upper limit for R is,

$$R = \sqrt{2C_d - 1} \tag{4.3}$$

This equation is based on the equal energy concept, which implies that the energy stored in the elastic system at the maximum deflection, u_{el}, is the same as that stored in the elastoplastic system at the maximum deflection, u_{pl}. The Japanese seismic code is based on this assumption and a comparison of Eq.(4.3) and the values of R and C_d regulated in this code are shown in Fig. 4-3. It is recognized that the R values in Fig. 4-3 are conservative compared to those in Fig. 4-2. A brief description of the Japanese code is given in Appendix 2.

In the structural design procedure, both wind load and seismic load are considered as external lateral forces, although their characteristics are completely different. Their design magnitude and vertical distribution over the height are also different, as shown in the previous sections, because they depend on not only the locations where buildings are constructed but also the structural systems (moment-resisting frames, braced frames, or shear wall structures), materials (steel, reinforced concrete, or masonry) and connection details (special detailing for achieving ductile behavior, or ordinary detailing). The NEHRP provisions have a wide-range of R factors which vary from 1.25 to 8.0 and C_d factors from 1.25 to 6.5 as shown in Fig. 4-4.

4.2 Ultimate Strength of Buildings

The R and C_d factors may be estimated by performing elastic-plastic analysis of structures and determining their ultimate strength. The ultimate strength of the various buildings included in this study has been analyzed by the mechanism method (upper bound theorem) under the assumption that there exist only three types of collapse mechanism for moment-resisting frames designed according to the procedure outline in Section 3.3. These mechanisms are shown in Fig. 4-5. The analysis method used in this study is explained in Appendix 3[4.3]. The load factor of ultimate strength, λ, is determined to be the smallest of the three values given by Eqs. (A3.5), (A3.10), and (A3.13) in Appendix 3 using the NEHRP pattern of vertical seismic load distribution.

$$\lambda = \text{Min.}[\lambda_C, \lambda_{CB}, \lambda_B] \tag{4.4}$$

where λ_C, λ_{CB}, and λ_B are the load factors for ultimate strength of column type sway mechanism, combined mechanism, and beam type sway mechanism, respectively.

The results of ultimate strength analyses are presented in Figs. 4-6 to 4-14. The dimensions and parameters of the structures analyzed are those given in Chapter 3. The story shear at each level, Q_j in Eqs. (A3.4), (A3.9), and (A3.13) in Appendix 3, for determining the λ-values is assumed to be the NEHRP design seismic load for an elastic system, which has the base shear given by Eq. (2.5) when R=1 and the distribution defined by Eq. (2.14). Therefore, in this study, the λ-value is the load factor of ultimate strength against the seismic design load for an elastic system.

Fig. 4-6 to Fig. 4-10 show that all of the collapse mechanism is the column type sway mechanism and the λ-values for the OMRSF's in CA are in the range of 0.45 to 0.35 and those for the SMRSF's are 0.25 to 0.2. The ultimate story shear capacity, Q_{ult}, for OMRSF is predicted by using the overstrength factor against the design load, Q_{pl}, under the assumption that all the columns are designed for the allowable stress, $0.6\sigma_y$ at the working load level and their axial force ratios are in the range of $N/N_p > 0.15$[4.4], for simplisity.

$$\begin{aligned} Q_{ult} &= 1.18 \times 1.14 \times (0.654/0.6) \cdot Q_{pl} \\ &= 1.465 \cdot Q_{pl} \end{aligned} \tag{4.5}$$

For SMRSF, the same procedure is available, that is,

$$\begin{aligned} Q_{ult} &= 1.18 \times 1.14 \times (0.573/0.6) \cdot Q_{pl} \\ &= 1.285 Q_{pl} \end{aligned} \tag{4.6}$$

This ultimate strength can be defined by using λ-factor determioned in Eq. (4.4) as follows:

$$Q_{ult} = \lambda \cdot Q_{el} \tag{4.7}$$

On the other hand, the design seismic load for an elastoplastic system, Q_{pl}, is determined from the design seismic load for an elastic system, Q_{el}, using R-factor as follows:

$$Q_{pl} = (1/R) \cdot Q_{el} \tag{4.8}$$

Substituting Eq. (4.5) into Eqs. (4.7) and (4.8) gives the following relationship for the OMRSF's:

$$\lambda = \frac{1.465}{R} \tag{4.9}$$

$$= 0.326 \text{ (when R=4.5)}$$

Substituting Eq. (4.6) also into Eqs. (4.7) and (4.8) gives the following for the SMRSF's:

$$\lambda = \frac{1.285}{R} \tag{4.10}$$

$$= 0.161 \text{ (when R=8.0)}$$

The relationship among Q_{ult}, Q_{pl}, and Q_{el} is indicated schematically in Fig. 4-15.

Considering the gravity load effect on column design, it is recognized that these results are reasonable for seismic-governed buidings. Actually, low rise buildings have greater λ-values than those for 12- to 15-story buildings, because the lower the buildings, the more controlling the gravity load becomes in the member design. However, the λ-values in NY vary from 0.50 to 1.25, which are much more than those in CA, because the building design is not determined by seismic loads but by wind loads and hence, most of the buildings in NY have the same λ-values whether or not they are designed as OMRSF or SMRSF.

In Figs. 4-5 to 4-9, the number shown just above the solid or dotted lines indicates the level at which the collapse mechanism occurs. If the number is 2, the collapse occurs at level 2. In this study all the collapse mechanisms are column type sway mechanisms, but the level where the collapse occurs is different for different structures as shown in Figs. 4-11 and 4-12.

For example, Fig. 4-13 shows the load factor for ultimate strength of each mechanism, level by level, of the 15-story OMRSFs with 75 psf, L/B=1.0, located in CA and NY. Figs. 4-14(a) and 4-14(b) illustrate the ultimate strength of the same buildings. These figures explain why the level at which the collapse occurs is different. The distribution shape of story moment capacity, the sum of column plastic moment capacity, which is presented as the solid line in Fig. 4-14(a) in CA is different from that in NY in Fig. 4-14(b). Therefore, the factored seismic force, which is shown as the dot-dashed line both in Figs. 4-14(a) and (b), reaches at level 2 in the seismic-governed buildings in CA while at level 12 in the wind-governed building in NY. The vertical load effect on the column design is the smallest at level 2, because the base columns at level 1 are fixed on the ground and they have the extra moment capacity for ultimate moment distribution. Therefore, for seismic-governed buildings, level 2 is the weakest story for seismic loading and the collapse mechanism generally forms at that level.

4.3 Required Ductility of Buildings

Using the obtained λ-values, the R-factors are evaluated by Eqs. (4.9) and (4.10) for various buildings including the wind governed buildings. The R-factors for buildings located both in CA and in NY are plotted in Fig. 4-16. In CA, the average value of analyzed R-factors for OMRSF are 3.81 and those for SMRSF are 6.04. These values are fairly close to the design values for seismic-governed buildings, with consideration of gravity load effects on member design.

In NY, the wind design loads exceed the seismic design loads for both the OMRSF's and SMRSF's except a few cases which are all cases for 5-bay OMRSF's and the 3-story OMRSF with DL=75 psf, L/B=1.0 and the 3- and 6-story OMRSF's with DL=100 psf, L/B=1.0. There is no need to design wind-governed buildings as SMRSF's. Therefore, in Fig. 4-16, the R-factors in NY are plotted only for wind-governed buildings using the relationship in Eq. (4.9). The R-factors for wind-governed buildings in NY vary from 1.0 to 3.0 with an average of 1.88. This implies that it is possible to design buildings in wind governing regions using a much lower R-value, say R=2.0, and they do not need to be detailed to provide ductile behavior required for SMRSFs, or to have the capability of nonlinear deformation as schematically shown in Fig. 4-17, because C_d=R under the assumption of equal maximum deflection. Some of them may require only elastic strength because they will behave elastically even against the design seismic loads for an elastic system.

5. SUMMARY AND CONCLUSIONS

Several national and regional codes now require the consideration of earthquake ground excitations in the design of buildings in many parts in the U.S., even in the eastern U.S.. These codes adopt static analysis and an equivalent lateral force procedure using force reduction factor(R) and ductility factor(C_d) without carrying out any true nonlinear analysis. In the eastern U.S., the governing lateral loading for building design is the wind loading, and even on the west coast the governing lateral loading for medium to high-rise buildings is the wind loading. However, the R and C_d factors for wind governed buildings are not clearly described in these codes.

Overview of Existing Codes in The United States

The comparison of existing codes in the U.S. has been done among the UBC, BOCA, and NEHRP. The seismic design loads for multistory office buildings with moment-resisting steel frames located in California and New York have been examined.

(1) Buildings in California are designed for the base shears which are 2.5 to 4.0 times of those for buildings in New York.

(2) Special Moment Resisting Steel Frames (SMRSF) may be designed by using only half of the base shear for Ordinary Moment Resisting Steel Frames (OMRSF), but in SMRSF's all connections are to be specially detailed to provide ductility.

Within this study, the seismic design loads have been determined in accordance with the NEHRP provisions, because the difference among these three seismic codes is so small that they can be considered together and the NEHRP has the most detailed regulations for seismic design. Also, the BOCA code has been used for the wind load regulation, because the UBC and BOCA codes are almost same and, when difference occurs, it is very small and negligible.

Criteria that Control Building Design

Horizontal loads that govern the building design have been investigated for various types of buildings with different configurations, heights, and gravity loads. The buildings studied are typical low- to medium-rise office buildings with moment-resisting frames located in California and New York. The analysis of the various 3-, 6-, 9-, 12-, and 15-story buildings. For three bays with a dead load of 75 psf has led to the following conclusions:

(1) In California, all buildings are governed by seismic loads except for the 15-story SMRSF.

(2) In New York, all buildings are governed by wind loads except for the 3-story OMRSF.

The effect of gravity loads and building configurations are recognized as follows;

(3) Two or three stories from the top may be governed by gravity loads both in New York and California.

(4) The greater the gravity loads, the more controlling the seismic load. Even in New York, considerable parts of low- to medium-rise OMRSF buildings with a dead loads of 100 psf are governed by seismic loads.

(5) The greater the ratio of L/B, or the greater the number of bays, the more governing is the seismic load. In the case of the 5-bay OMRSF buildings in New York, the seismic loads govern all the stories for the 3- to 6-story low rise buildings, and several stories for the 9- to 15-story medium rise buildings.

Seismic Requirements on Wind Governed Buildings

These buildings have an overstrength when compared with the seismic design load and a greater ultimate strength than that implied in the seismic design procedure. The selected building frames have been designed for the controlling loading and analyzed for their ultimate strength by the mechanism methods (upper bound theorem). The analysis assumed the NEHRP pattern of vertical seismic load distribution.

(1) In California, the load factors of ultimate strength against the seismic design load for an elastic system, λ-values, are in the range of 0.45 to 0.35 for OMRSFs and 0.25 to 0.2 for SMRSFs. These values are larger than the estimated values in the seismic design procedure using R-factors, because the gravity loads effect on the member design.

(2) In New York, λ-values vary from 0.50 to 1.25 regardless whether the buildings are designed to be OMRSF or SMRSF, because their design is not determined by seismic loads but by wind loads except in a few cases.

The required R and C_d factors for various buildings including wind-governed buildings were derived by using the ultimate strength of buildings obtained by the analyses.

(3) The R-factors for buildings in CA are around 4.0 for OMRSF and 7.0 for SMRSF, which are fairly close to the design values for seismic-governed buildings, with consideration of the vertical load effects on the member design.

(4) The analyzed R factors vary from 1.0 to 3.0 for wind-governed buildings in New York. The C_d factors are also 1.0 to 3.0 for these buildings, corresponding to the R factors under the assumption of equal maximum deflection.

These results imply that the buildings governed by wind loads can be designed by using much lower R-factors, say R=1.0 to 3.0, than the required values regulated in the codes, and it is not necessary to detail to provide such ductile behavior as for SMRSF or to have the substantial capability of nonlinear deformations.

(5) There is a possibility to use simply detailed and economical connections, for example, top-and-seat-angle, end-plate, and T-stub connection types, the cyclic behavior of which is now being studied at Lehigh University[5.1].

The buildings analyzed in this study are limited only to moment-resisting steel frames. The moment-resisting steel frames are often favorably adapted in seismic design because of their ductile behavior. On the other hand, braced frames are considered to be inferior to the moment-resisting frames because of their deteriorated hysterisis loops under cyclic loading. Braced frames, however, are the favored system for wind-governed buildings. Since R values of only 1.0 to 3.0 is required for seismic resistance of wind-governed buildings, it appears that braced frames would also have sufficient ductility to be used in moderately active seismic regions.

Table 2-1. Base Shear Coefficient at the working load level for OMRSF

$C_{sw}=V_w/W$	California (CA)	New York (NY)
(1) UBC	$\frac{0.075}{T^{2/3}}$ (≤ 0.137)	$\frac{0.0281}{T^{2/3}}$ (≤ 0.0515)
(2) BOCA	$\frac{0.06}{T^{1/2}}$ (≤ 0.105)	$\frac{0.0225}{T^{1/2}}$ (≤ 0.0394)
(3) NEHRP	$\frac{0.0837}{T^{2/3}}$ (≤ 0.145)	$\frac{0.0209}{T^{2/3}}$ (≤ 0.0363)

Table 2-2. Base Shear Coefficient at the working load level for SMRSF

$C_{sw}=V_w/W$	California (CA)	New York (NY)
(1) UBC	$\frac{0.0375}{T^{2/3}}$ (≤ 0.0687)	$\frac{0.0141}{T^{2/3}}$ (≤ 0.0258)
(2) BOCA	$\frac{0.0402}{T^{1/2}}$ (≤ 0.0704)	$\frac{0.0151}{T^{1/2}}$ (≤ 0.0264)
(3) NEHRP	$\frac{0.0413}{T^{2/3}}$ (≤ 0.0716)	$\frac{0.0103}{T^{2/3}}$ (≤ 0.0179)

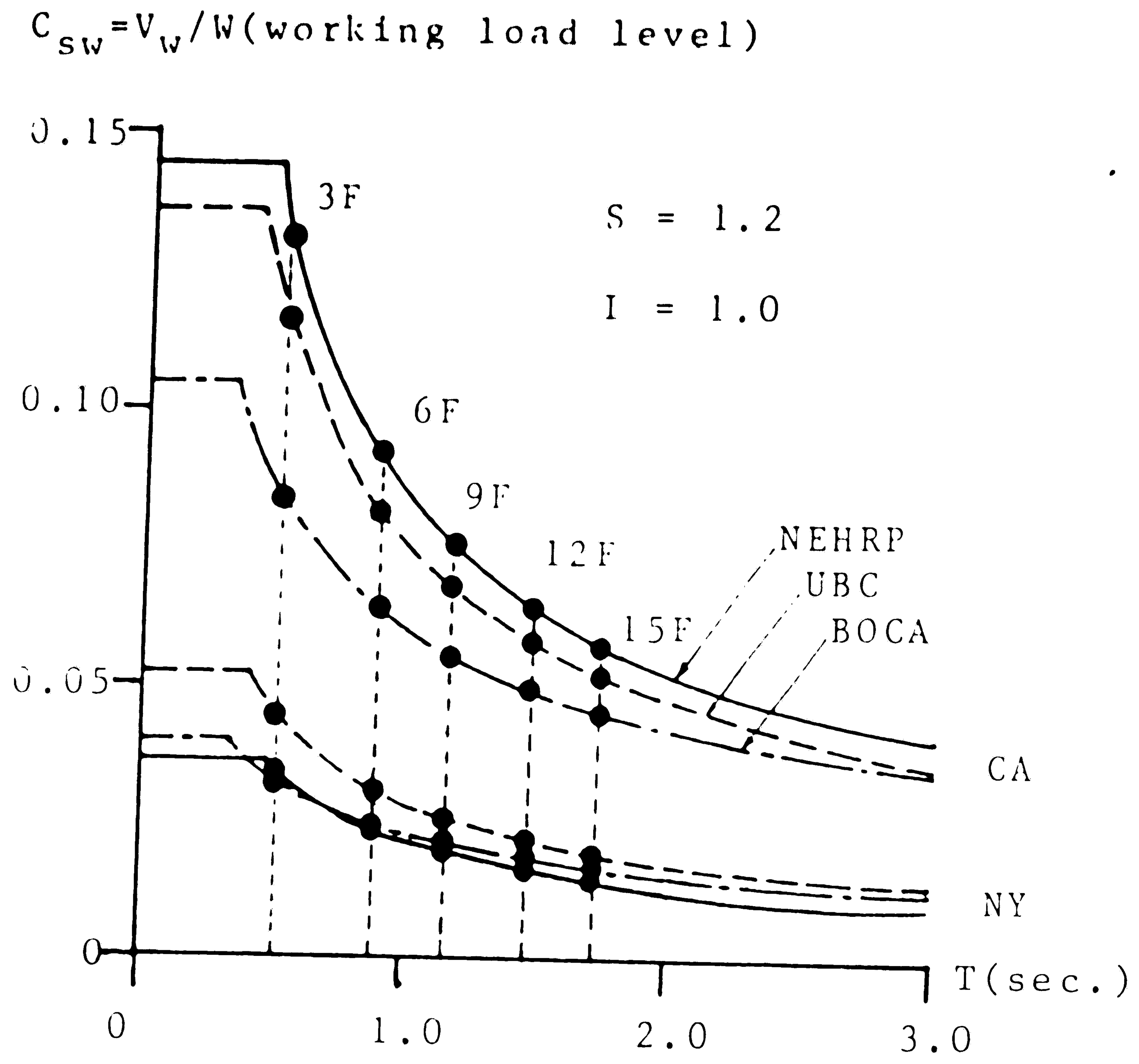


Fig.2-1 Design base shear coefficients for OMRSF

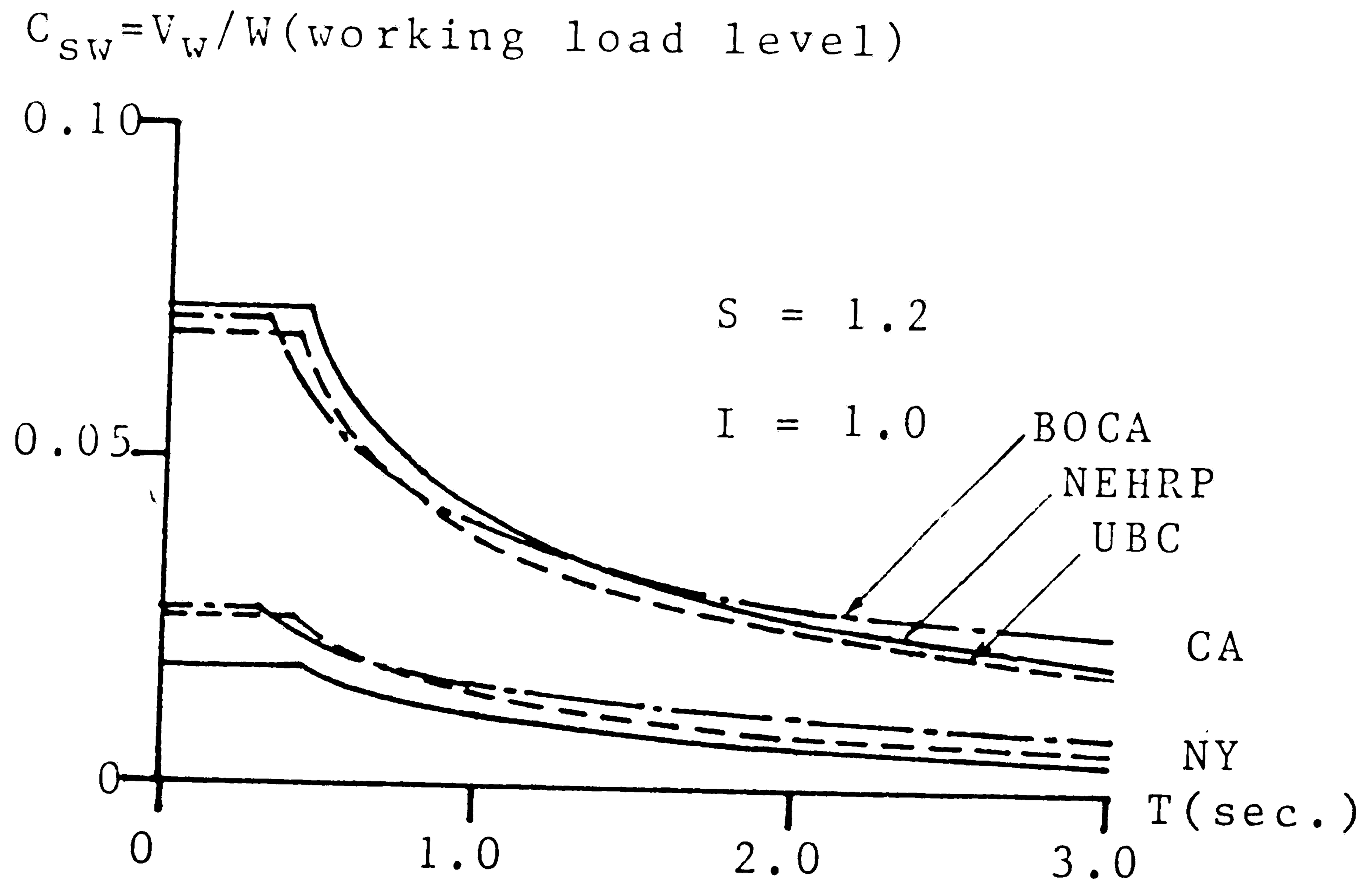
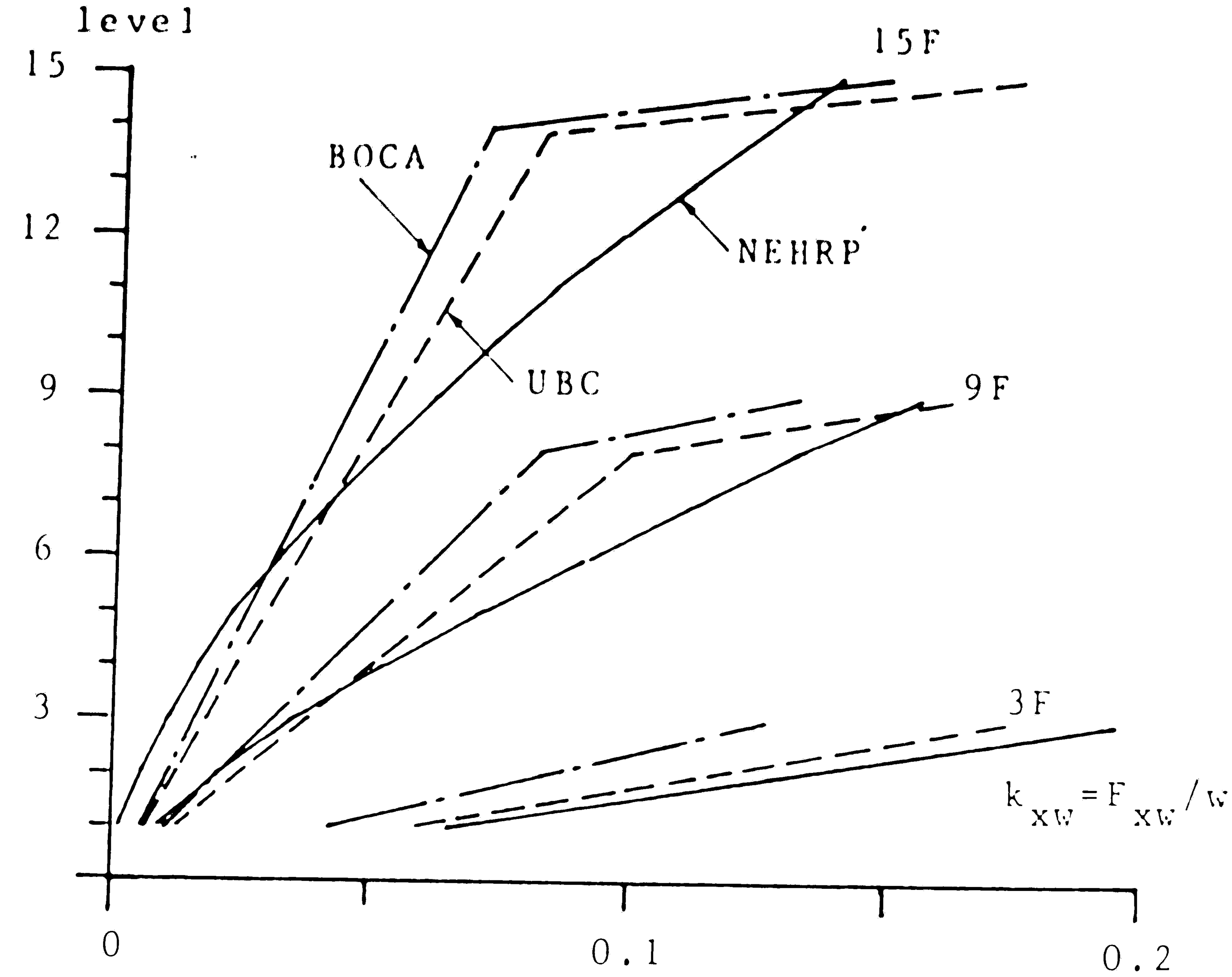
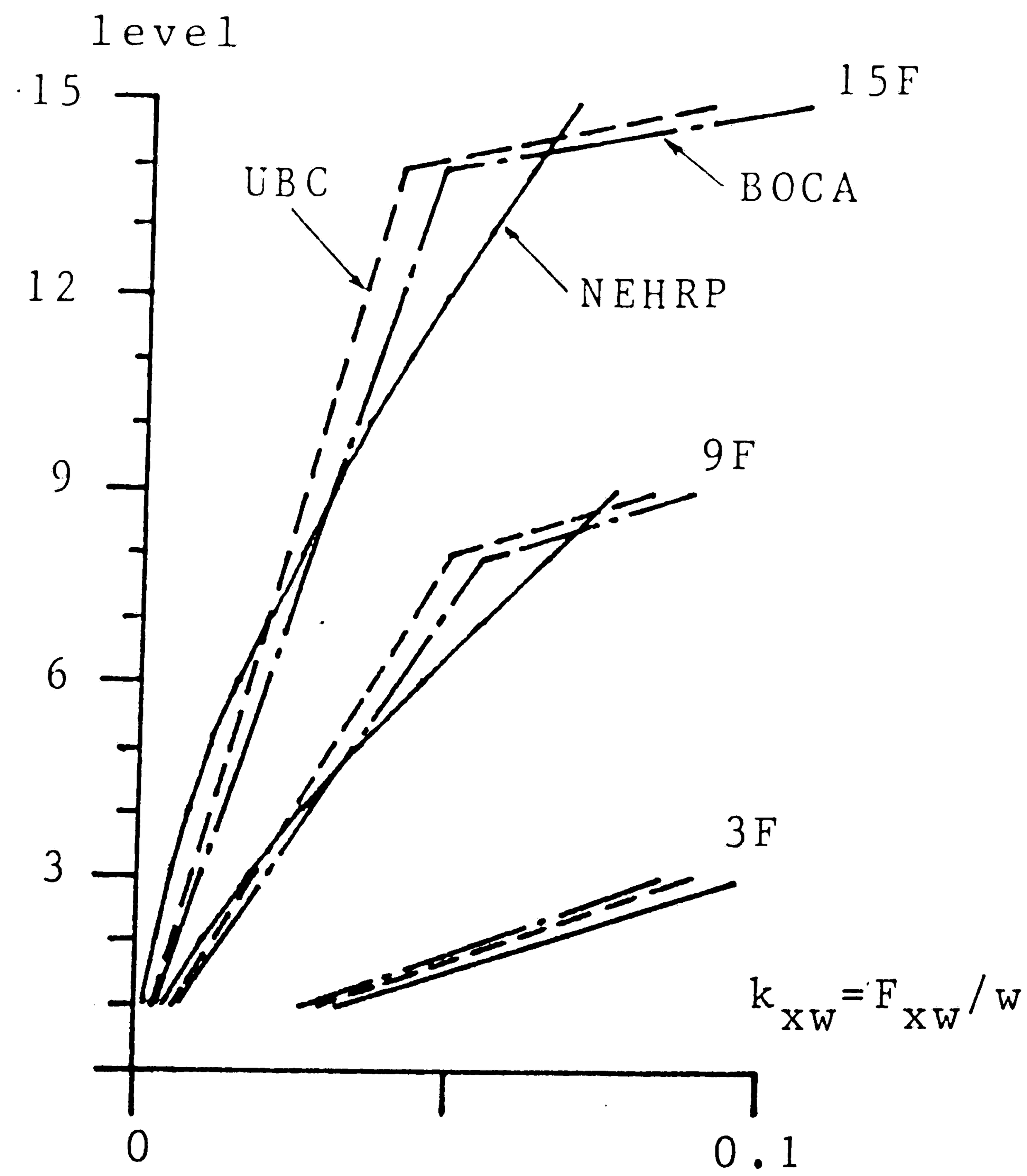


Fig.2-2 Design base shear coefficients for SMRSF



(a) OMRSF



(b) SMRSF

Fig.2-3 Vertical distribution of seismic coefficients for buildings in California

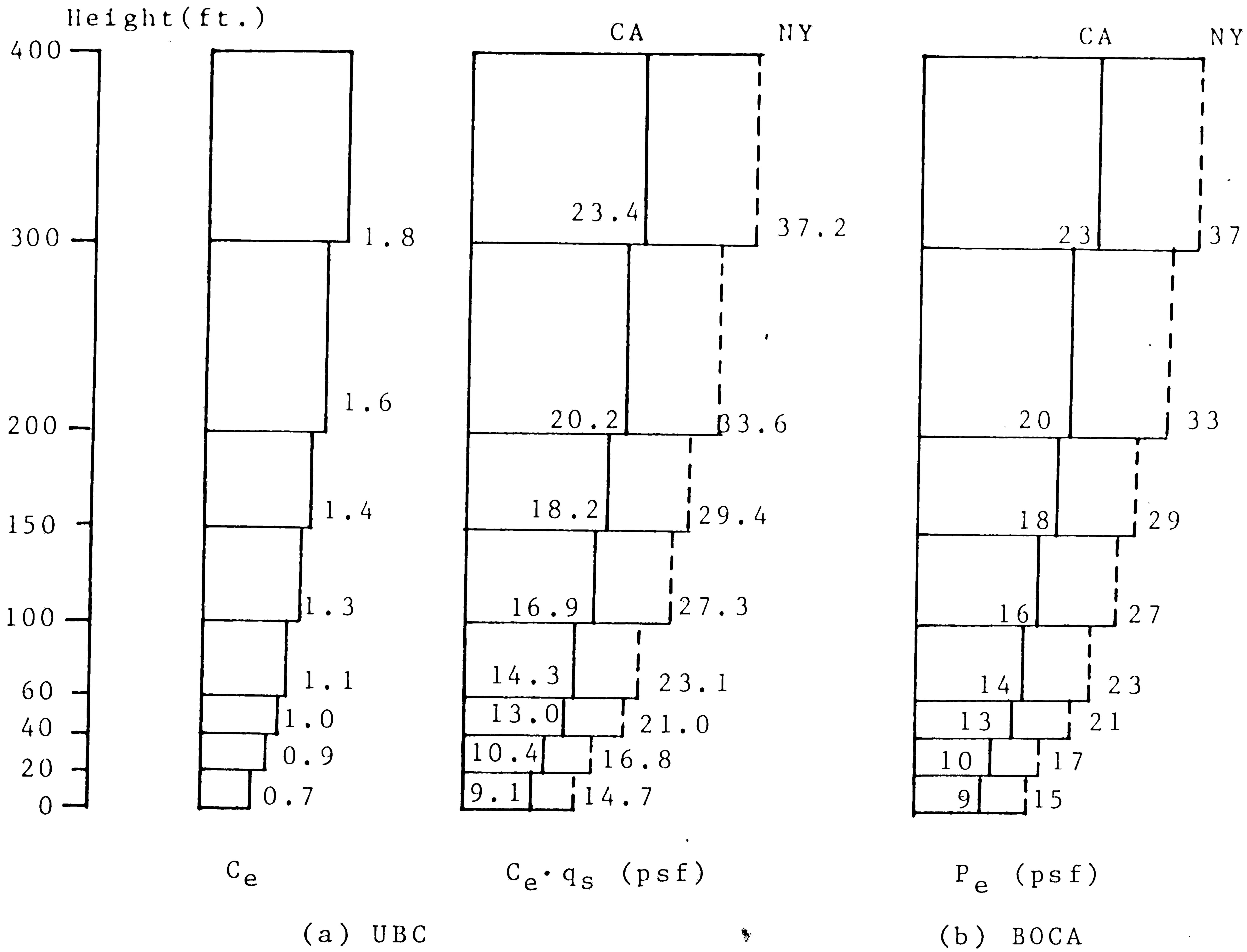


Fig.2-4 Comparison of wind pressures between the UBC and the BOCA

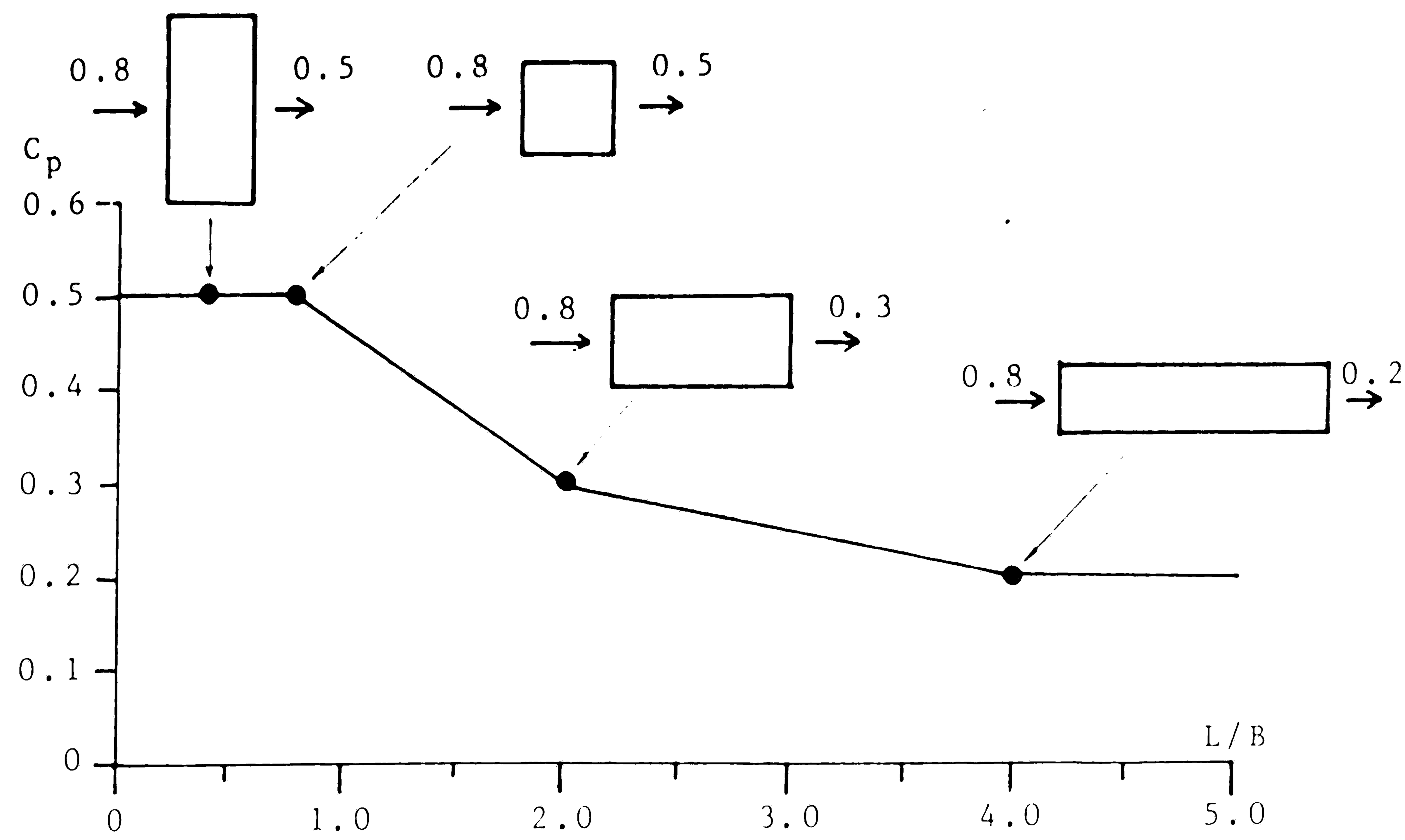


Fig.2-5 External wind pressure coefficients in the BOCA

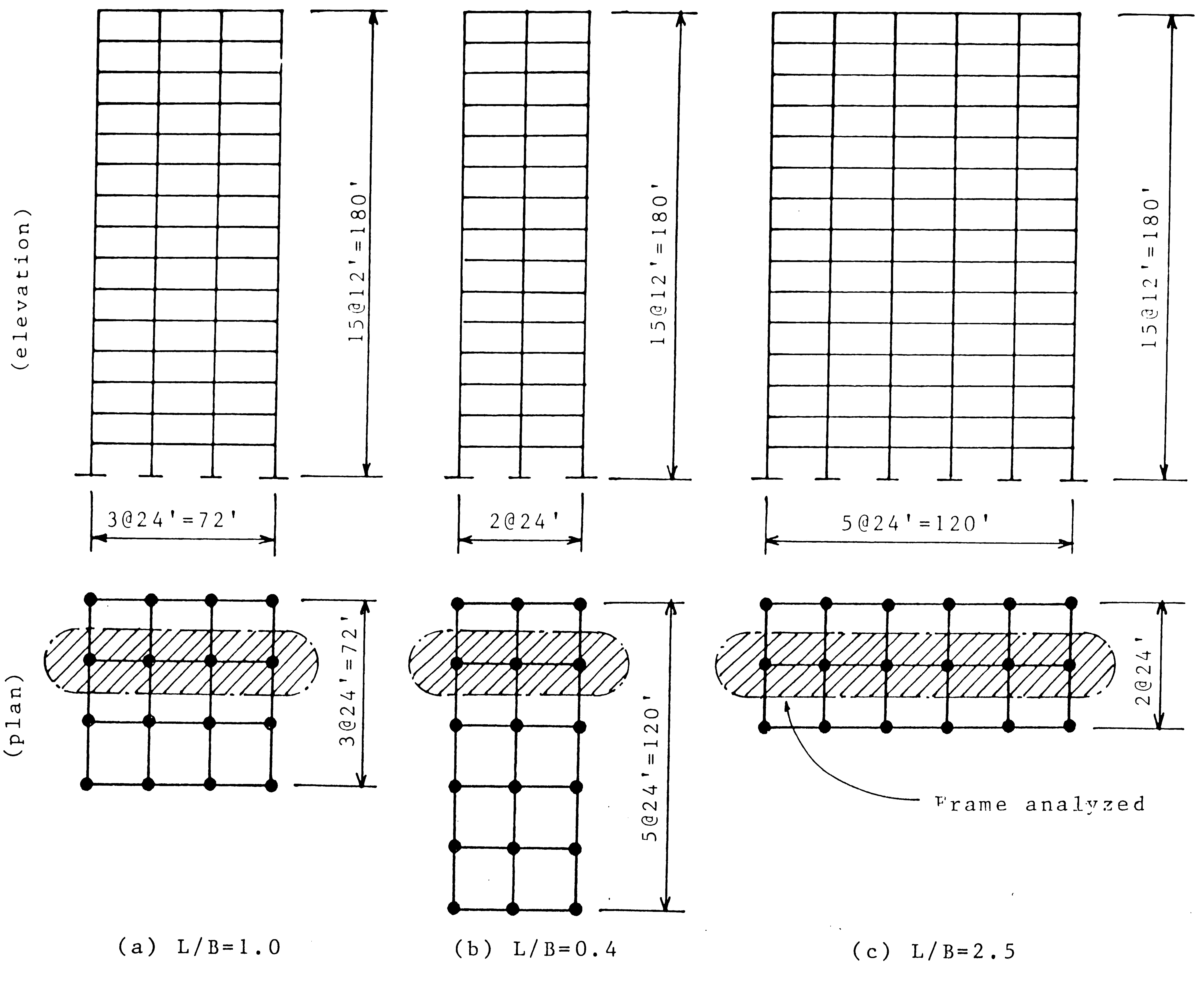
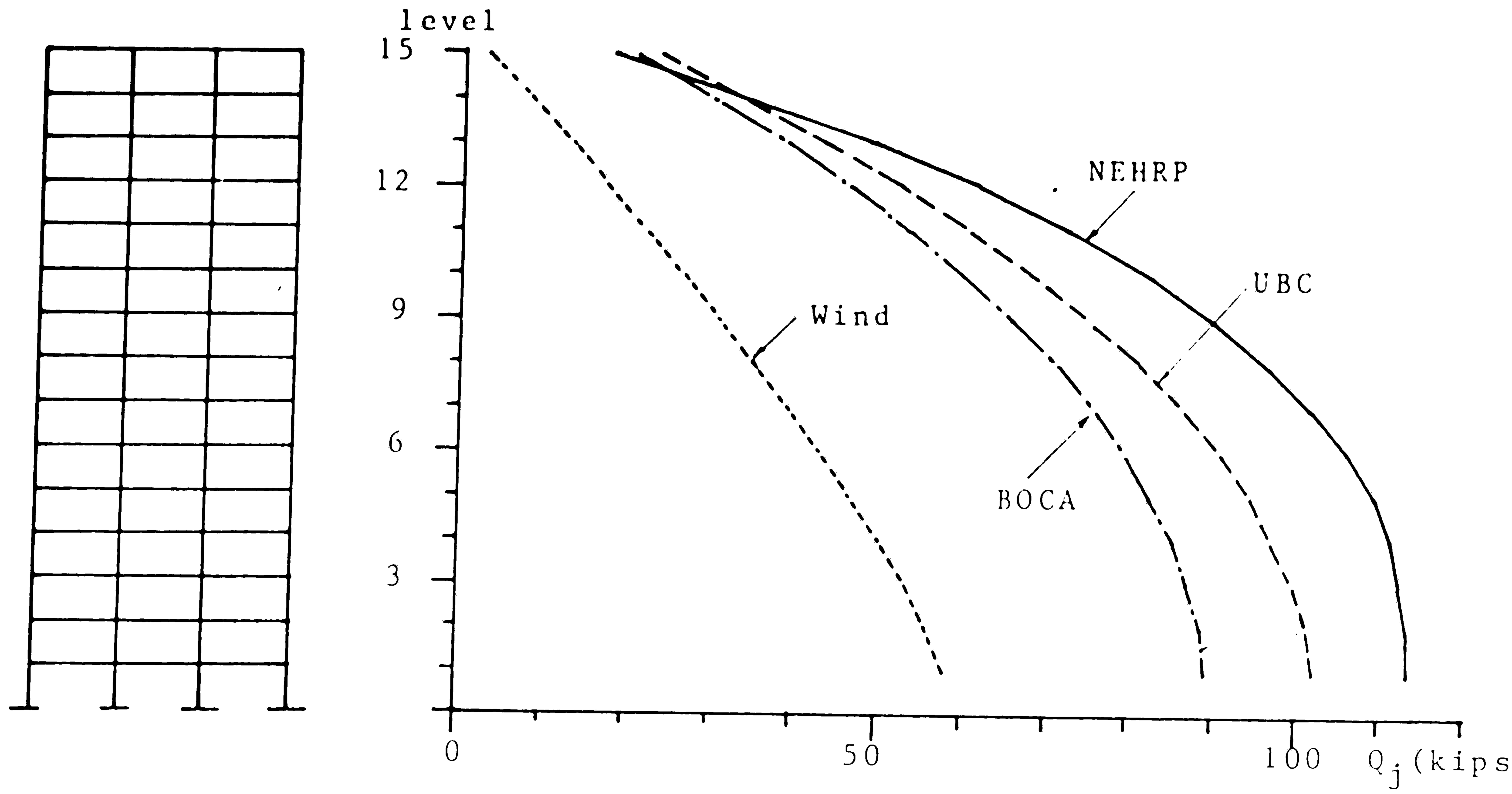
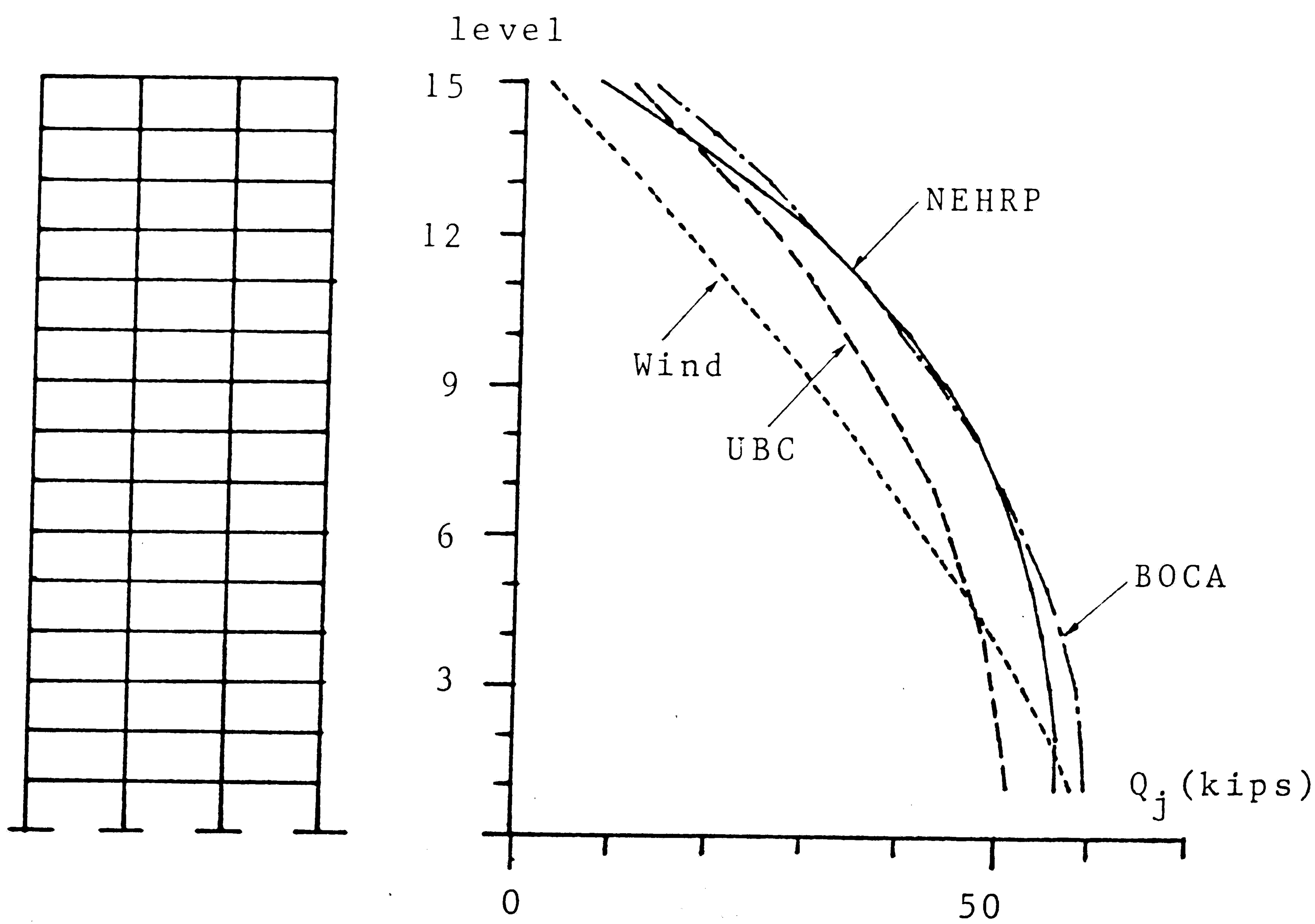


Fig.3-1 Building configurations and dimensions

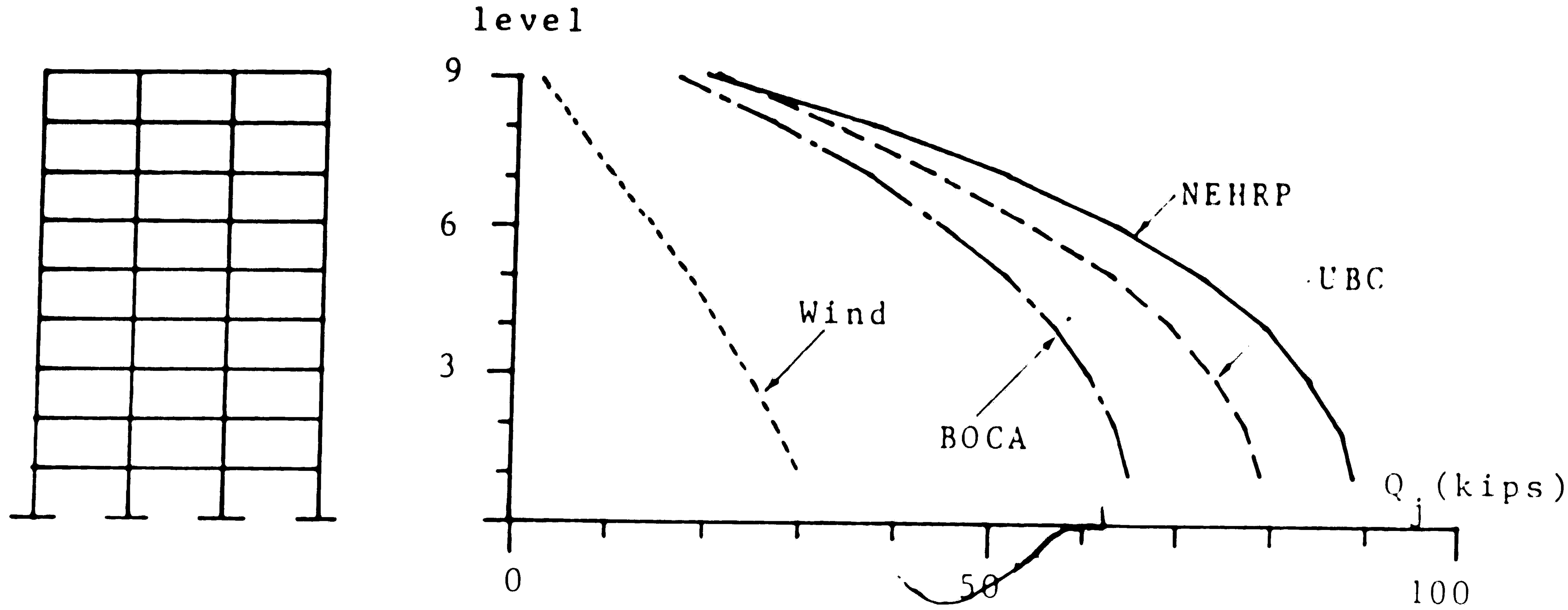


(a) 15-story OMRSF with 75psf, L/B=1.0

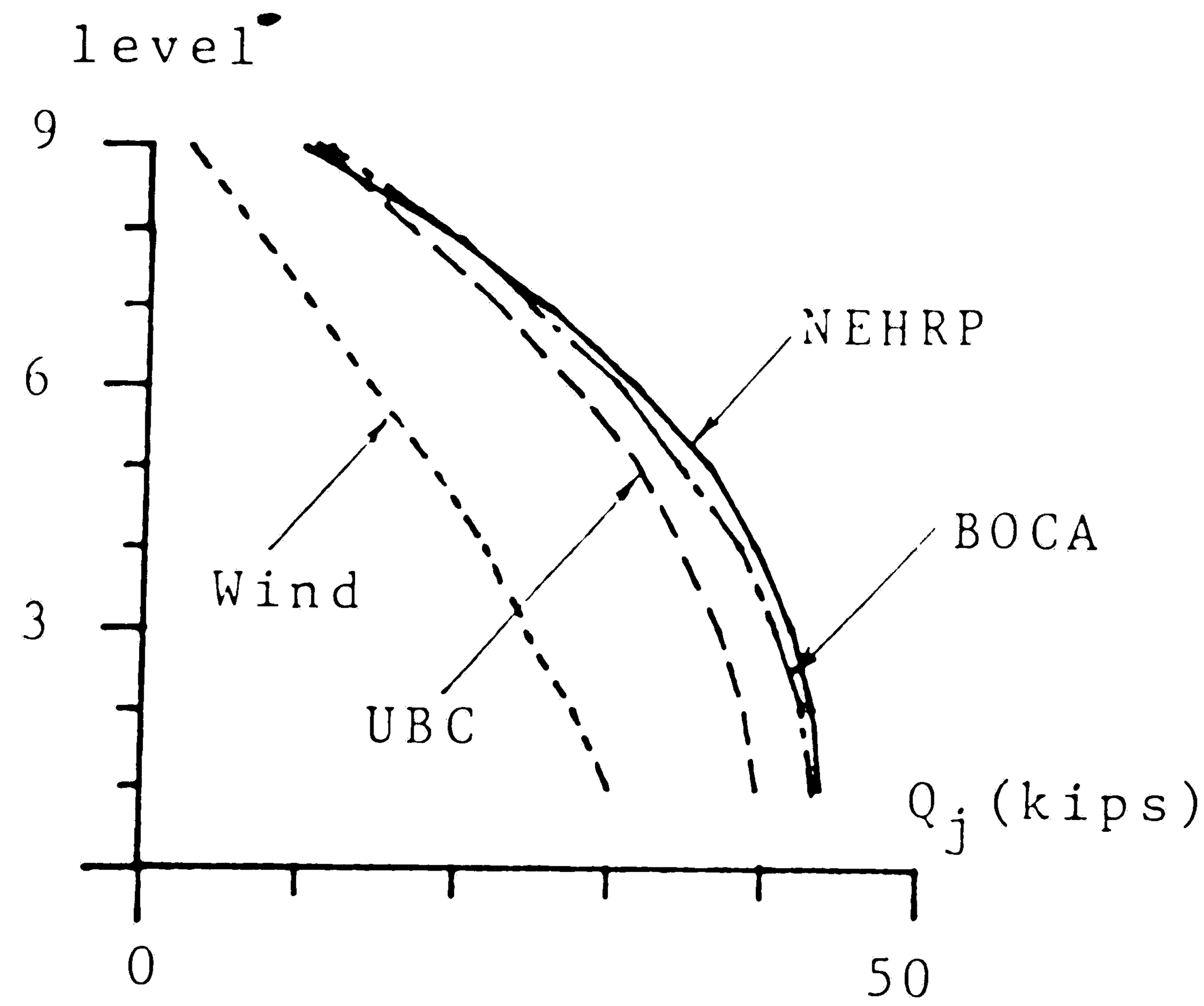


(b) 15-story SMRSF with 75psf, L/B=1.0

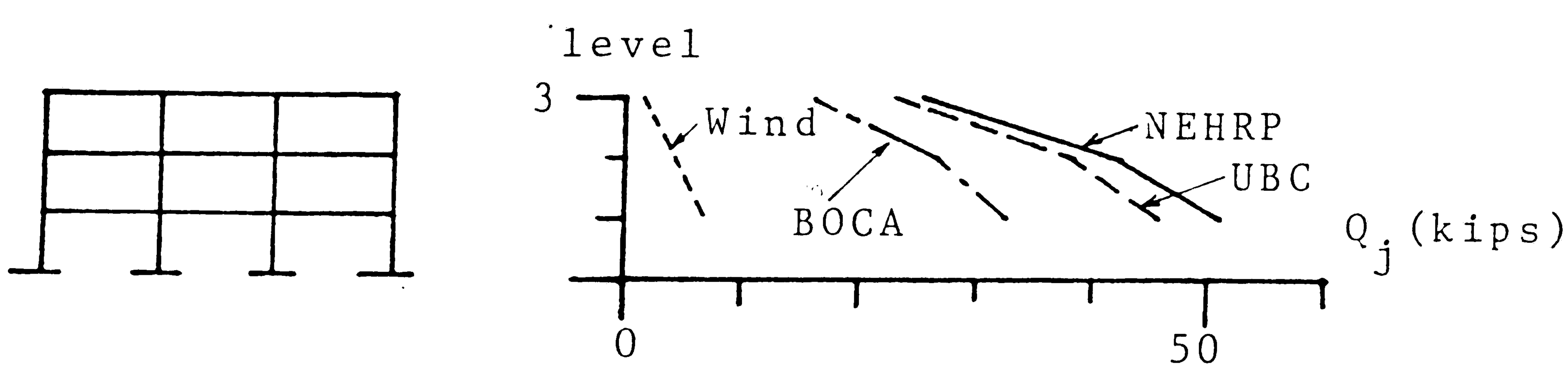
Fig.3-2 Design story shear at the working load level in California



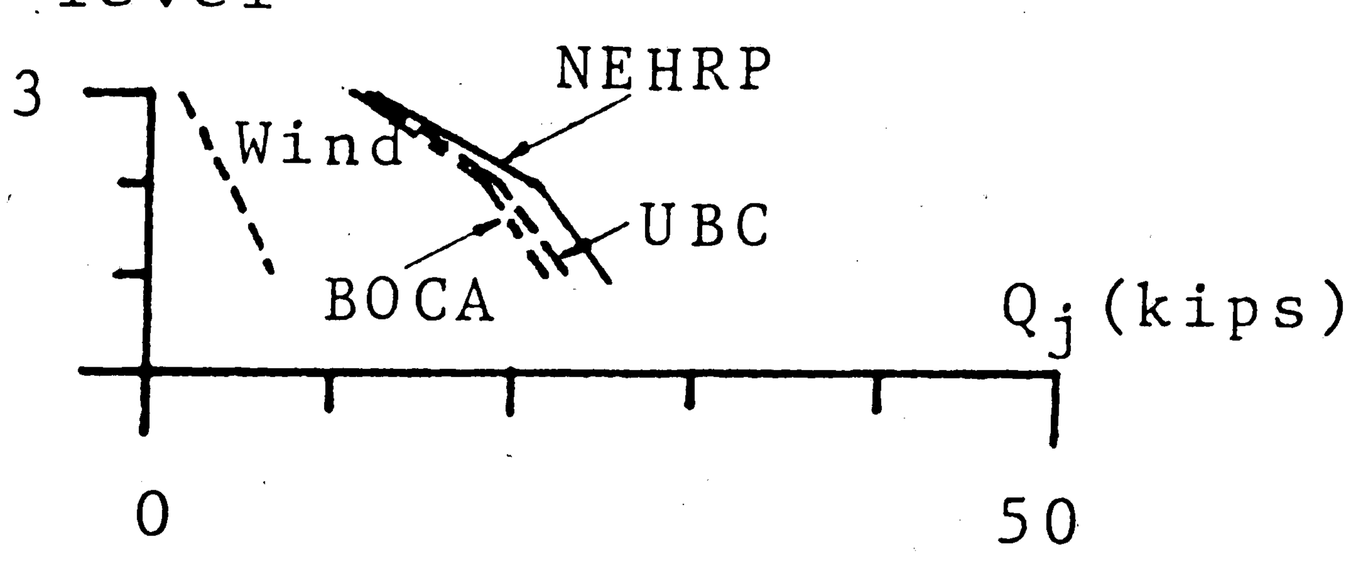
(c) 9-story OMRSF with 75psf, L/B=1.0



(d) 9-story SMRSF with 75psf, L/B=1.0

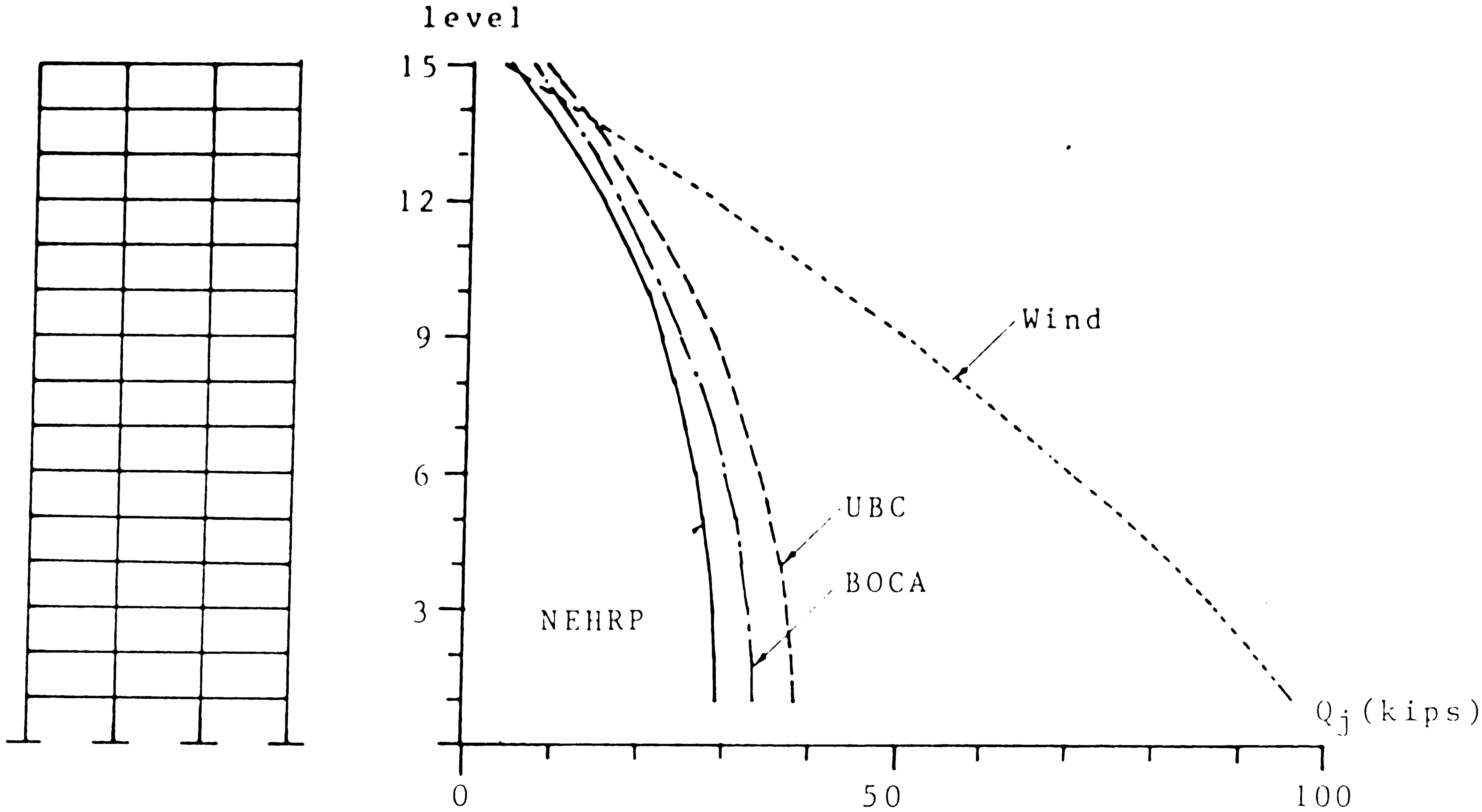


(e) 3-story OMRSF with 75psf, L/B=1.0

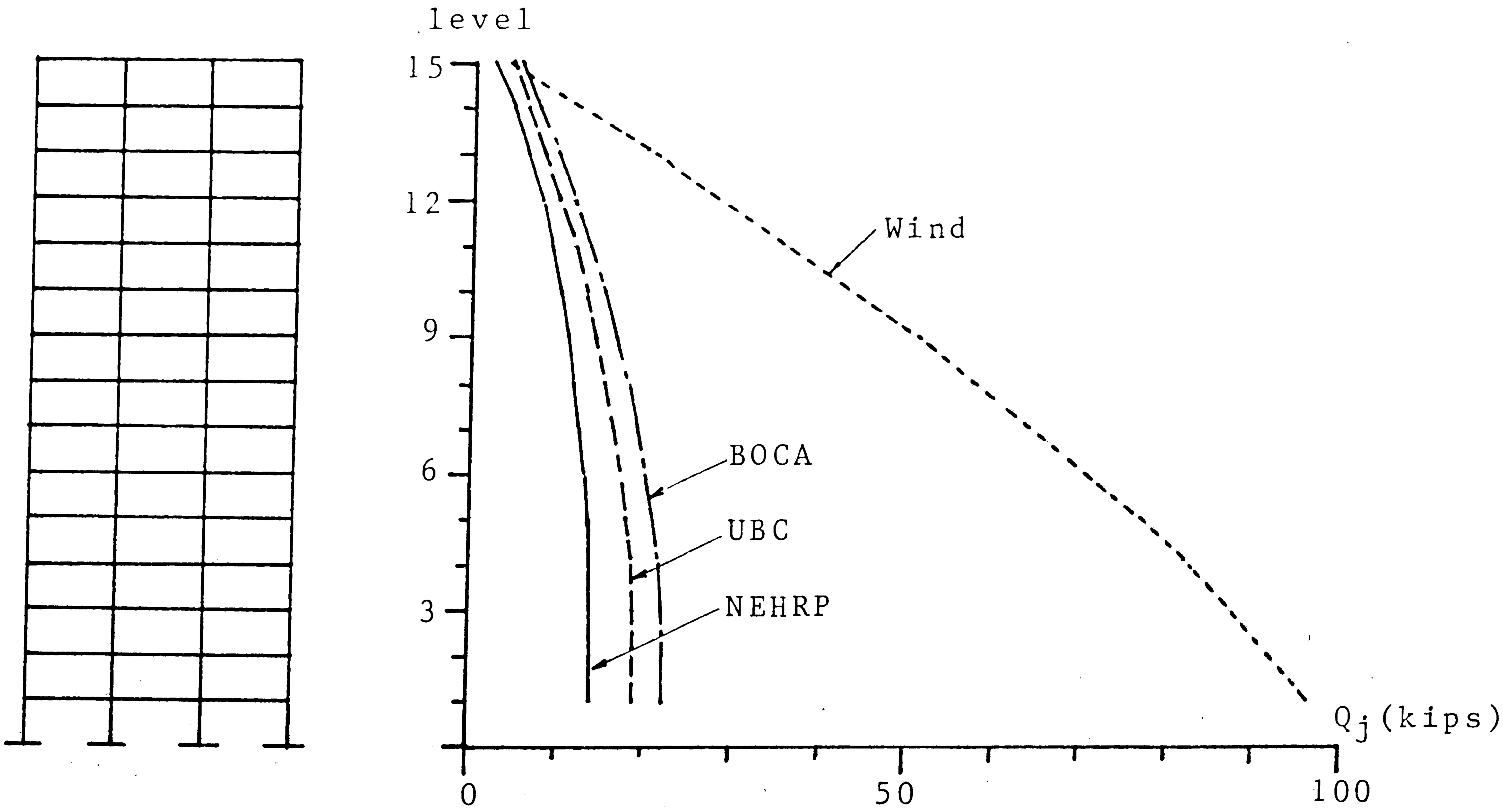


(f) 3-story SMRSF with 75psf, L/B=1.0

Fig.3-2 Design story shear at the working load level in California

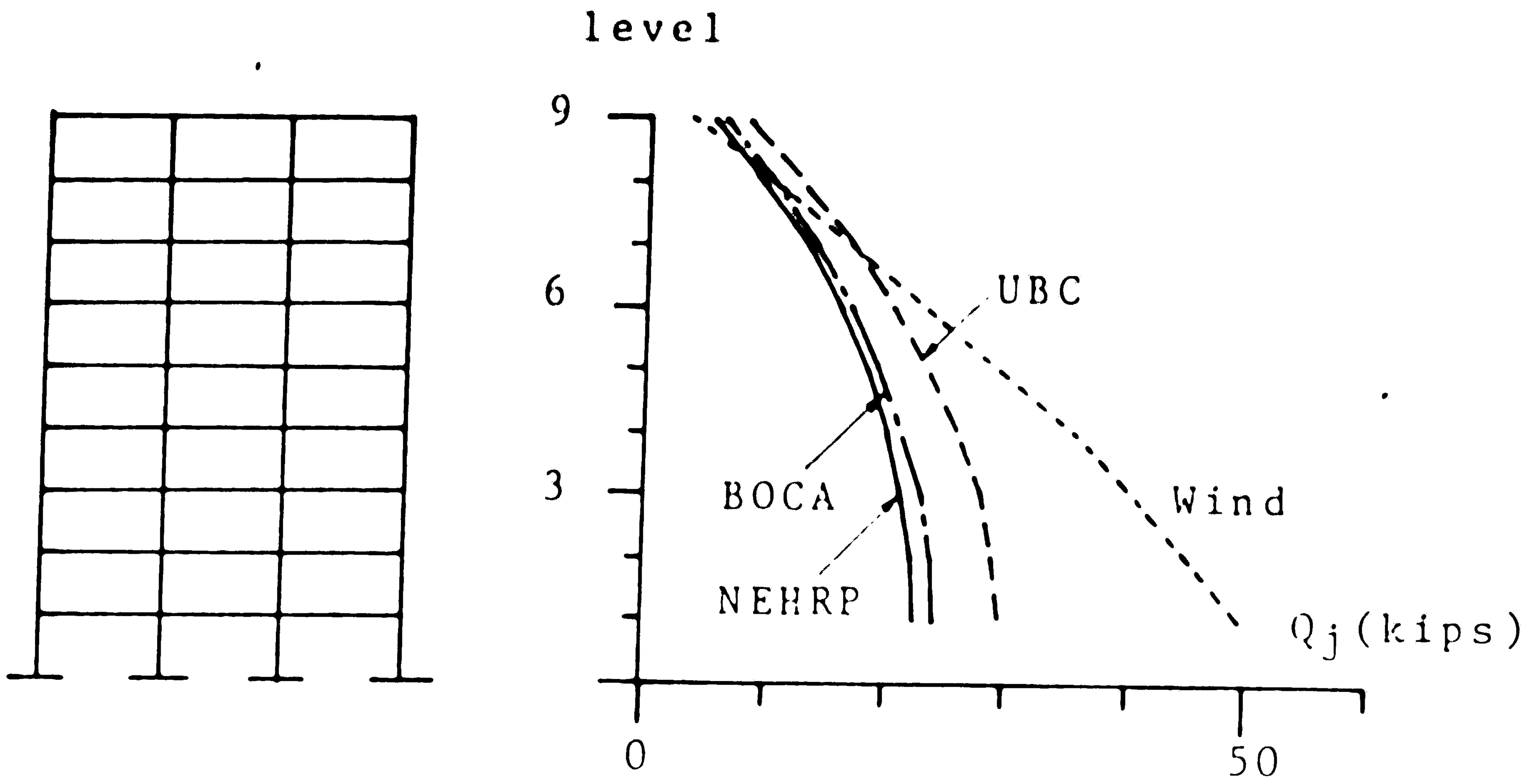


(a) 15-story OMRSF with 75psf, L/B=1.0

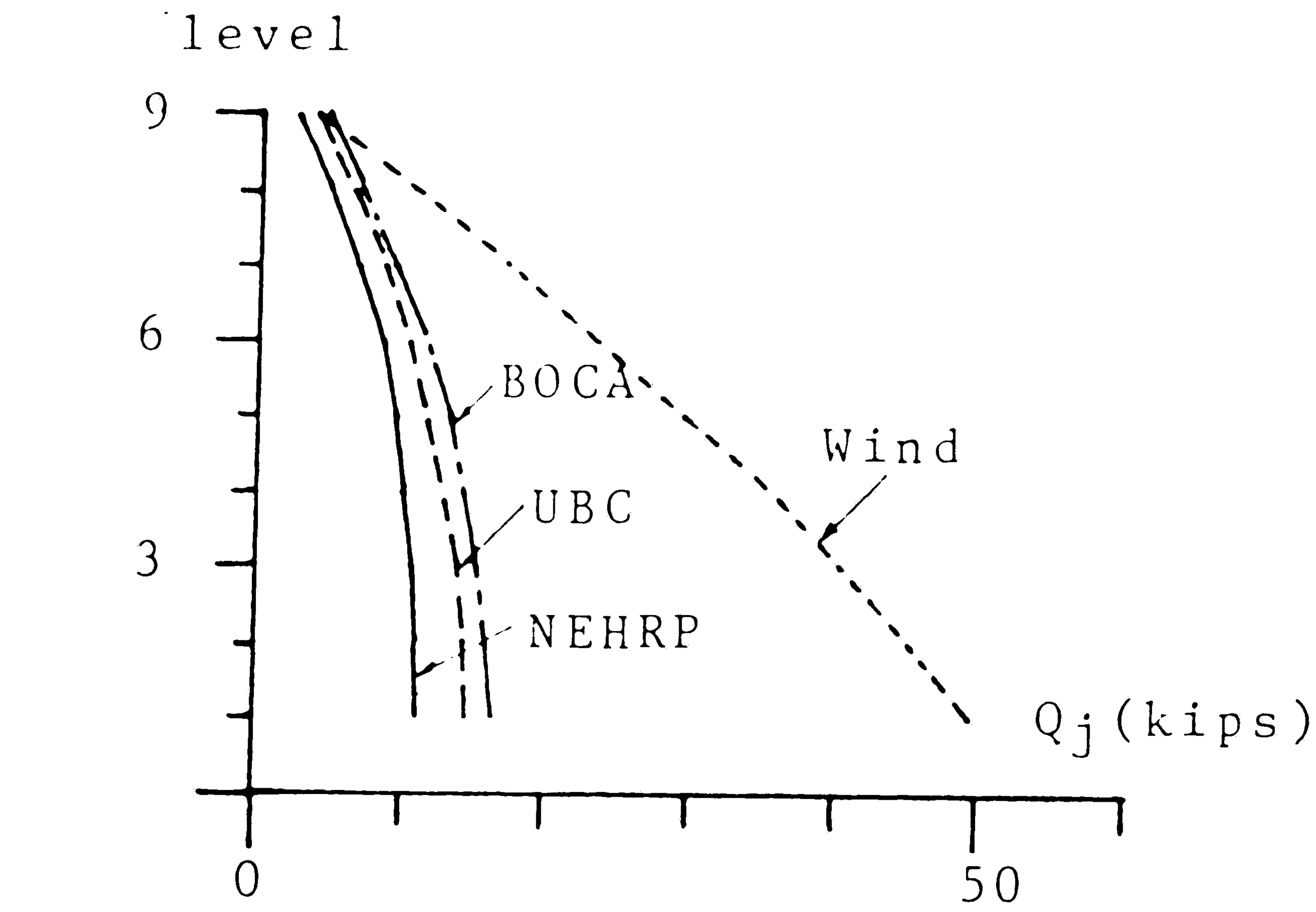


(b) 15-story SMRSF with 75psf, L/B=1.0

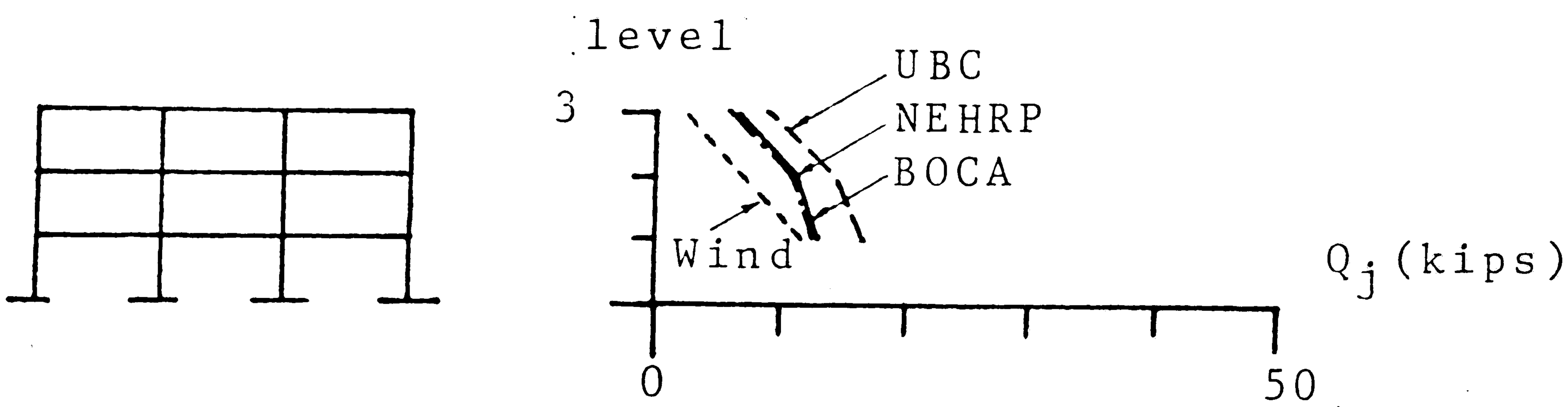
Fig.3-3 Design story shear at the working load level in New York



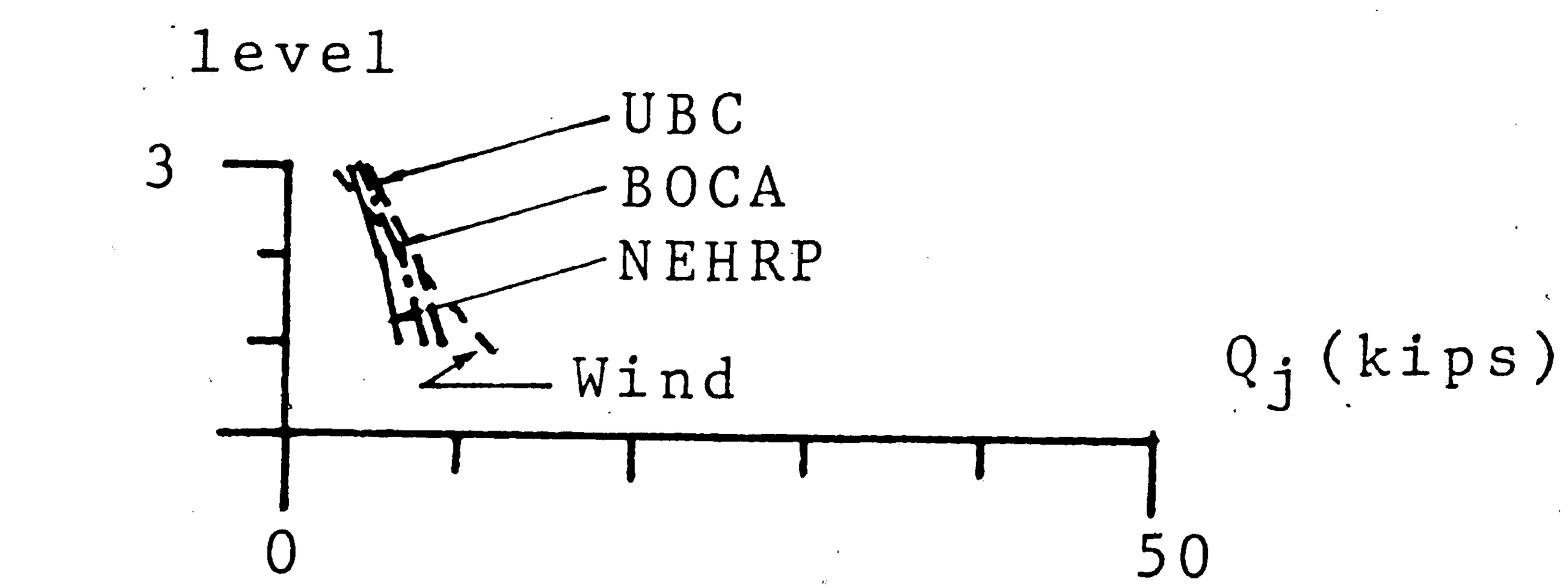
(c) 9-story OMRSF with 75psf, L/B=1.0



(d) 9-story SMRSF with 75psf, L/B=1.0

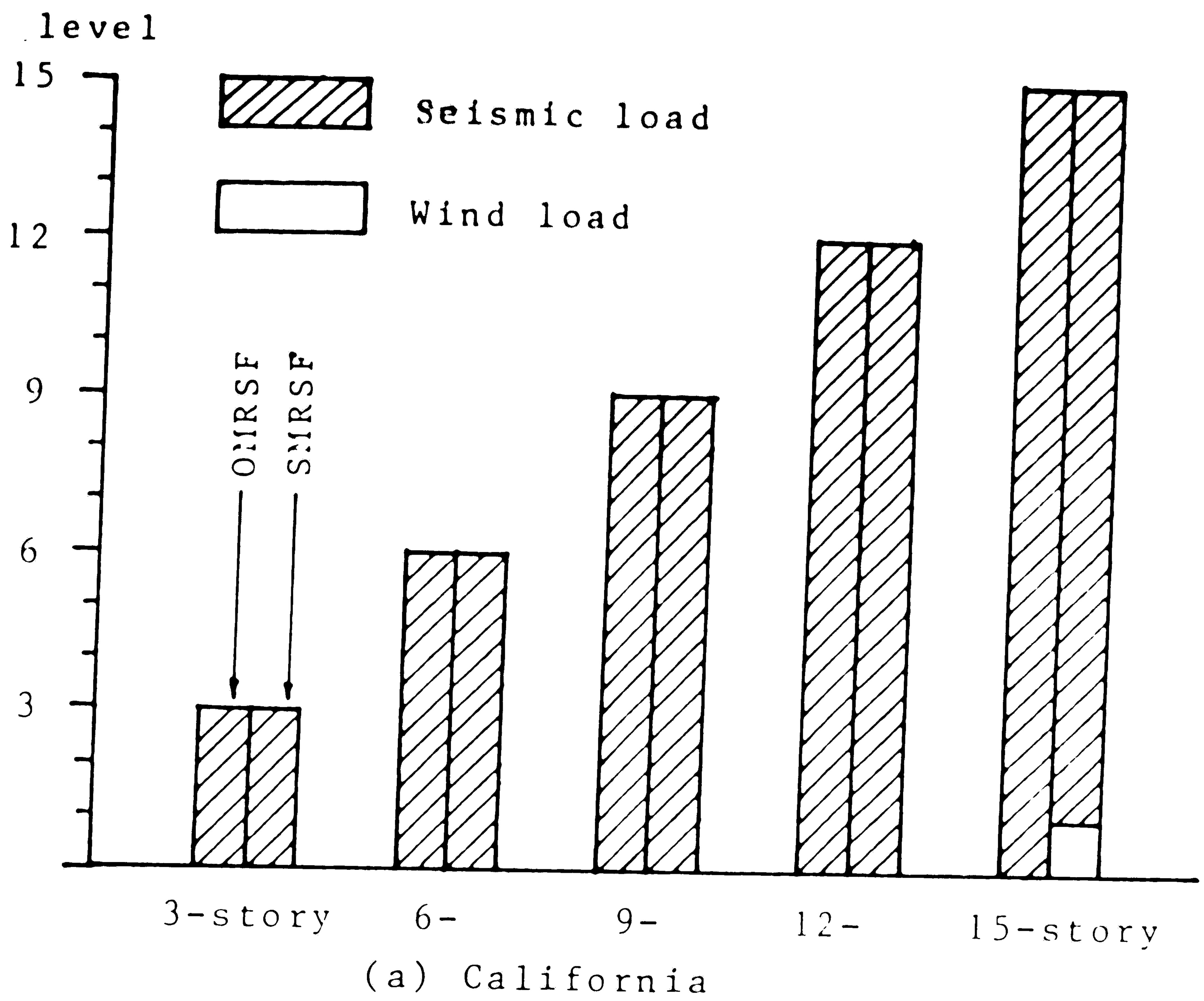


(e) 3-story OMRSF with 75psf, L/B=1.0



(f) 3-story SMRSF with 75psf, L/B=1.0

Fig.3-3 Design story shear at the working load level in New York



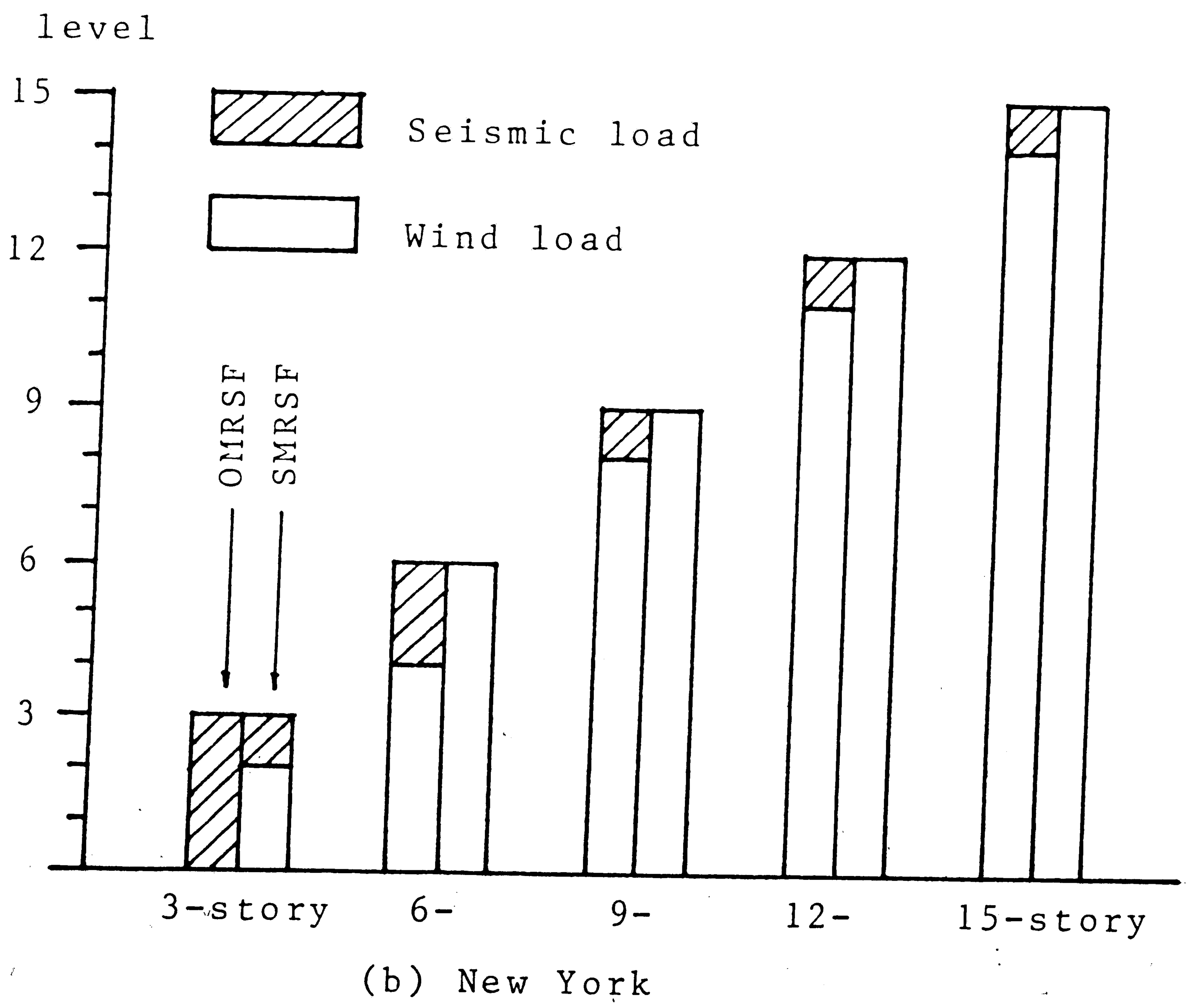
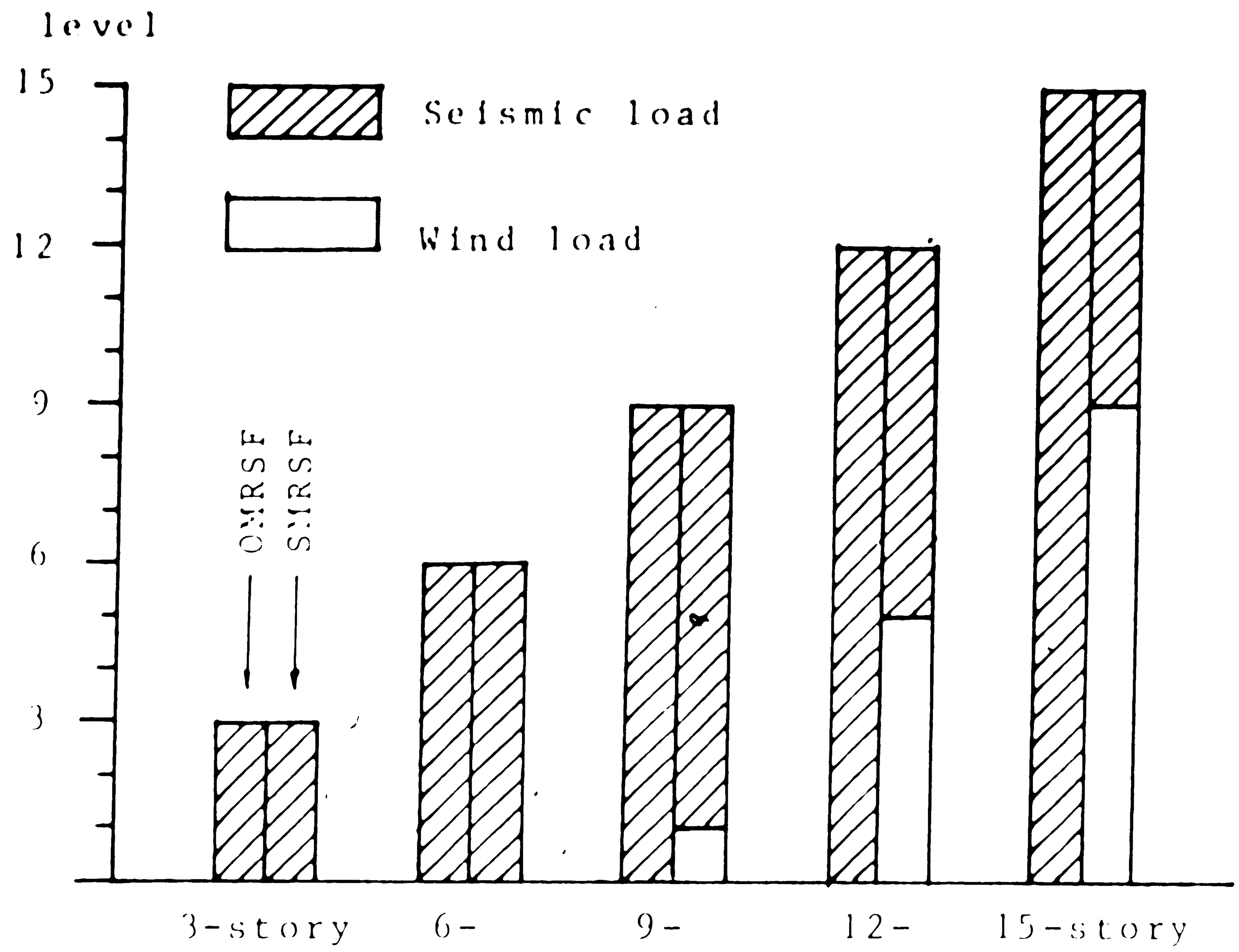
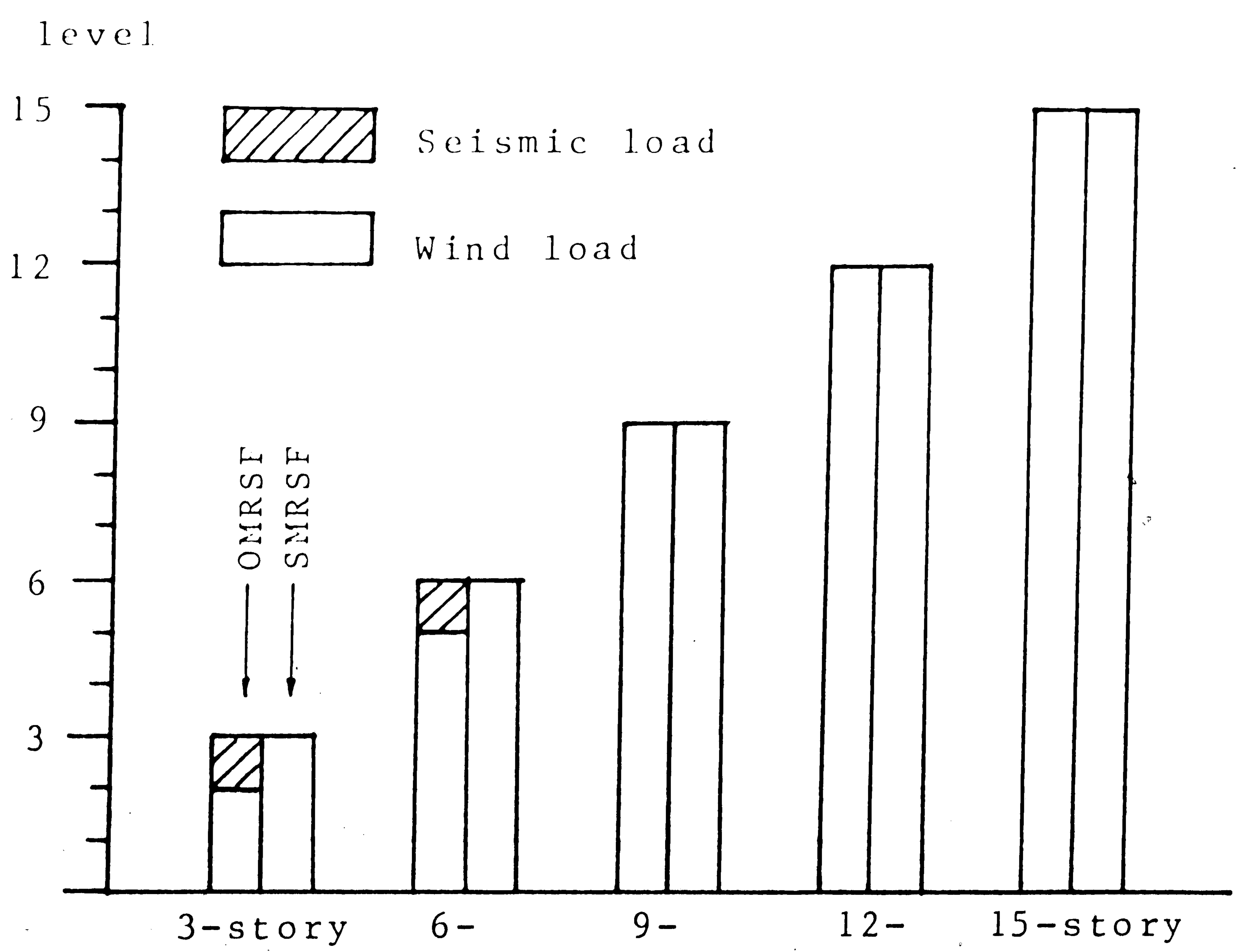


Fig.3-4. Governing horizontal loads (DL=75psf, L/B=1.0)

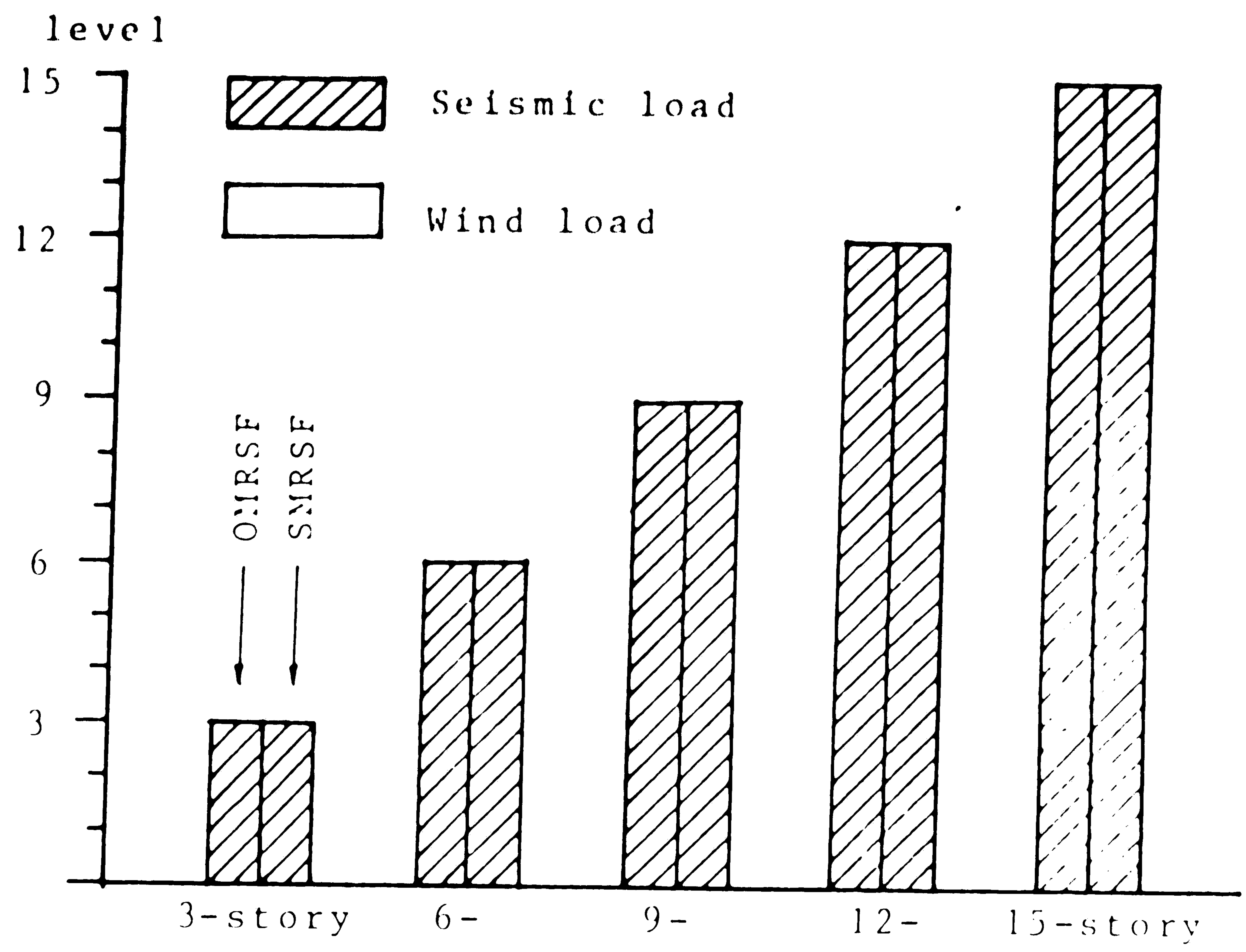


(a) California

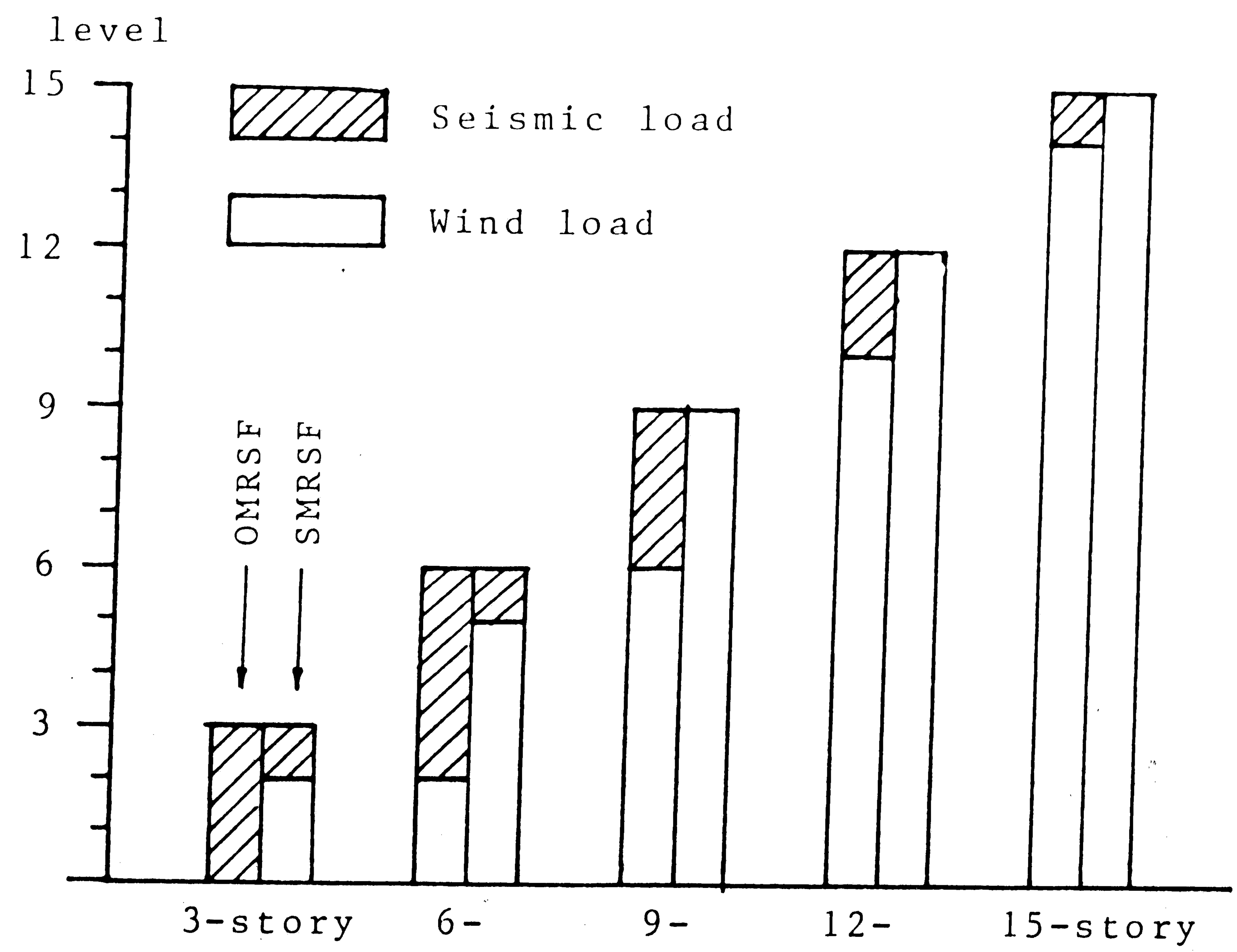


(b) New York

Fig.3-5 Governing horizontal loads (DL=50psf, L/B=1.0)



(a) California



(b) New York

Fig.3-6 Governing horizontal loads (DL=100psf, L/B=1.0)

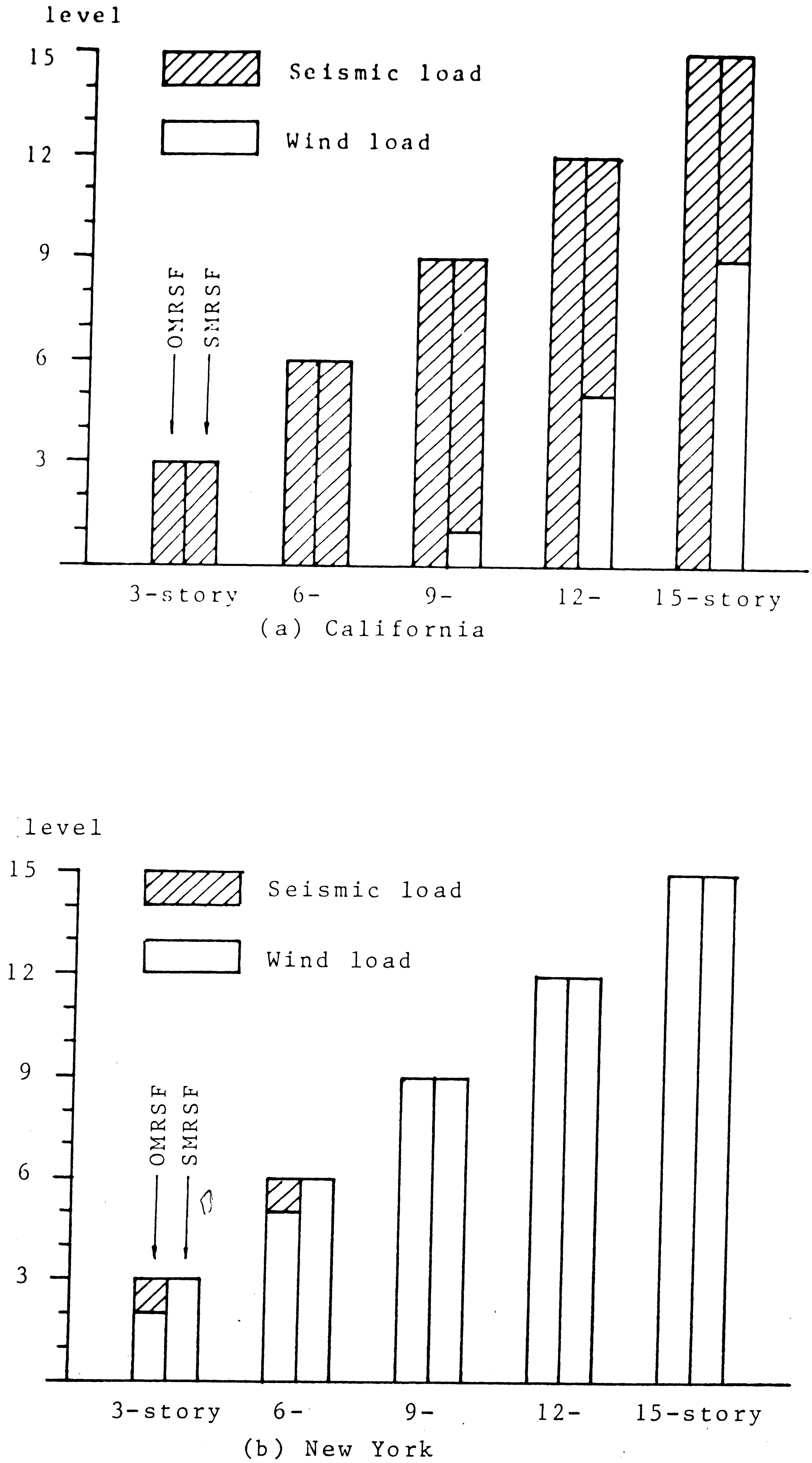
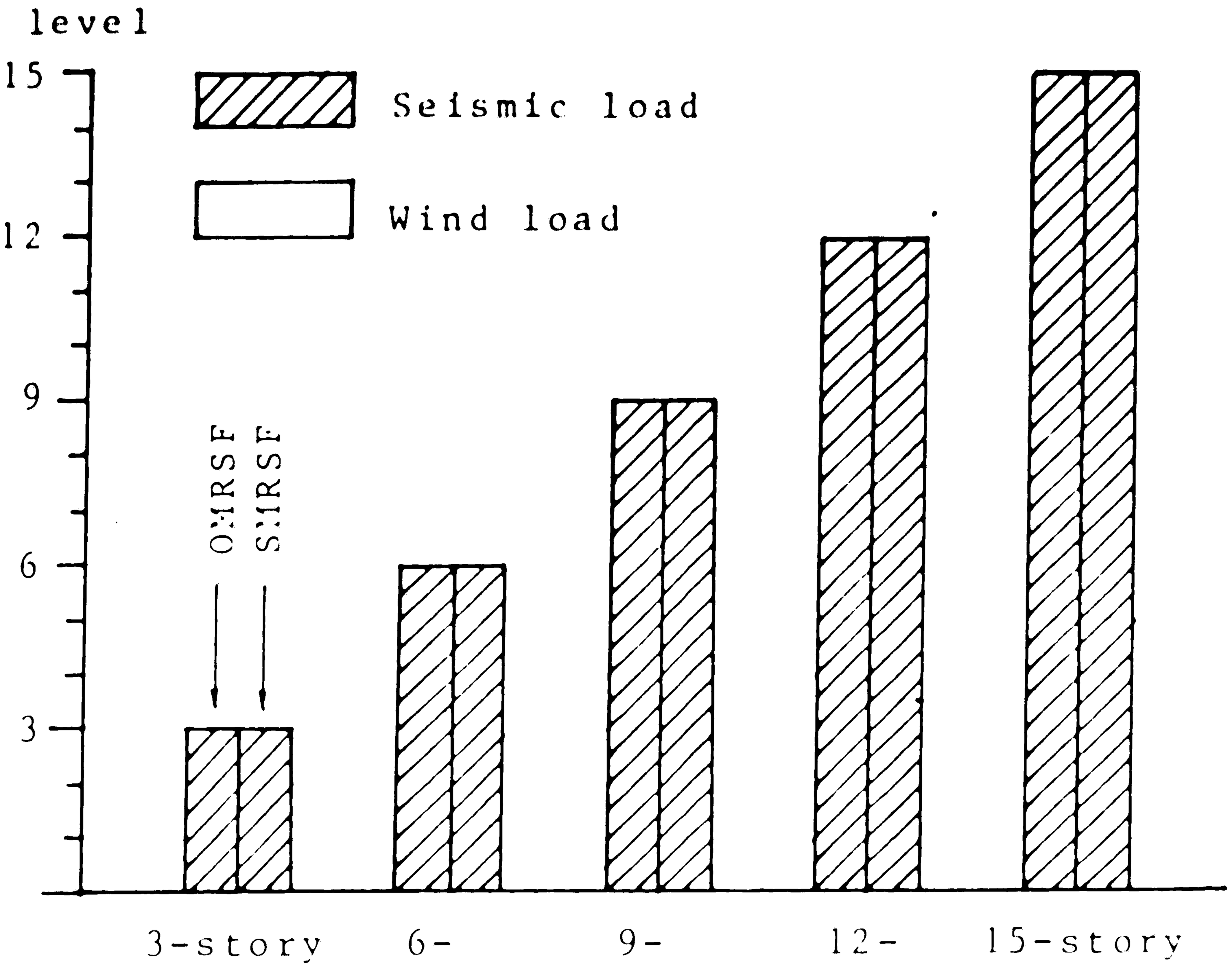
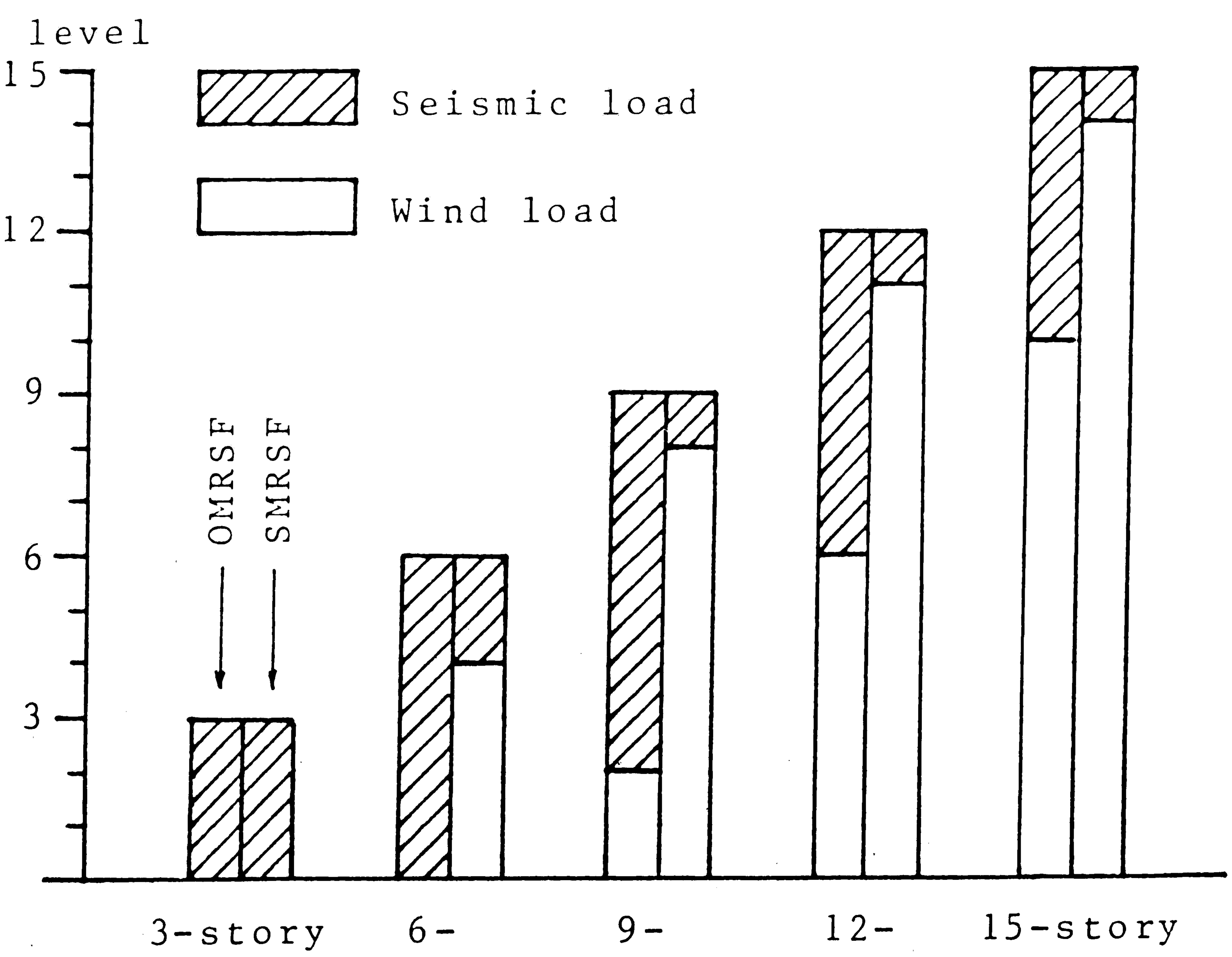


Fig.3-7 Governing horizontal loads (DL=75psf, L/B=0.4)



(a) California



(b) New York

Fig.3-8 Governing horizontal loads (DL=75psf, L/B=2.5)

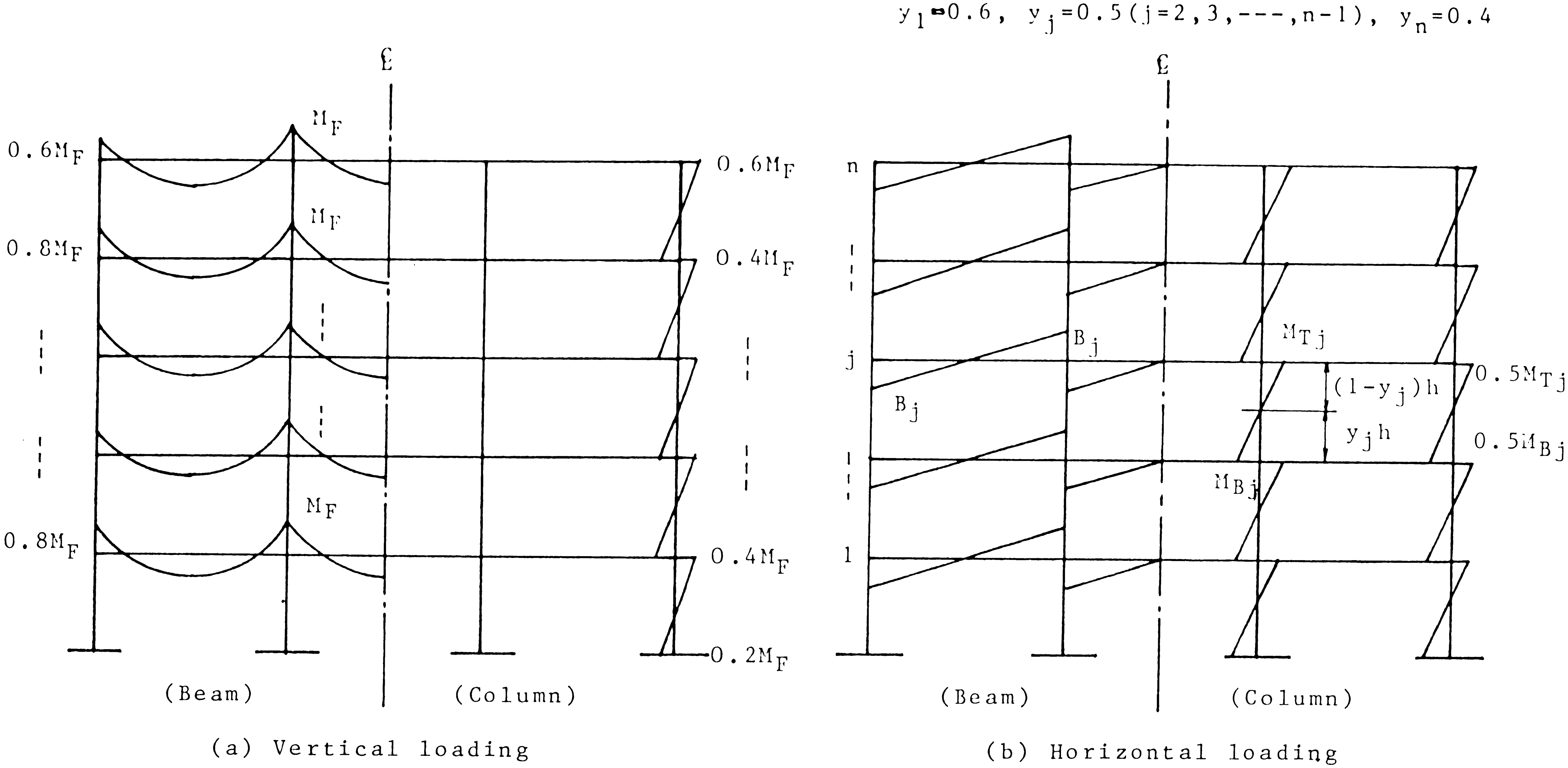
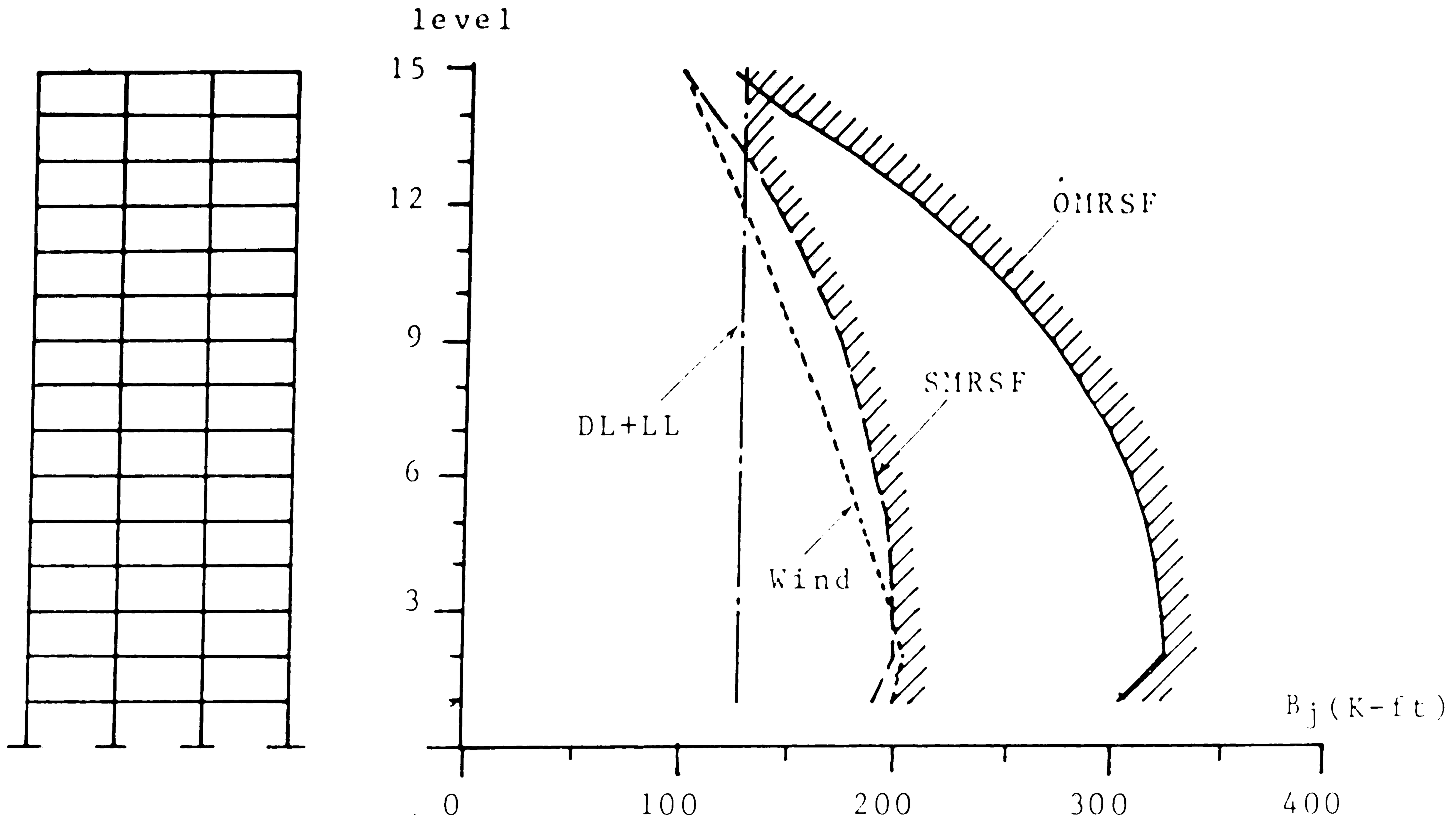
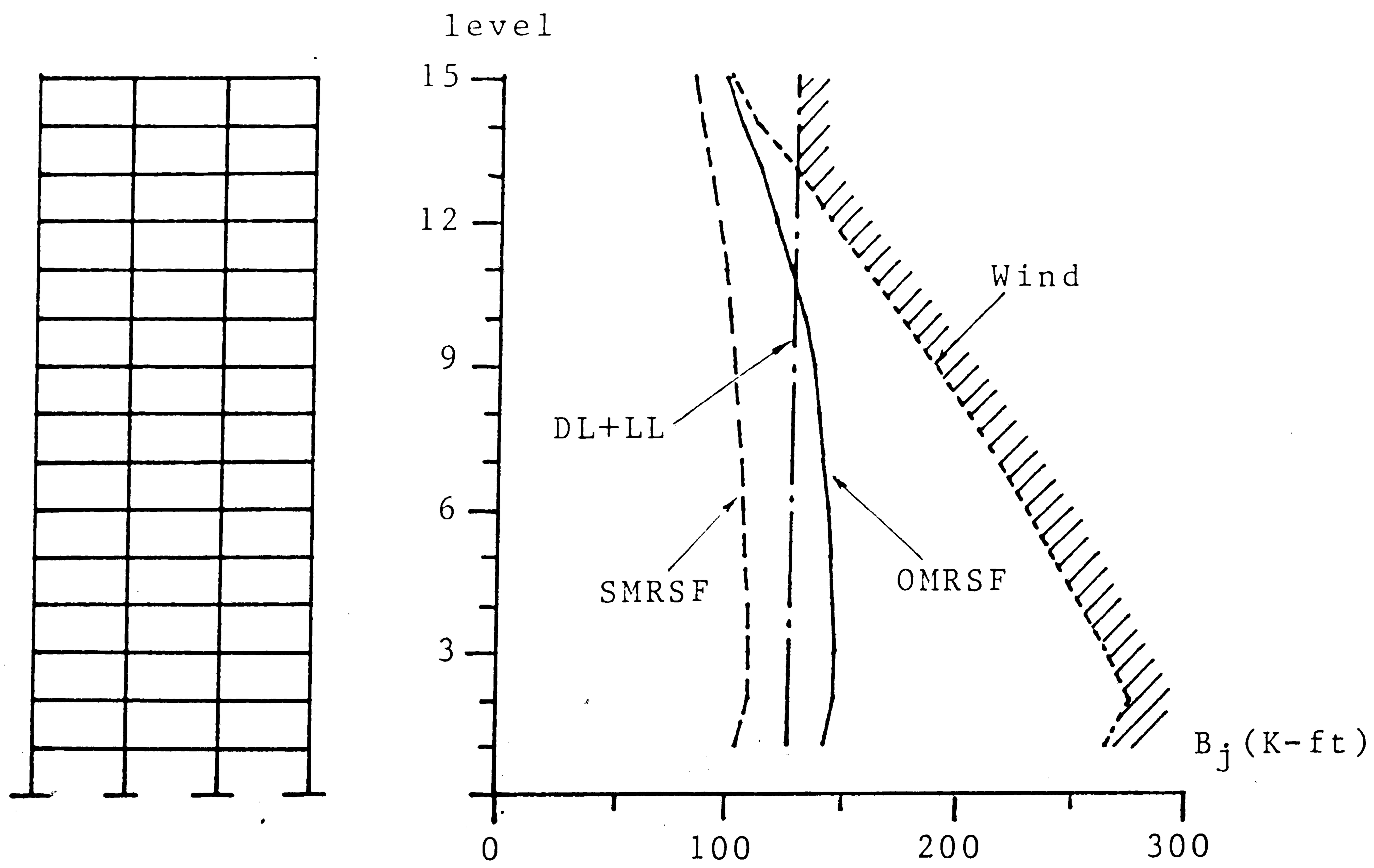


Fig.3-9 Moment diagrams of moment-resisting frames

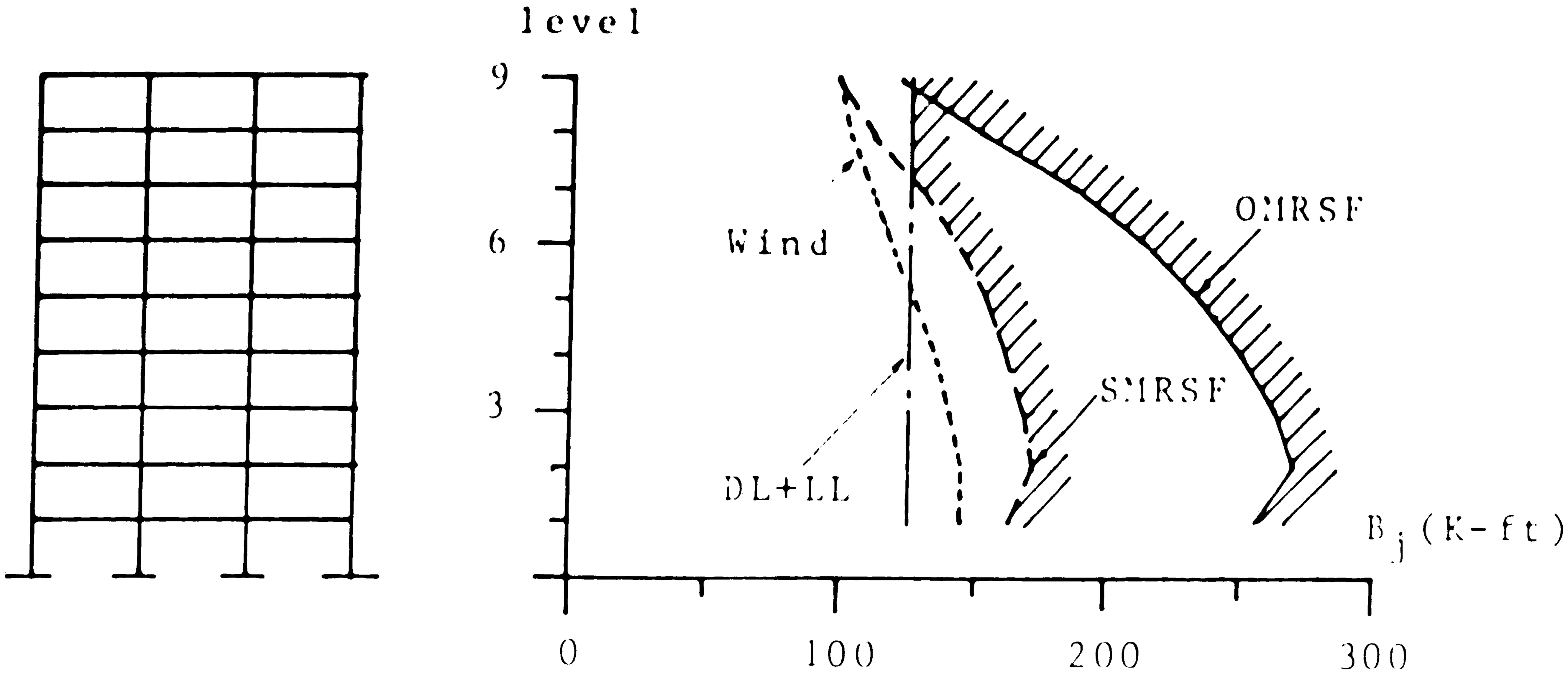


(a) 15-story buildings with 75psf, L/B=1.0 in CA

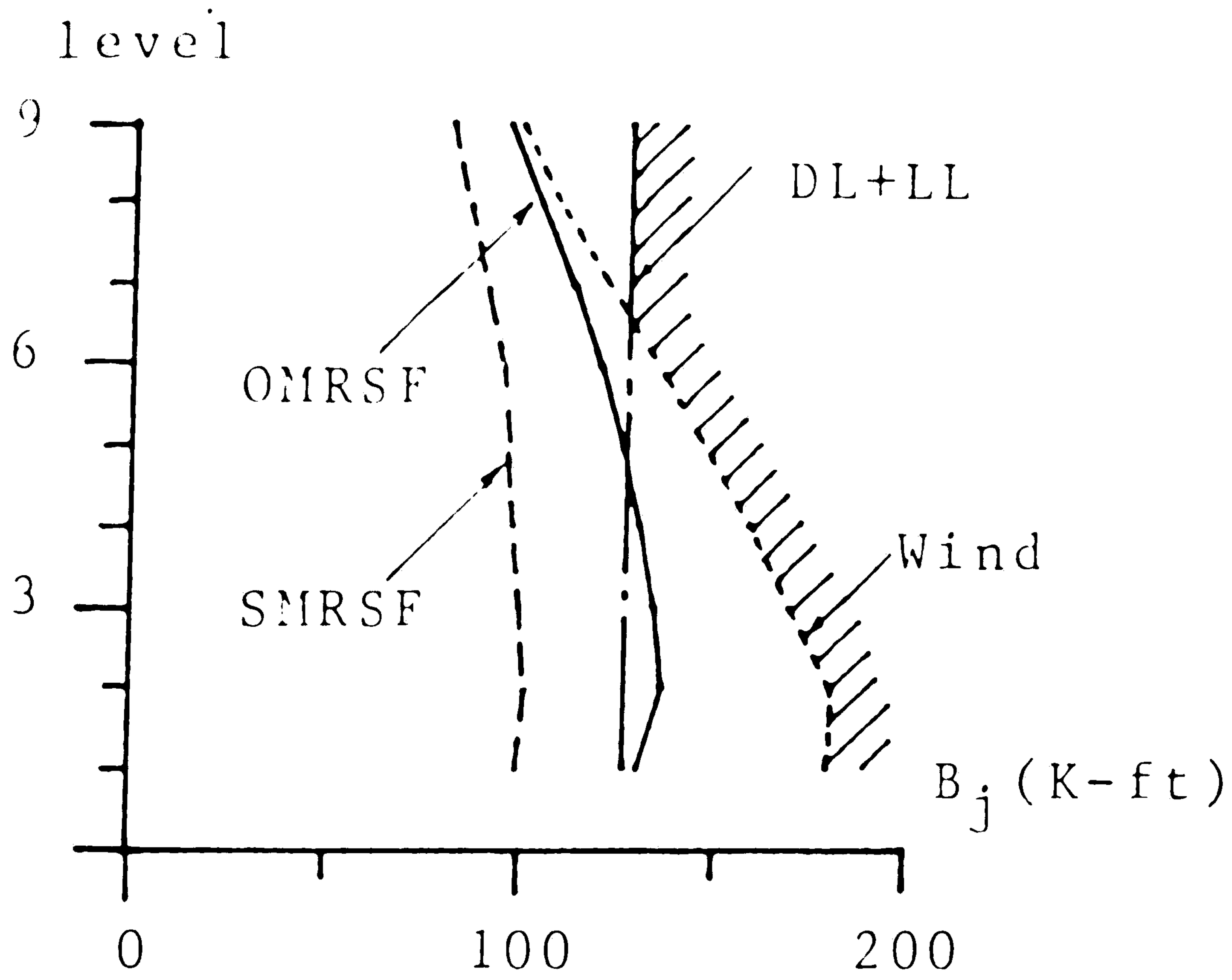


(b) 15-story buildings with 75psf, L/B=1.0 in NY

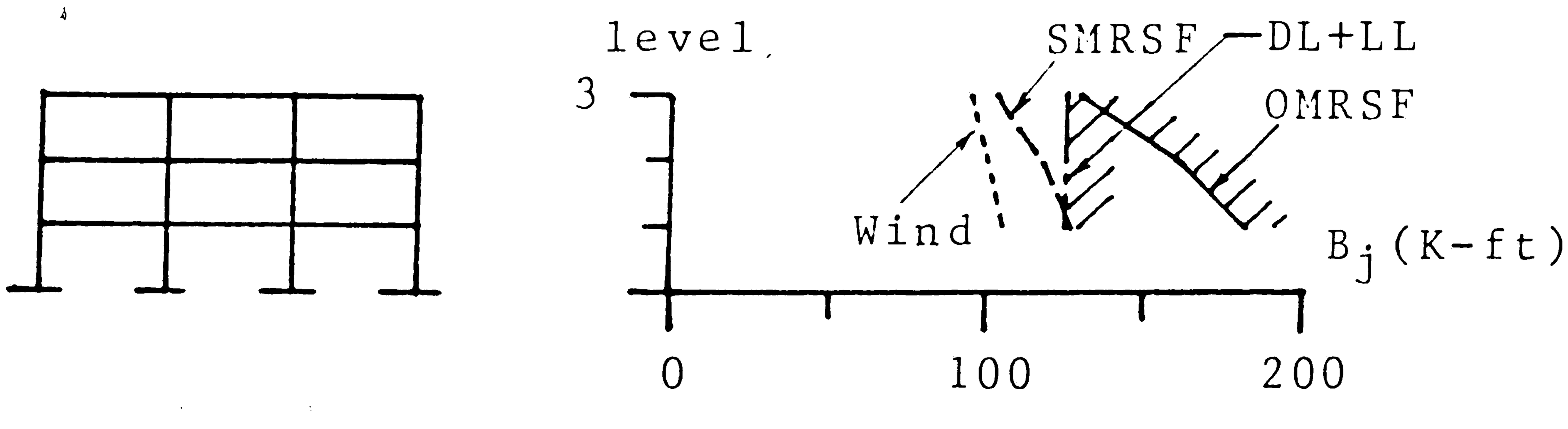
Fig.3-10 Beam design moment for buildings with DL=75psf, L/B=1.0



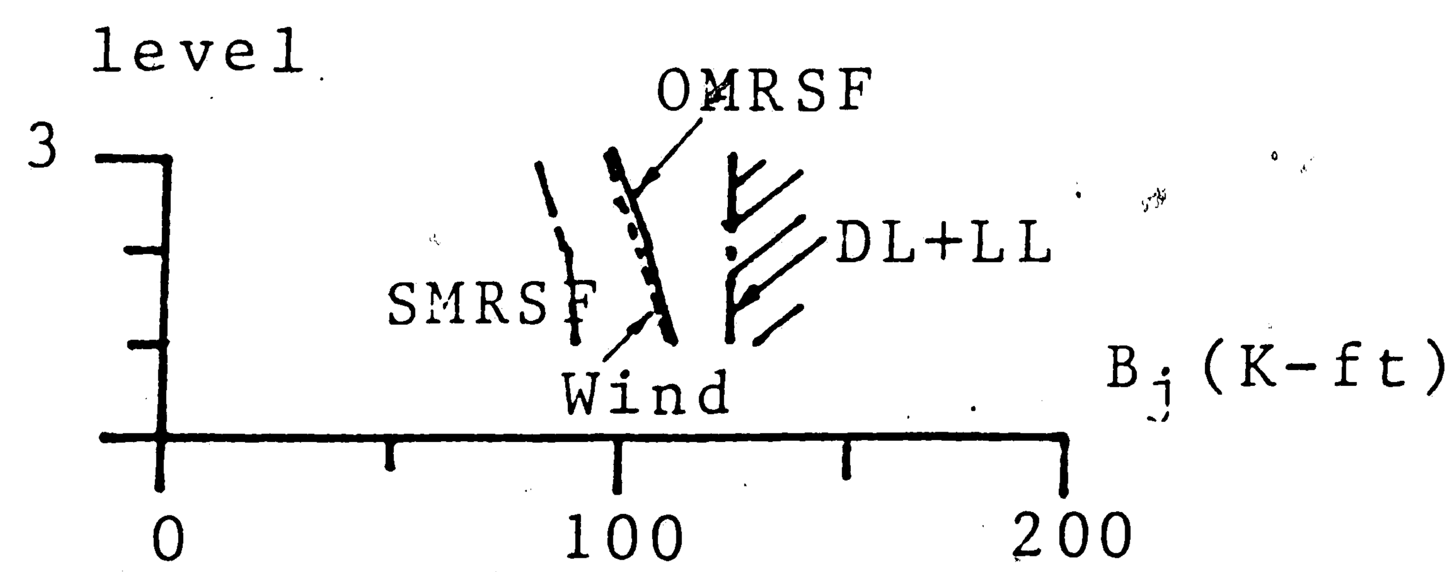
(c) 9-story buildings with 75psf, L/B=1.0 in CA



(d) 9-story buildings with 75psf, L/B=1.0 in NY

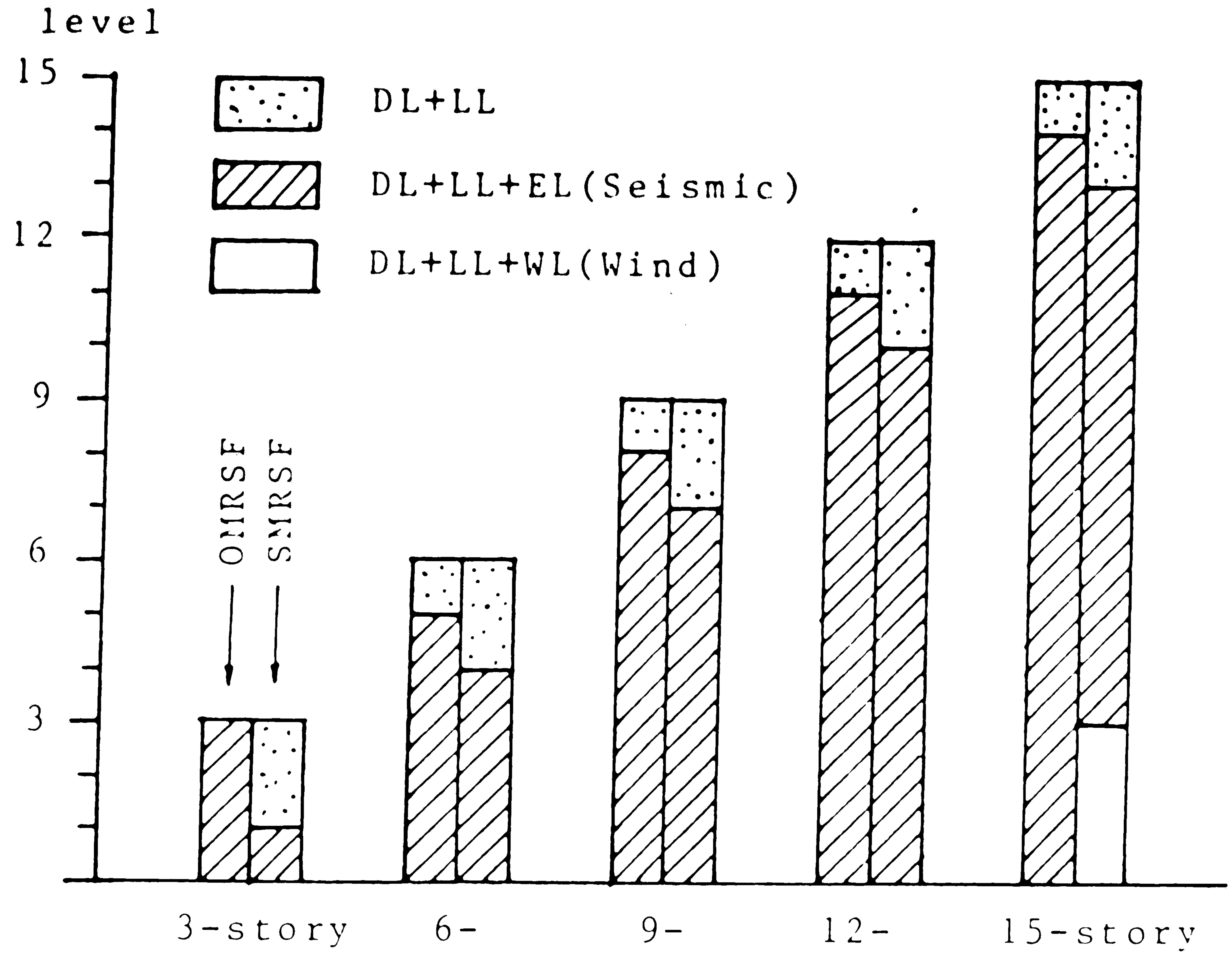


(e) 3-story buildings with 75psf, L/B=1.0 in CA

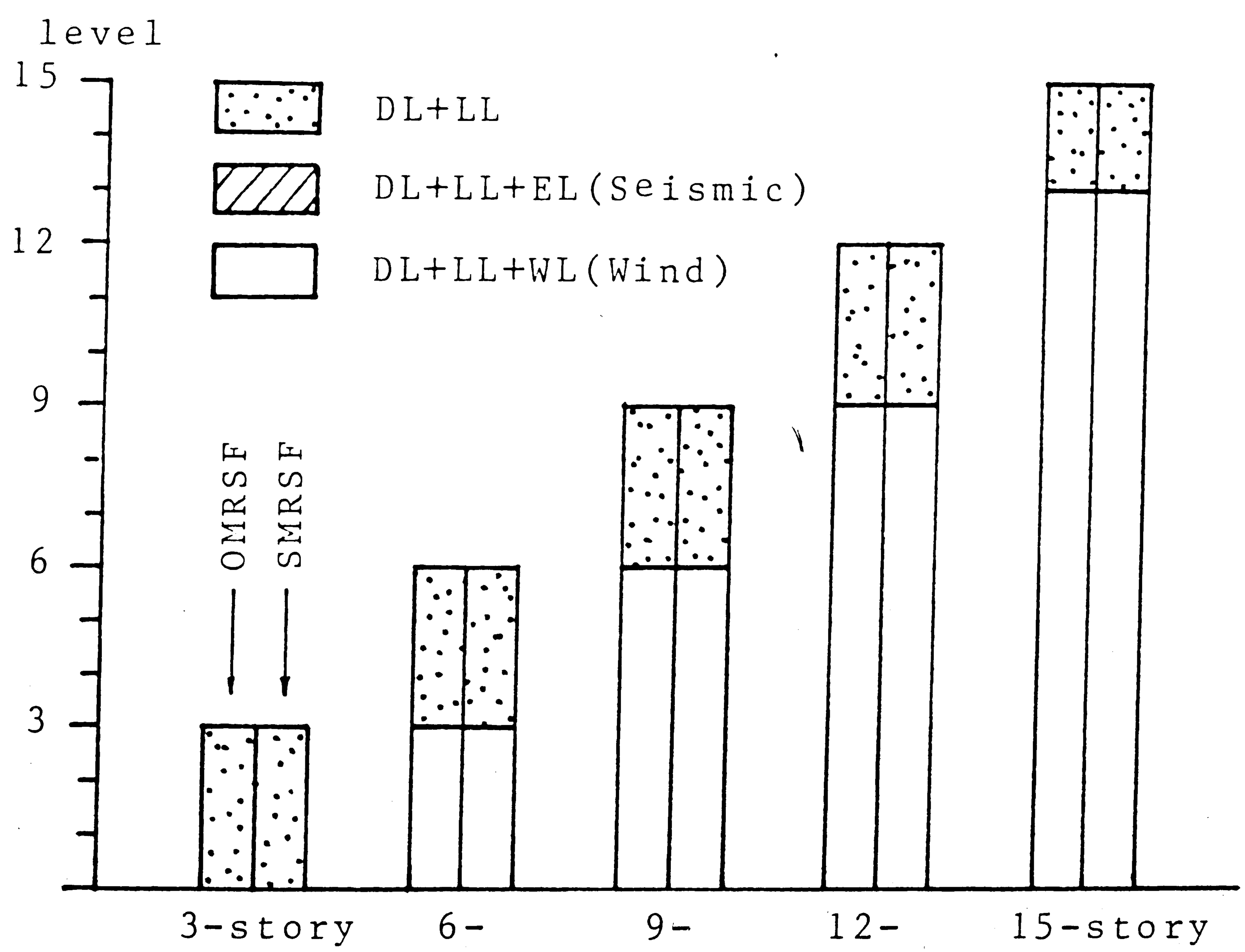


(f) 3-story buildings with 75psf, L/B=1.0 in NY

Fig.3-10 Beam design moment for buildings with DL=75psf, L/B=1.0

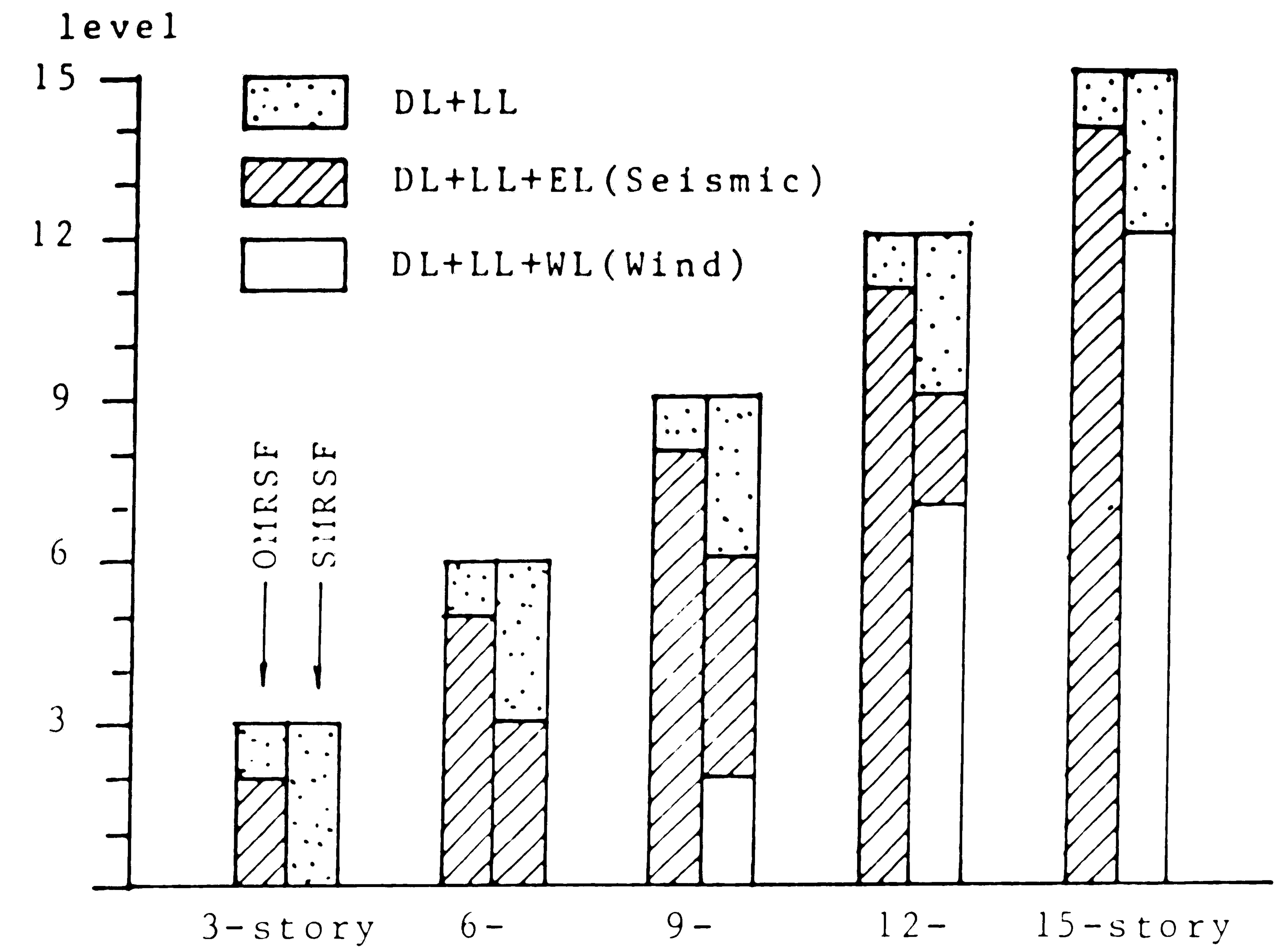


(a) California

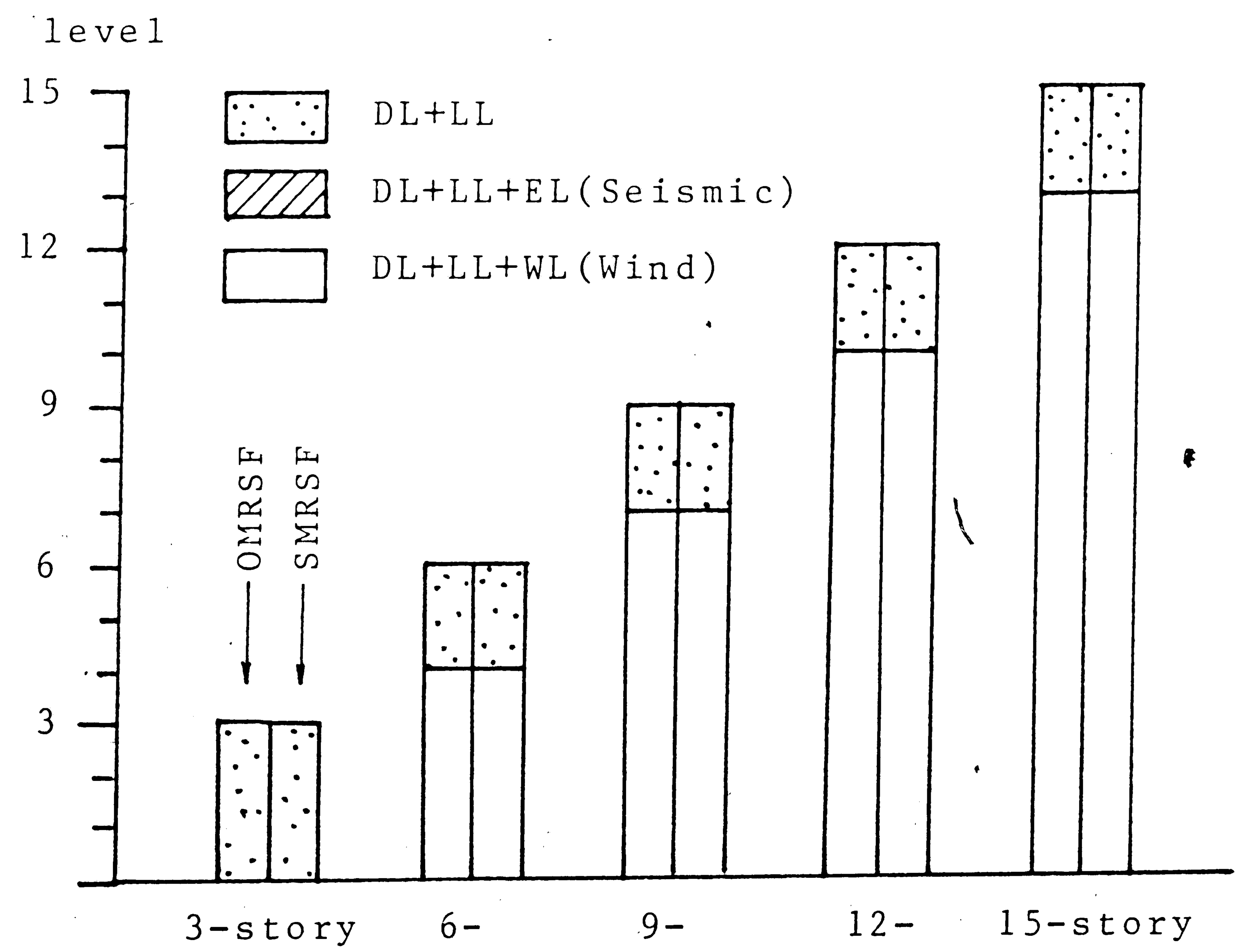


(b) New York

Fig.3-11 Controlling criteria for beam design (DL=75psf, L/B=1.0)

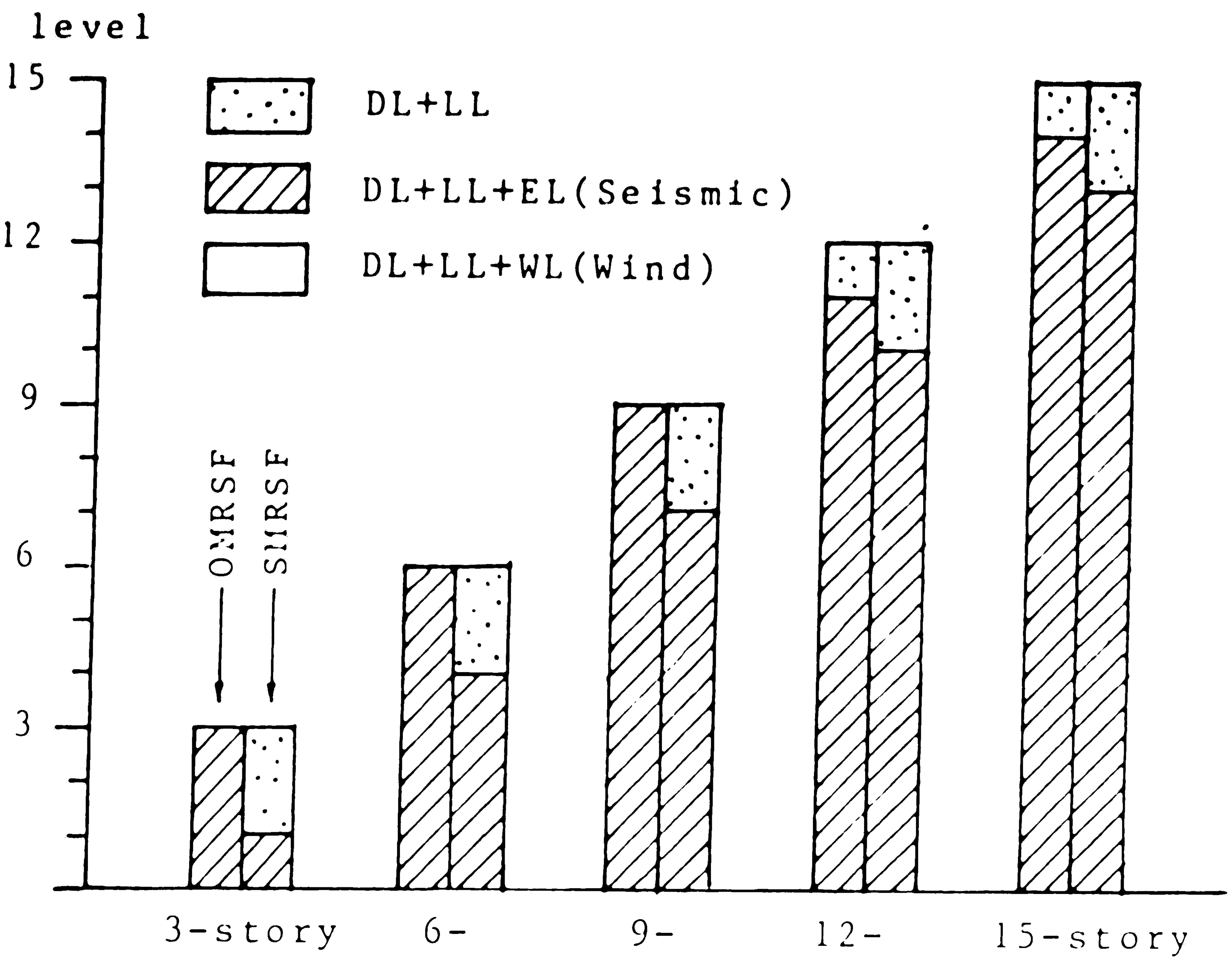


(a) California

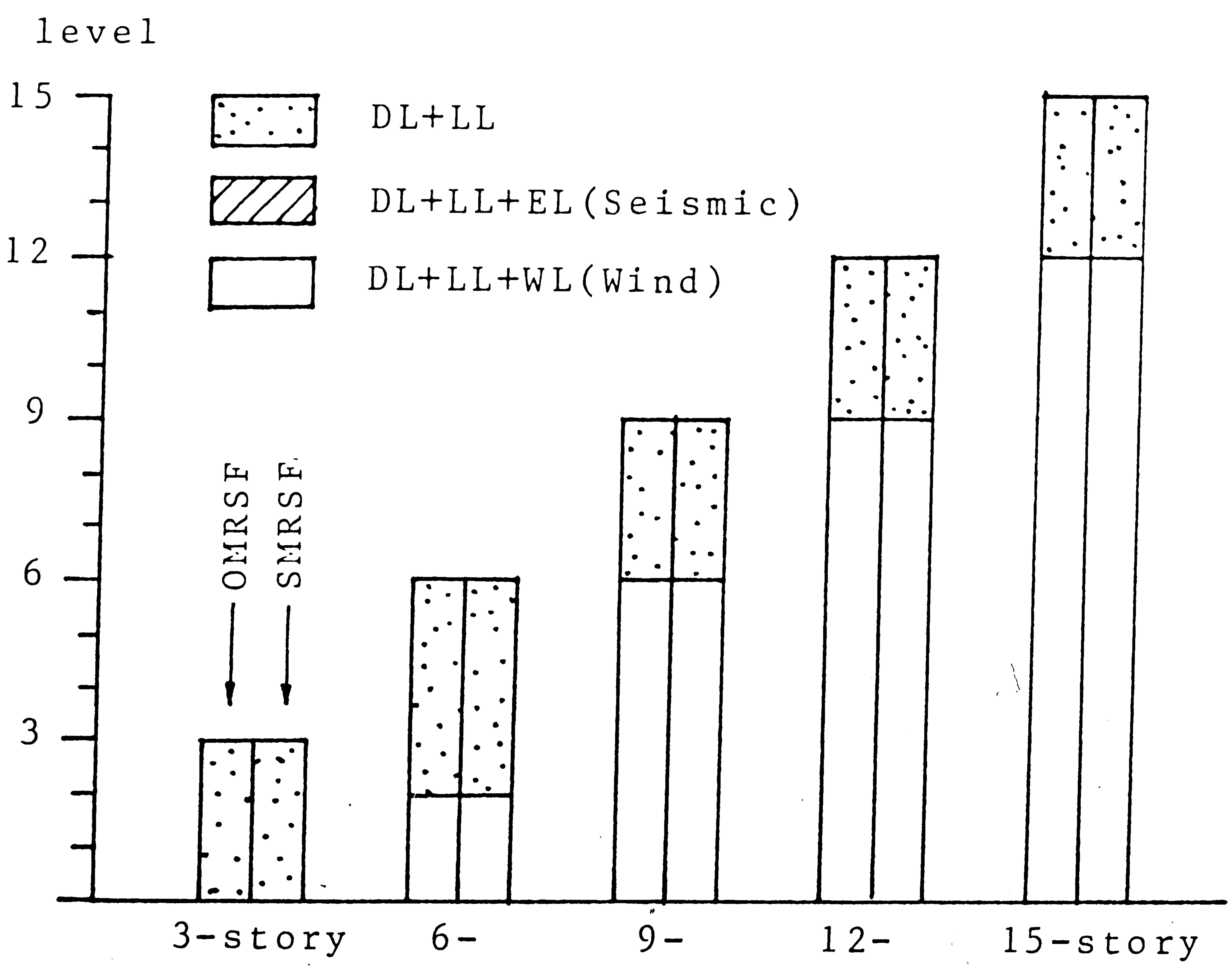


(b) New York

Fig.3-12 Controlling criteria for beam design (DL=50psf, L/B=1.0)

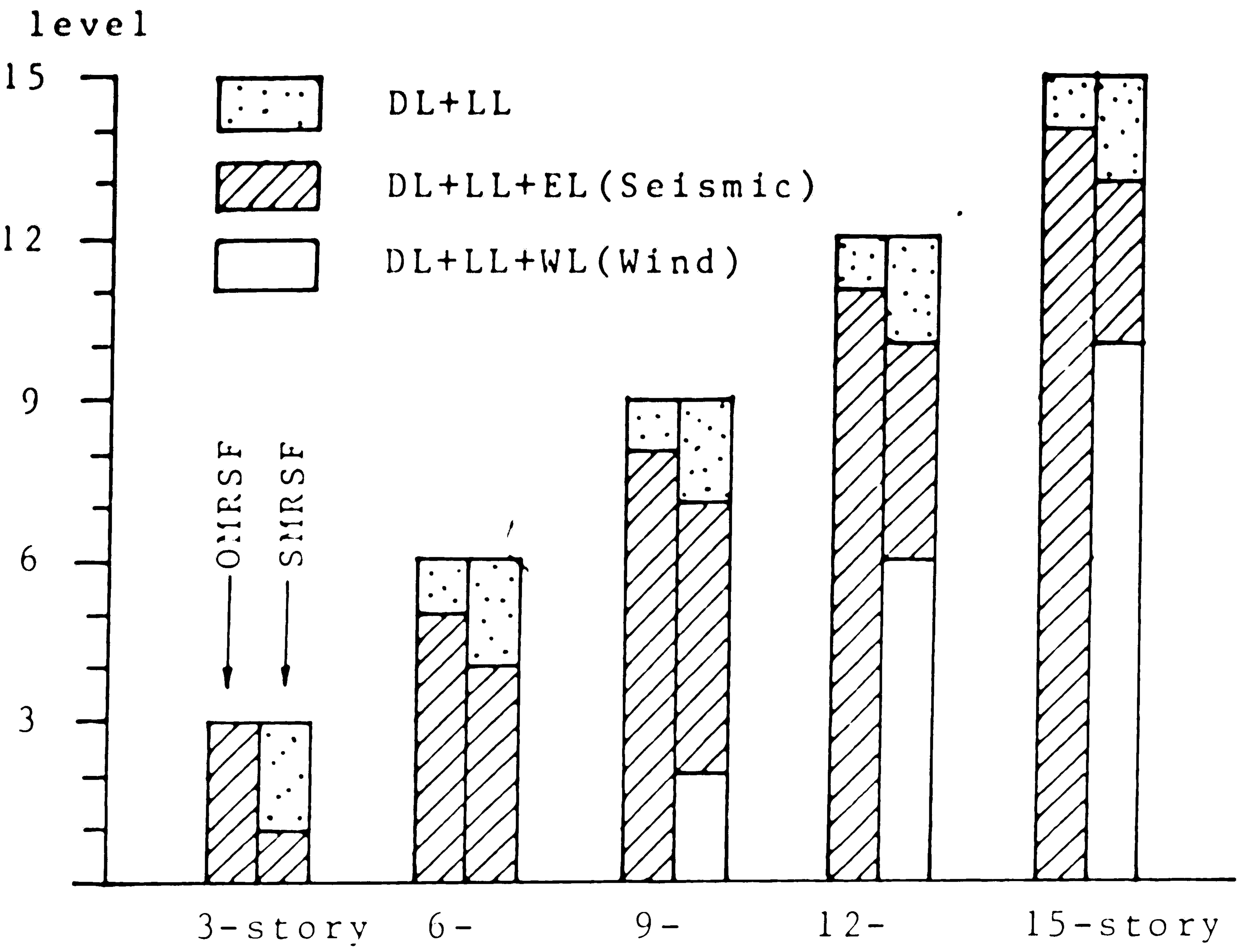


(a) California

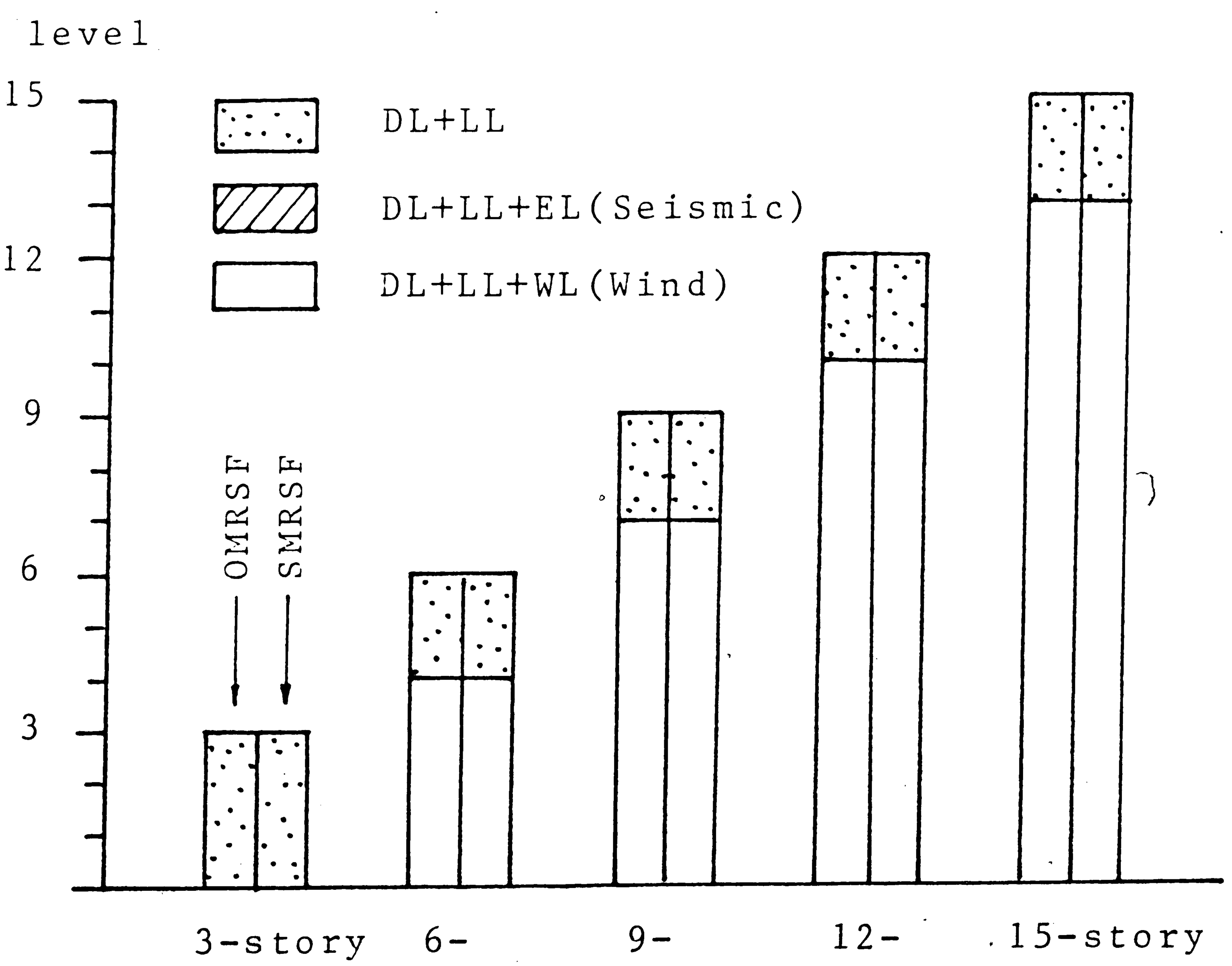


(b) New York

Fig.3-13 Controlling criteria for beam design (DL=100psf, L/B=1.0)

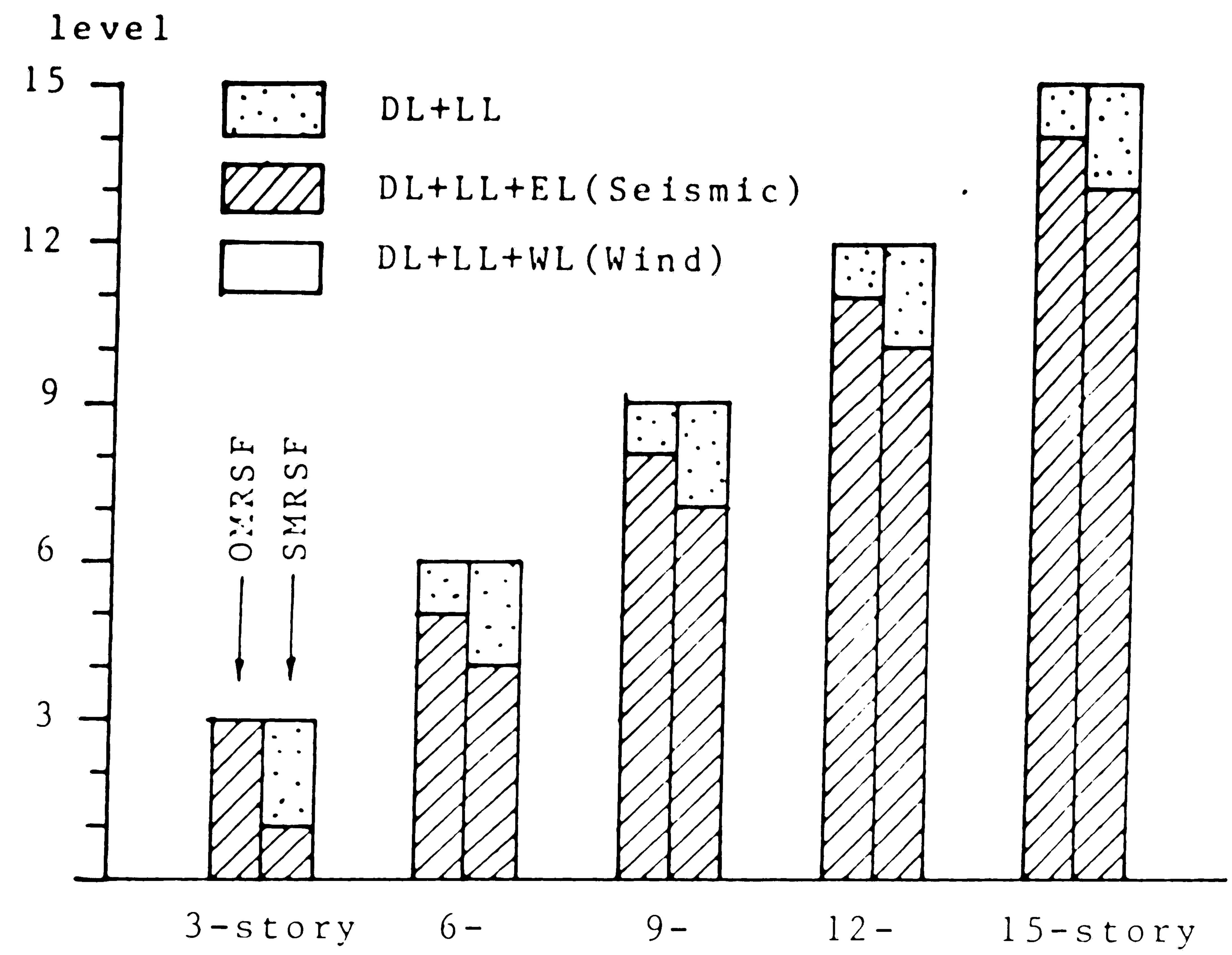


(a) California

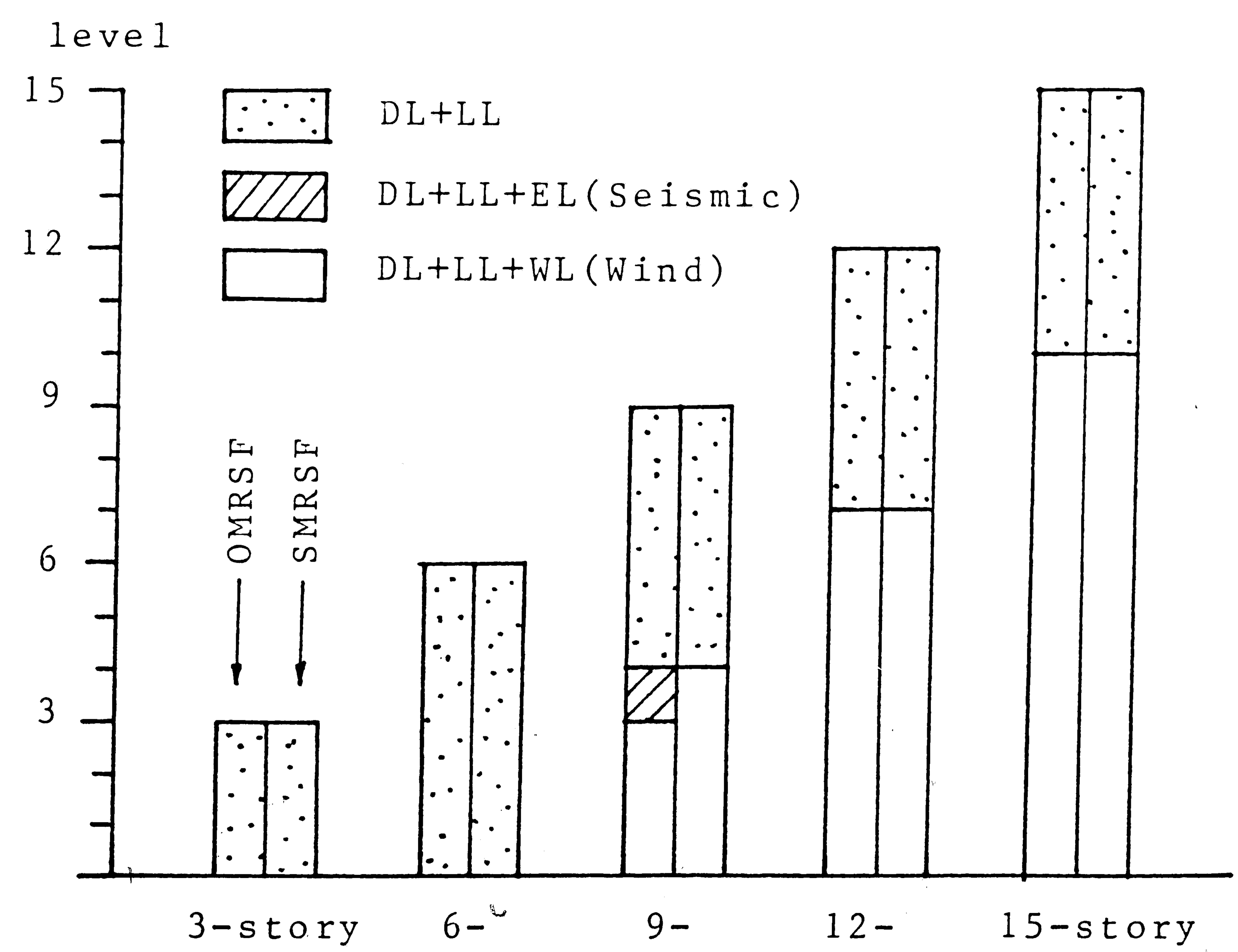


(b) New York

Fig.3-14 Controlling criteria for beam design (DL=75psf, L/B=0.4)



(a) California



(b) New York

Fig.3-15 Controlling criteria for beam design (DL=75psf, L/B=2.5)

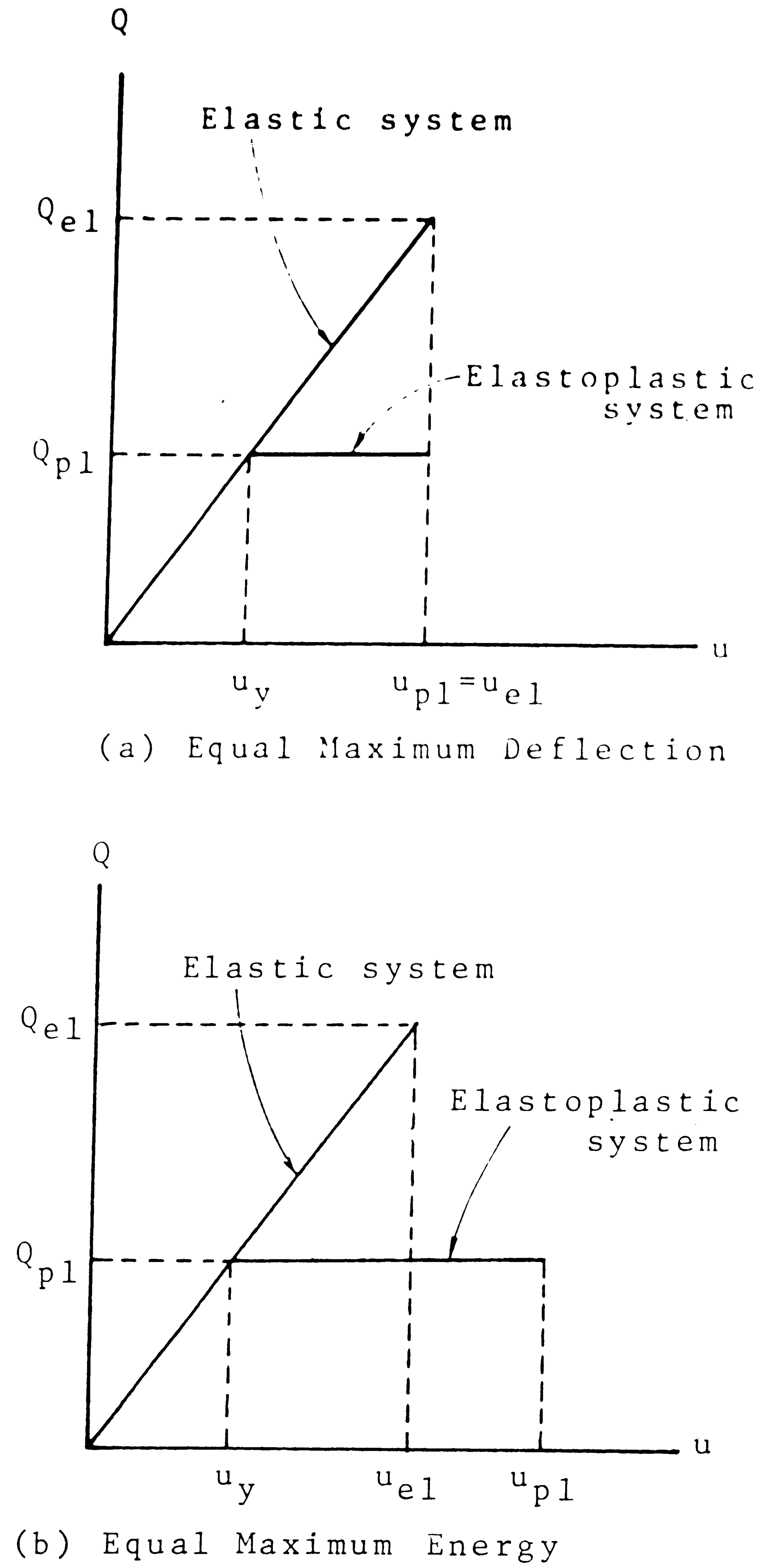


Fig.4-1 Elastoplastic behavior assumptions

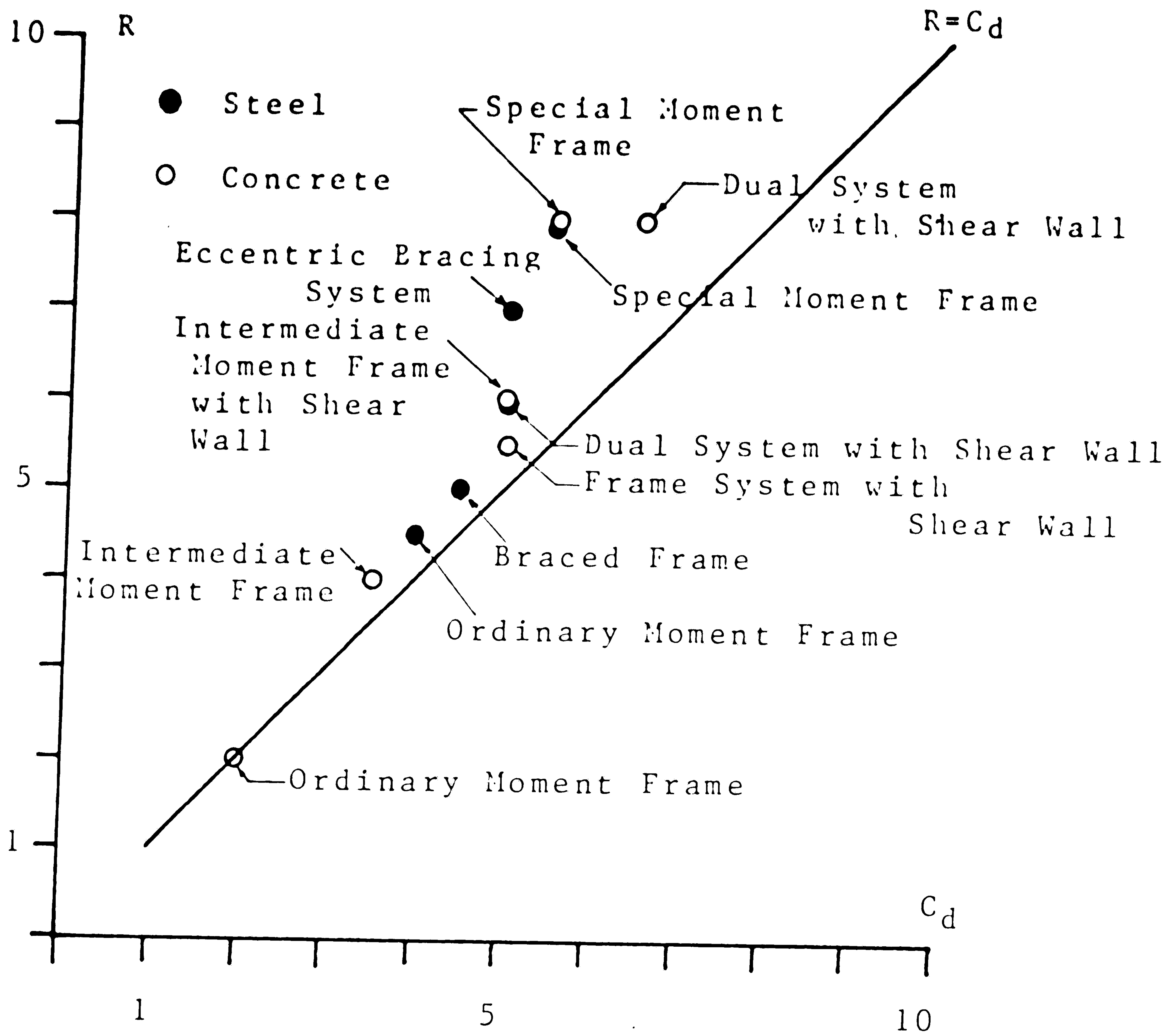


Fig.4-2 R and C_d factors in the NEHRP provisions

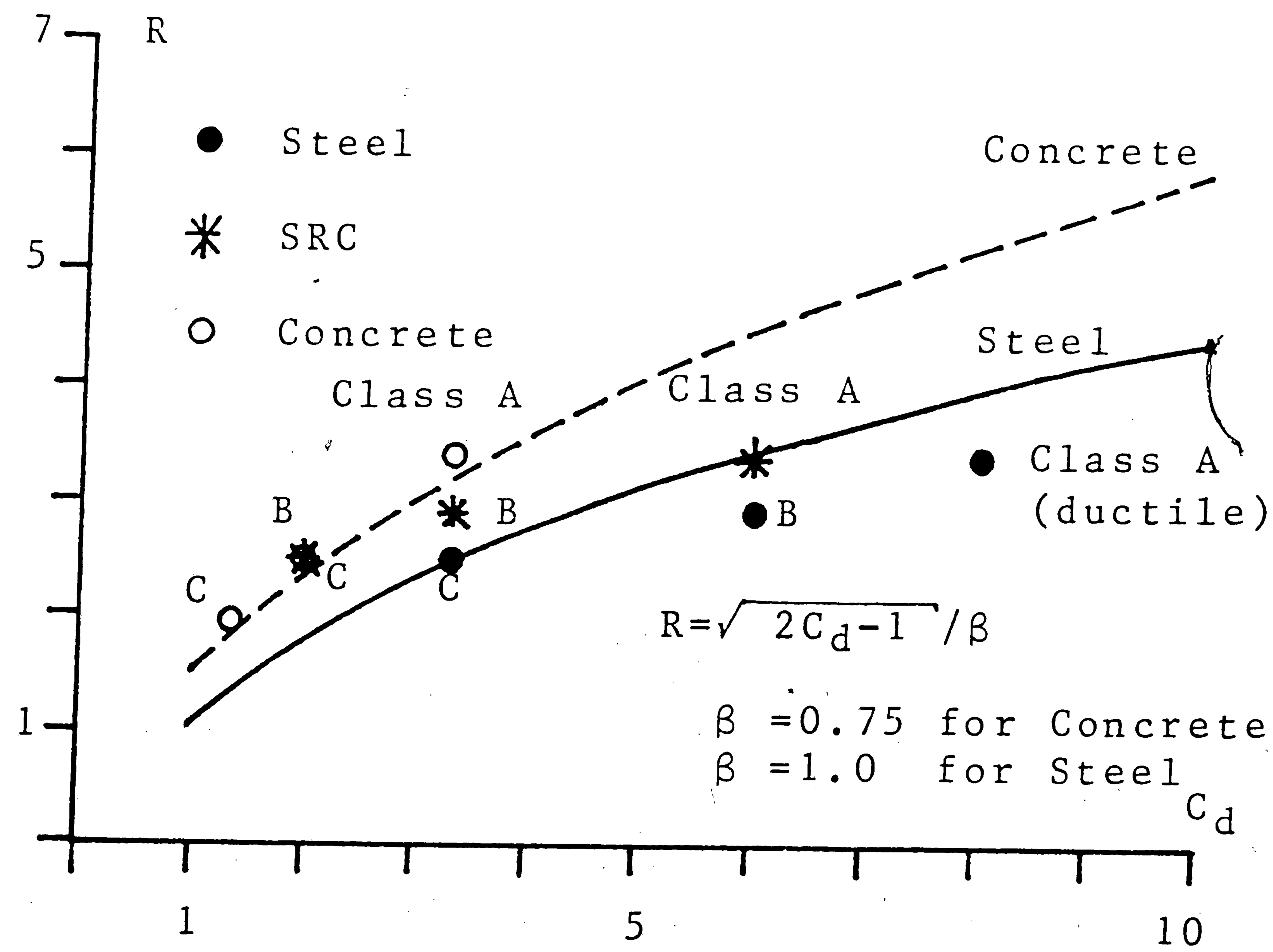


Fig.4-3 R and C_d factors in the Japanese seismic code

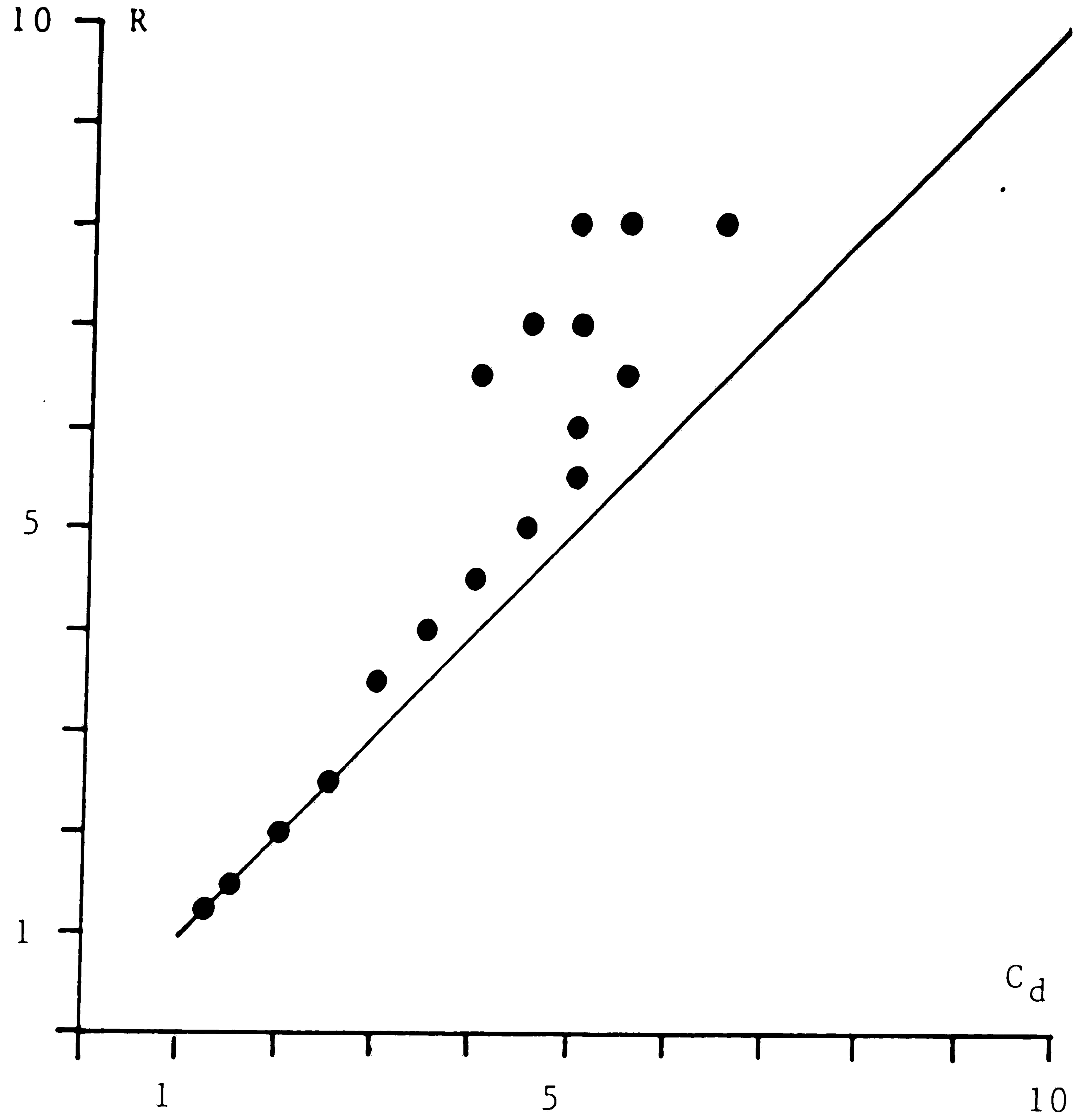
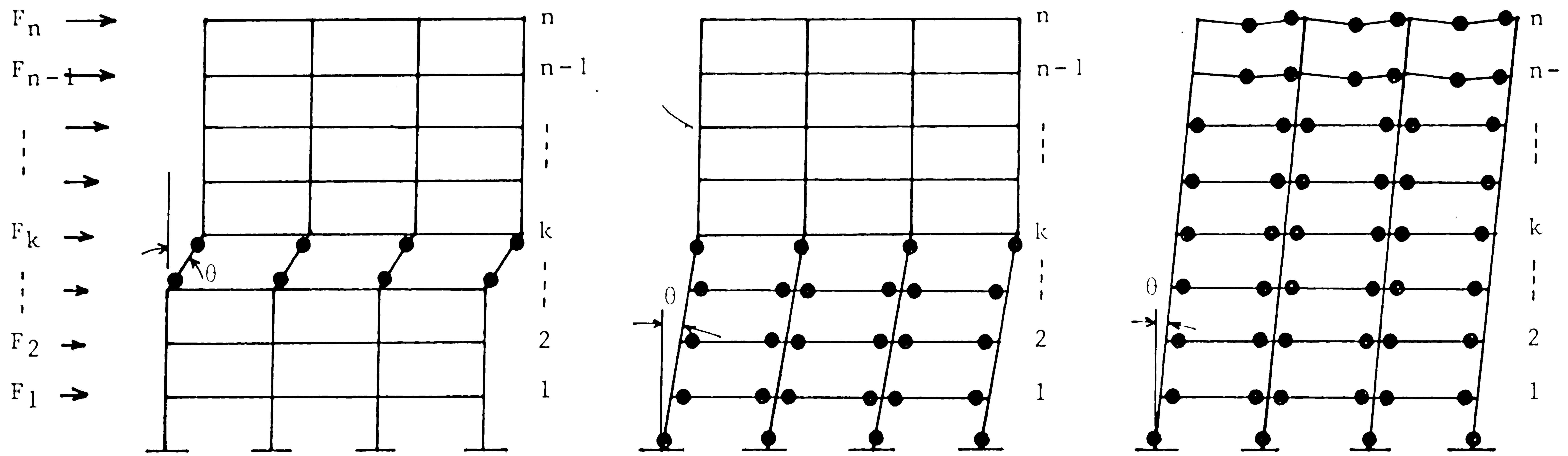


Fig.4-4 Variety of R and C_d factors in the NEHRP provisions



(a) Column type sway mechanism

(b) Combined mechanism

(c) Beam type sway mechanism

Fig.4-5 Analyzed collapse mechanisms

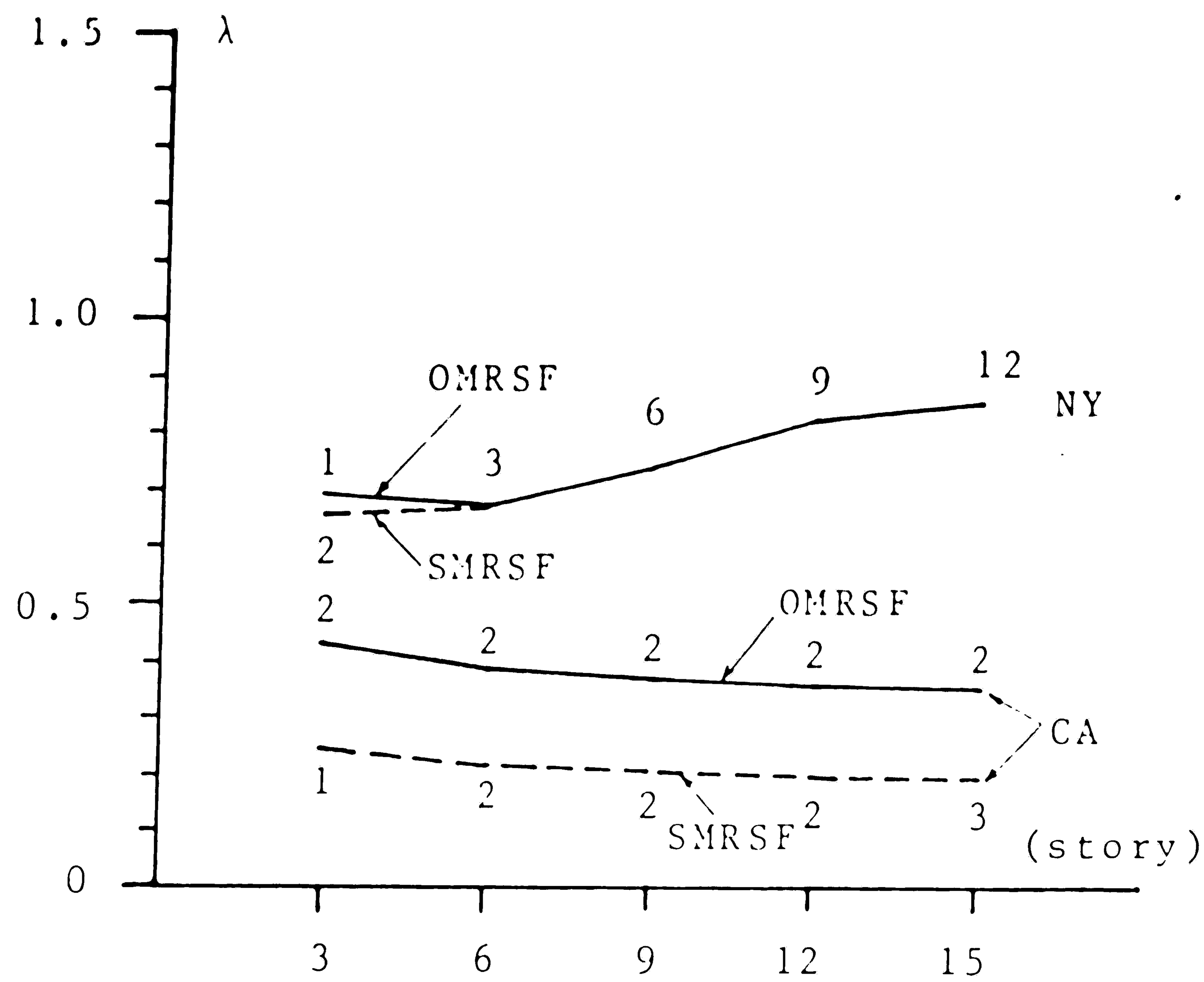


Fig.4-6 Ultimate strength of buildings (DL=75psf, L/B=1.0)

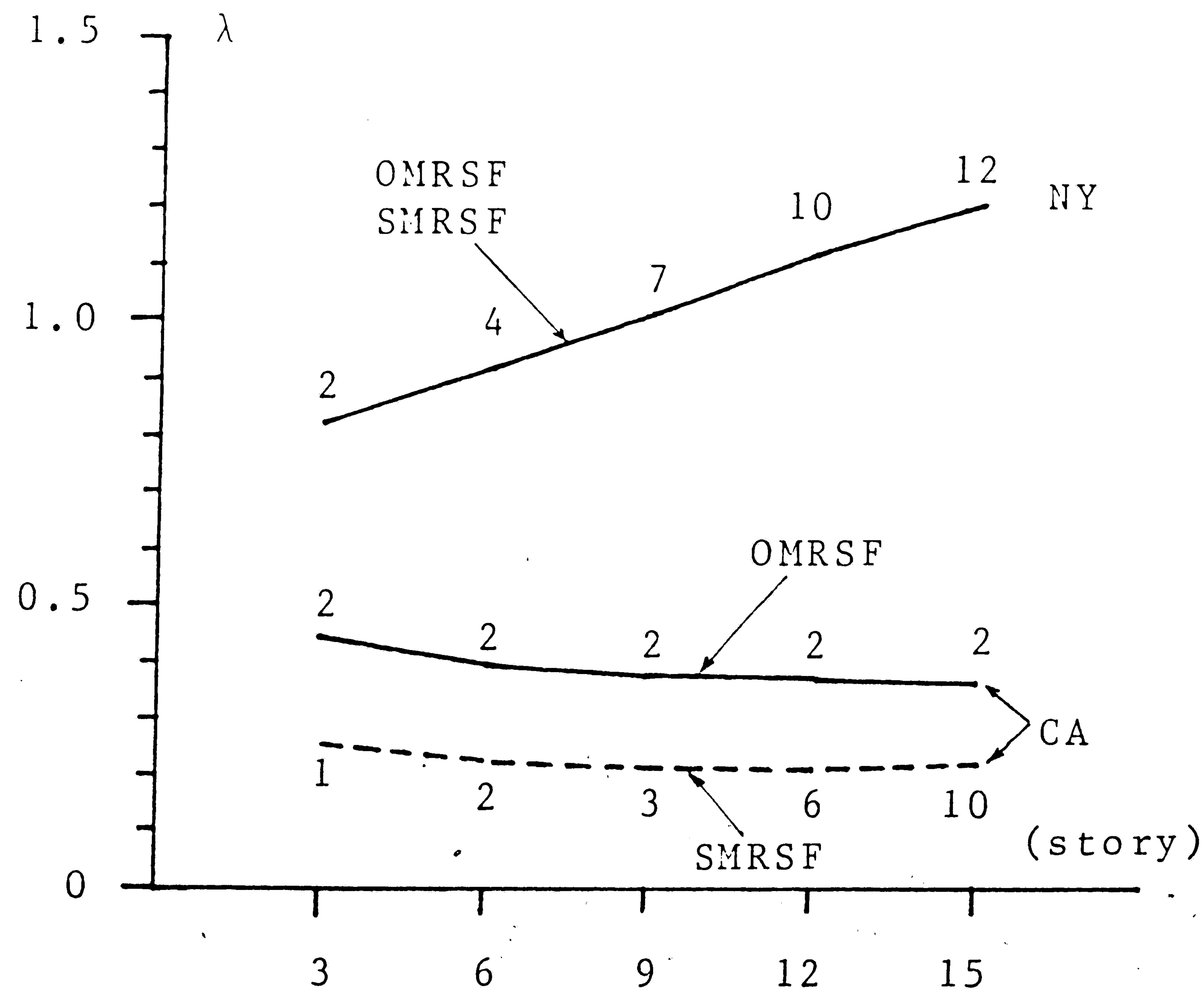


Fig.4-7 Ultimate strength of buildings (DL=50psf, L/B=1.0)

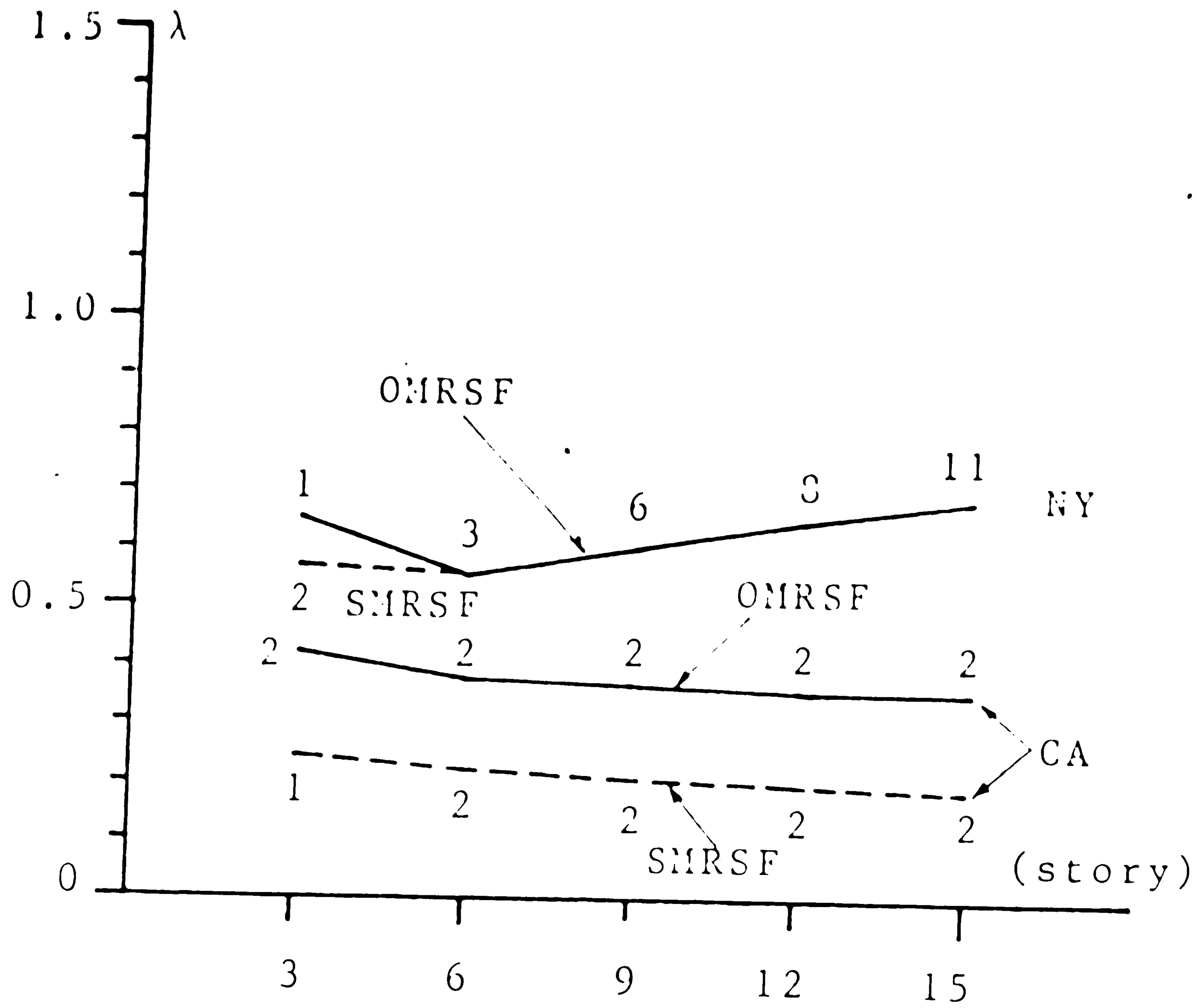


Fig.4-8 Ultimate strength of buildings (DL=100psf, L/B=1.0)

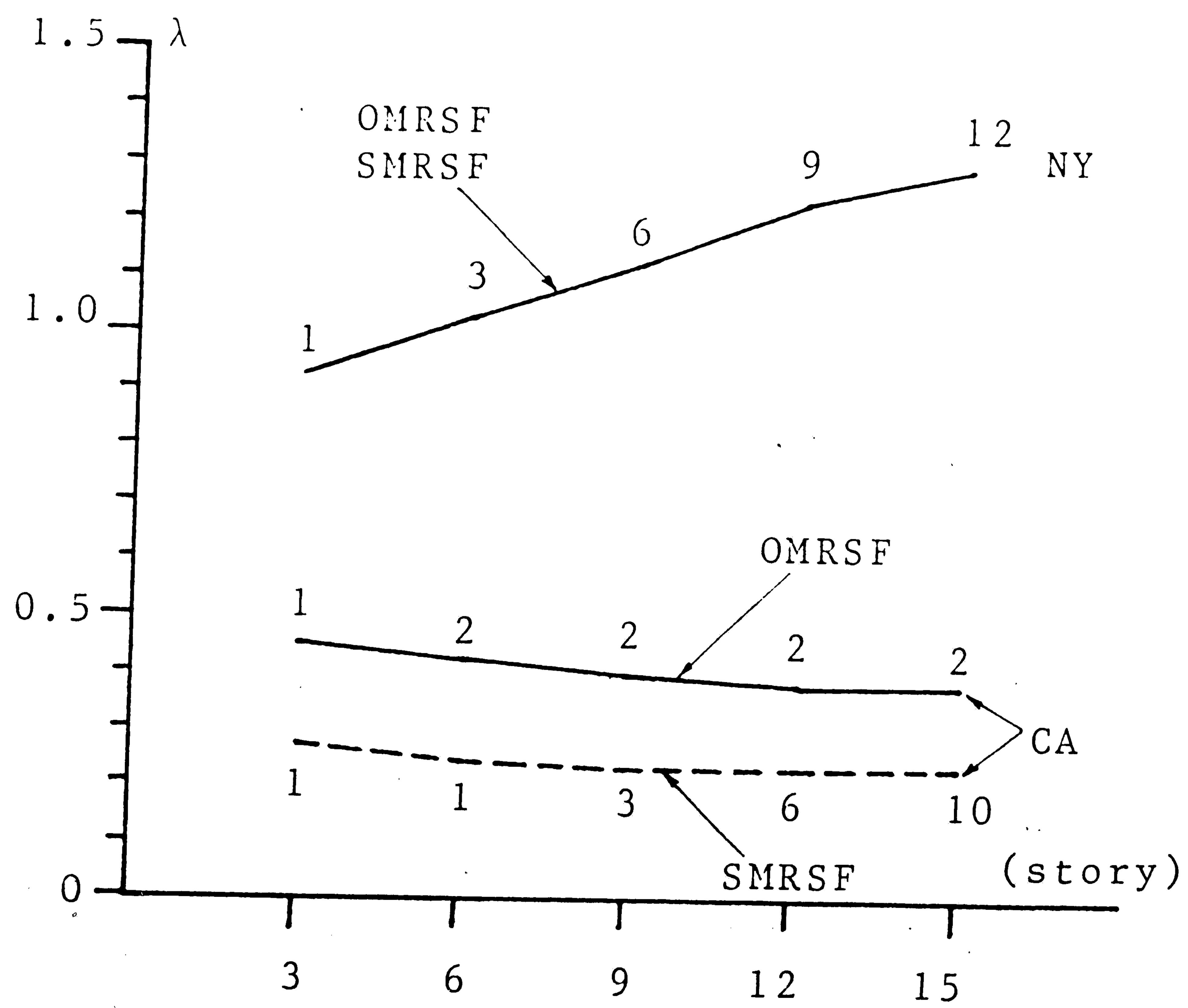


Fig.4-9 Ultimate strength of buildings (DL=75psf, L/B=0.4)

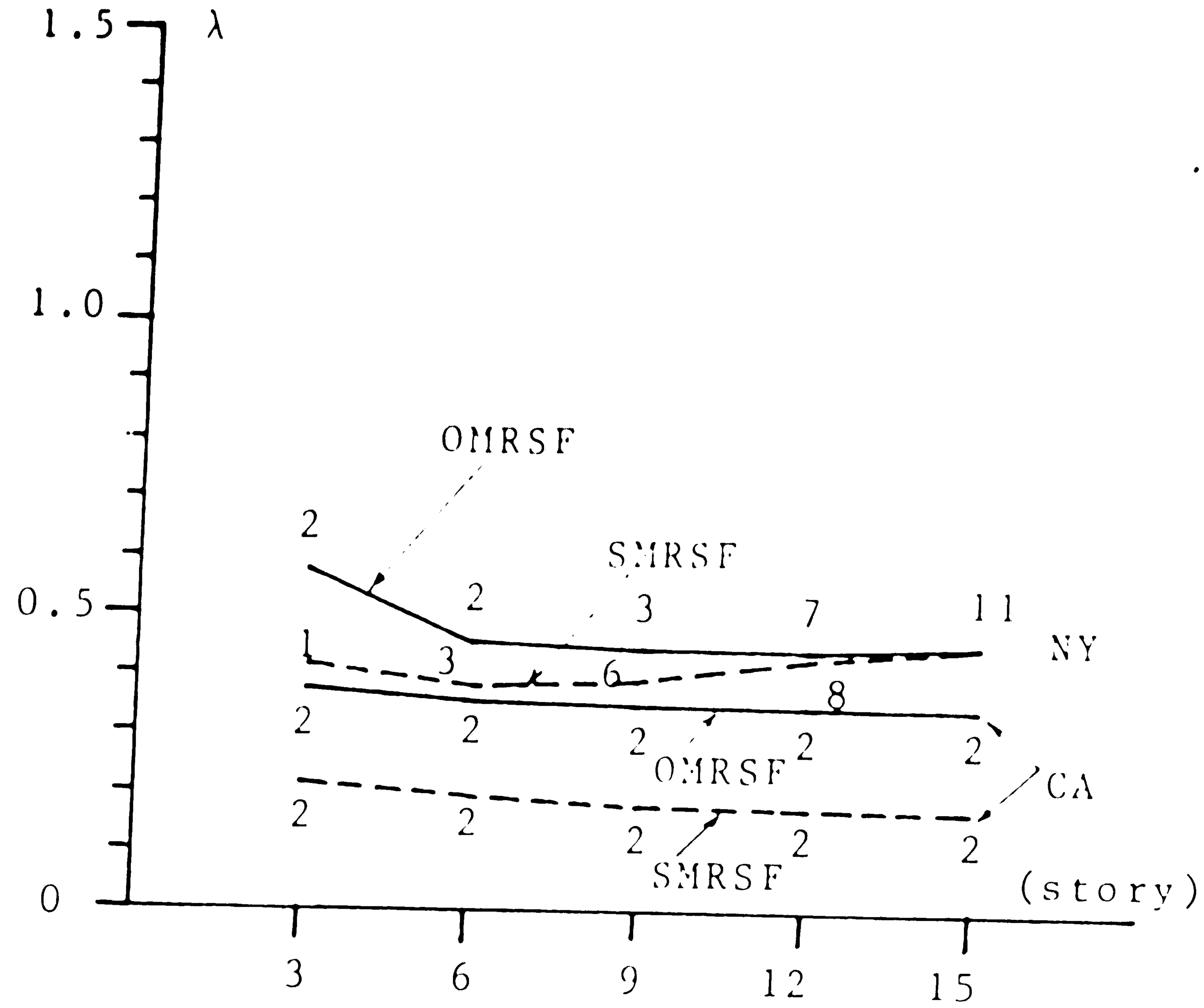
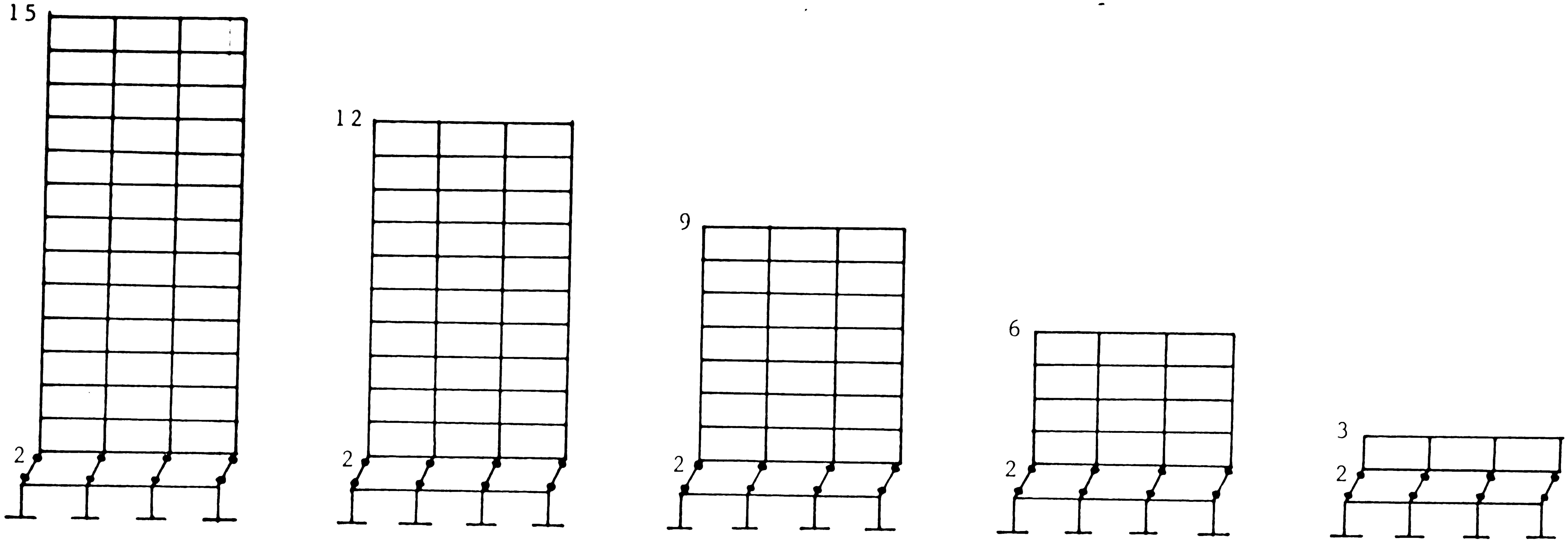
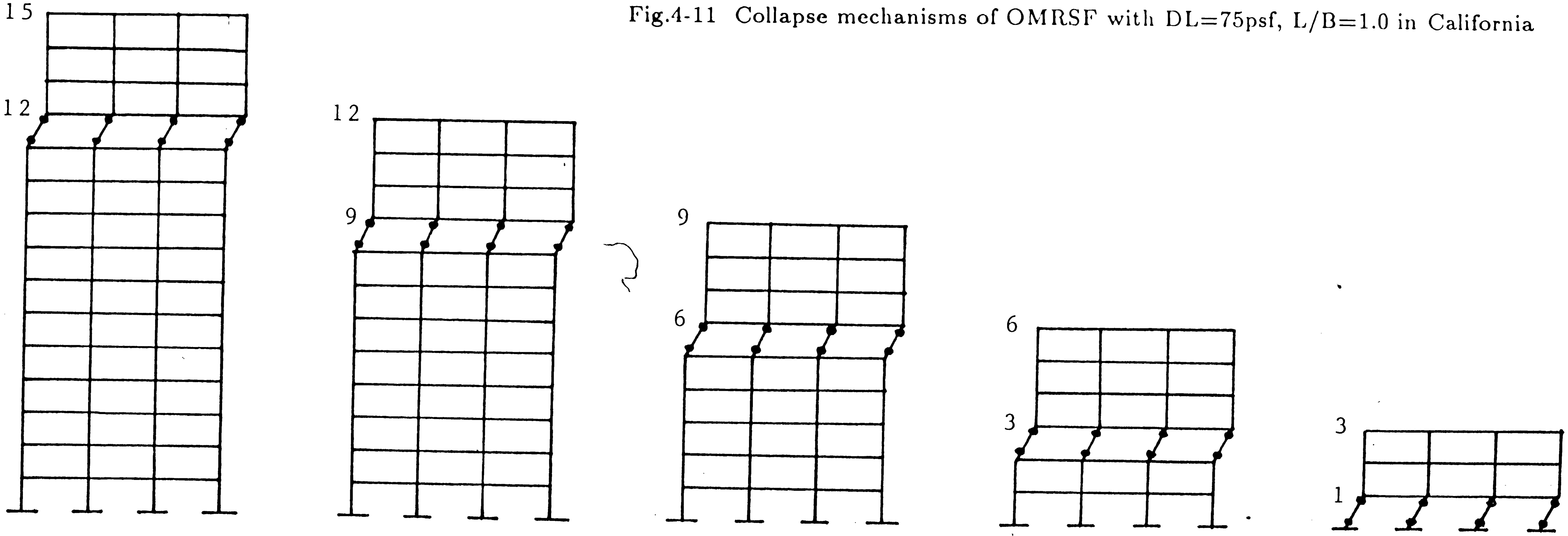


Fig.4-10 Ultimate strength of buildings (DL=75psf, L/B=2.5)



(a) $\lambda_{C2}=0.364$ (b) $\lambda_{C2}=0.369$ (c) $\lambda_{C2}=0.376$ (d) $\lambda_{C2}=0.389$ (e) $\lambda_{C2}=0.429$

Fig.4-11 Collapse mechanisms of OMRSF with DL=75psf, L/B=1.0 in California



(a) $\lambda_{C12}=0.864$ (b) $\lambda_{C9}=0.820$ (c) $\lambda_{C6}=0.741$ (d) $\lambda_{C3}=0.682$ (e) $\lambda_{C1}=0.696$

Fig.4-12 Collapse mechanisms of OMRSF with DL=75psf, L/B=1.0 in New York

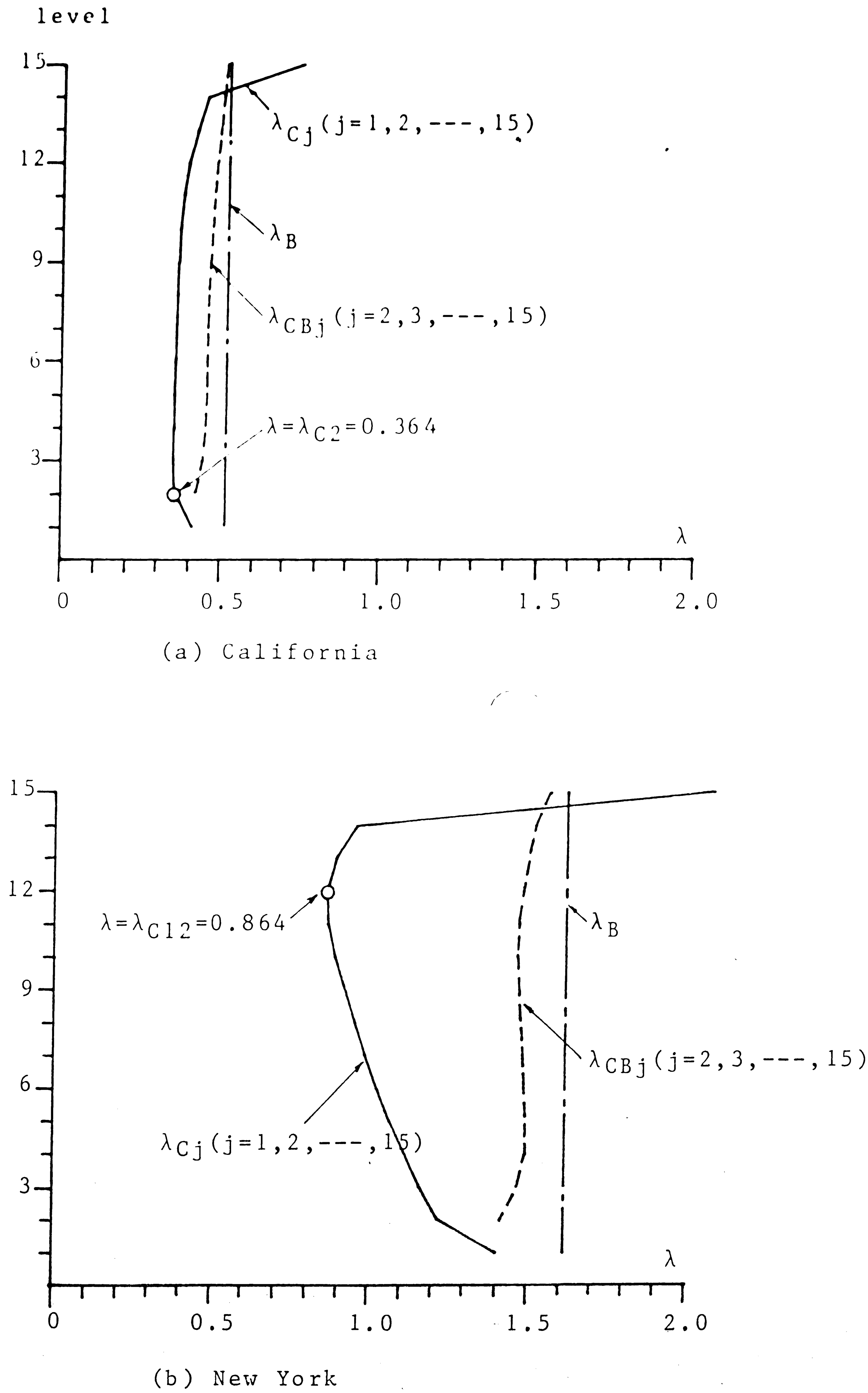
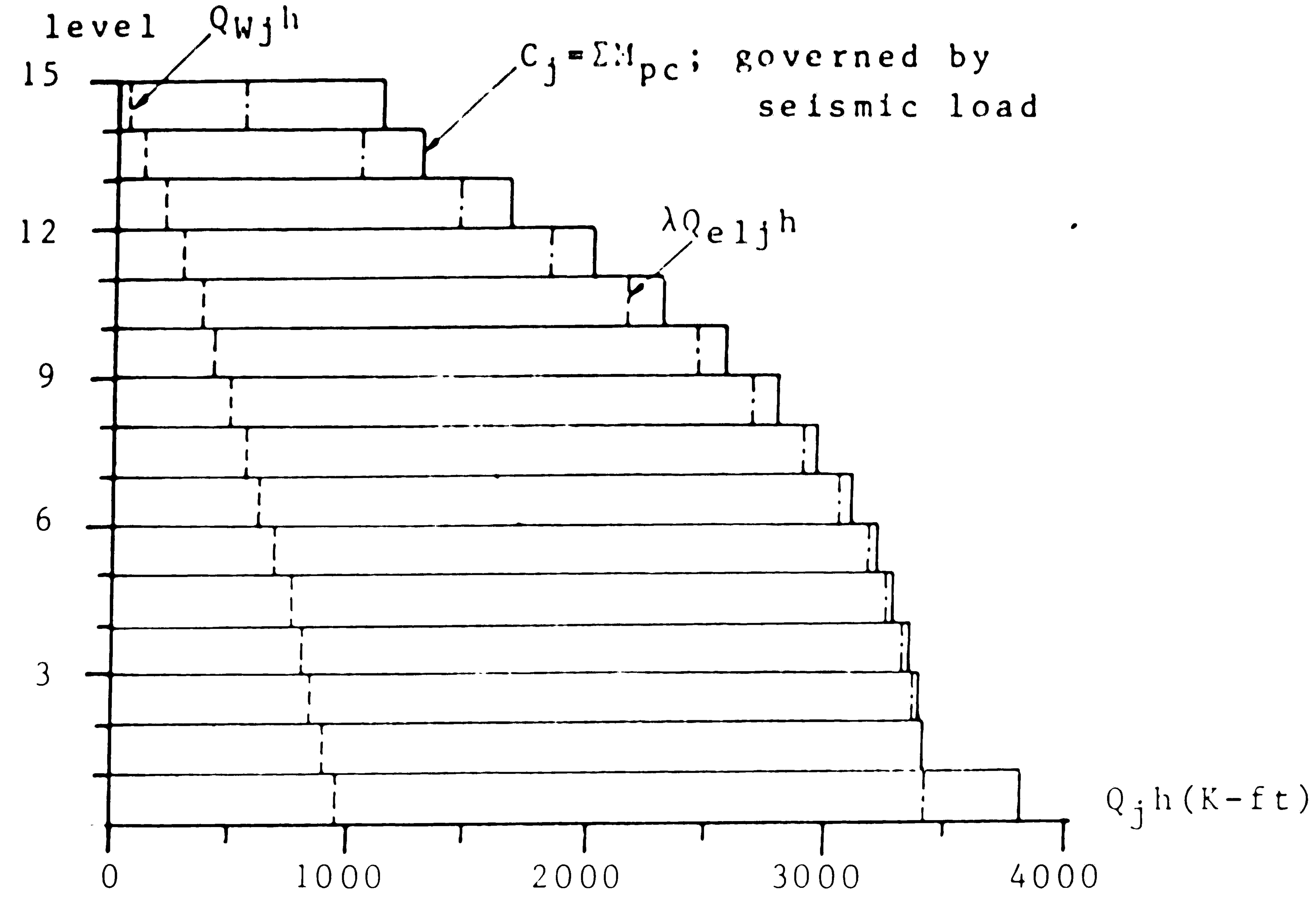
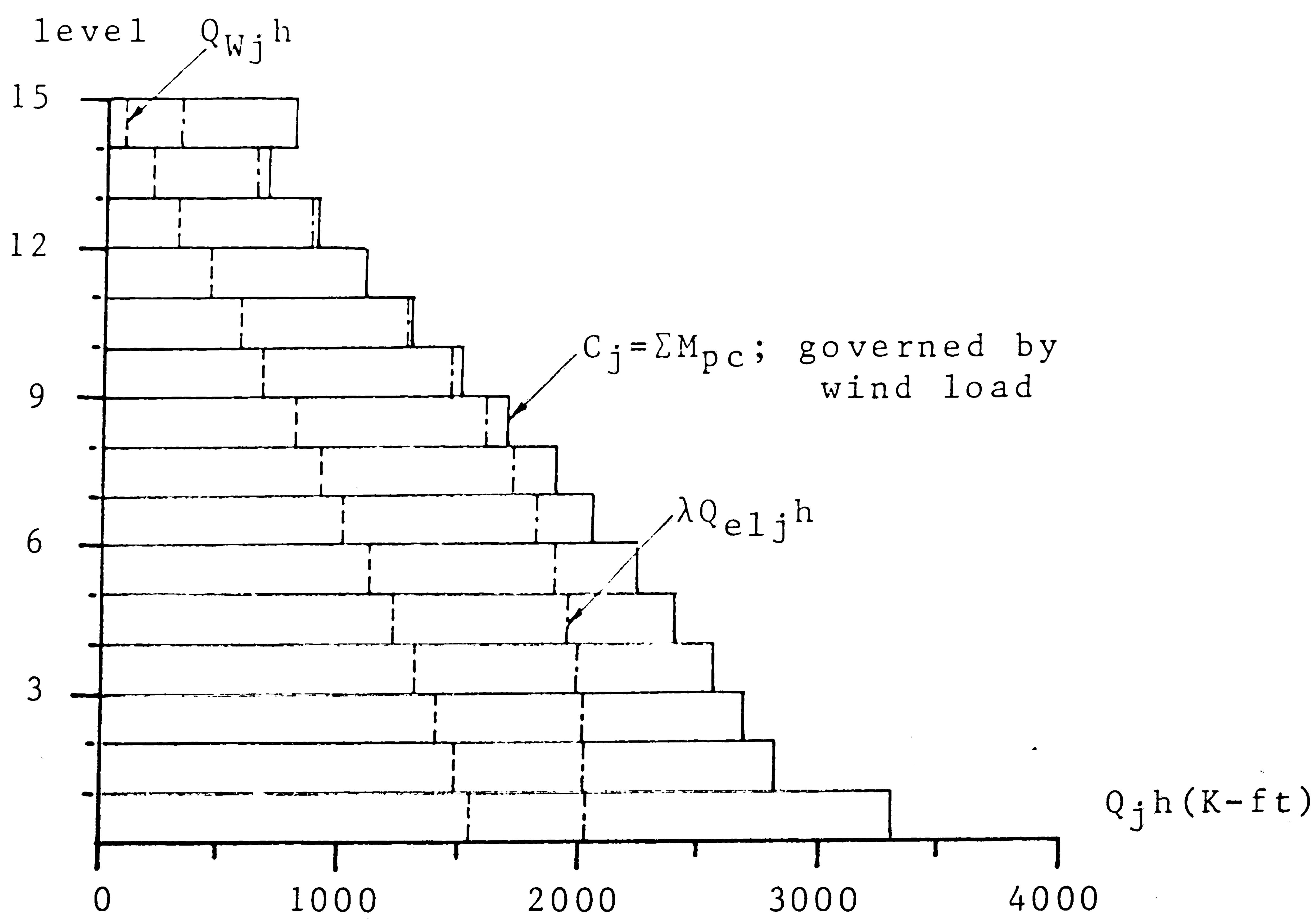


Fig.4-13 Comparison of λ_{Cj}, λ_{CBj}, λ_B of 15-story OMRSF with 75psf, L/B=1.0

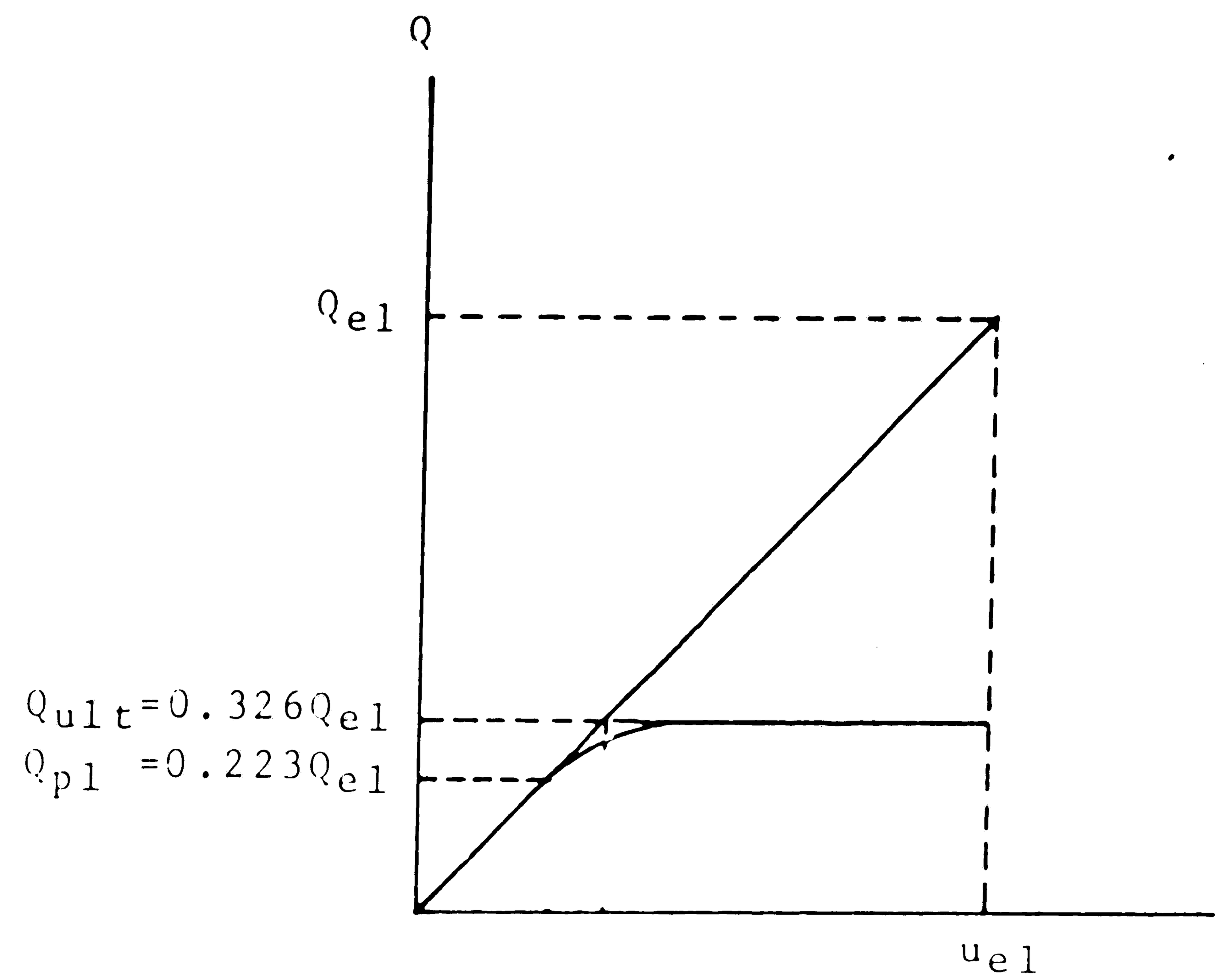


(a) California

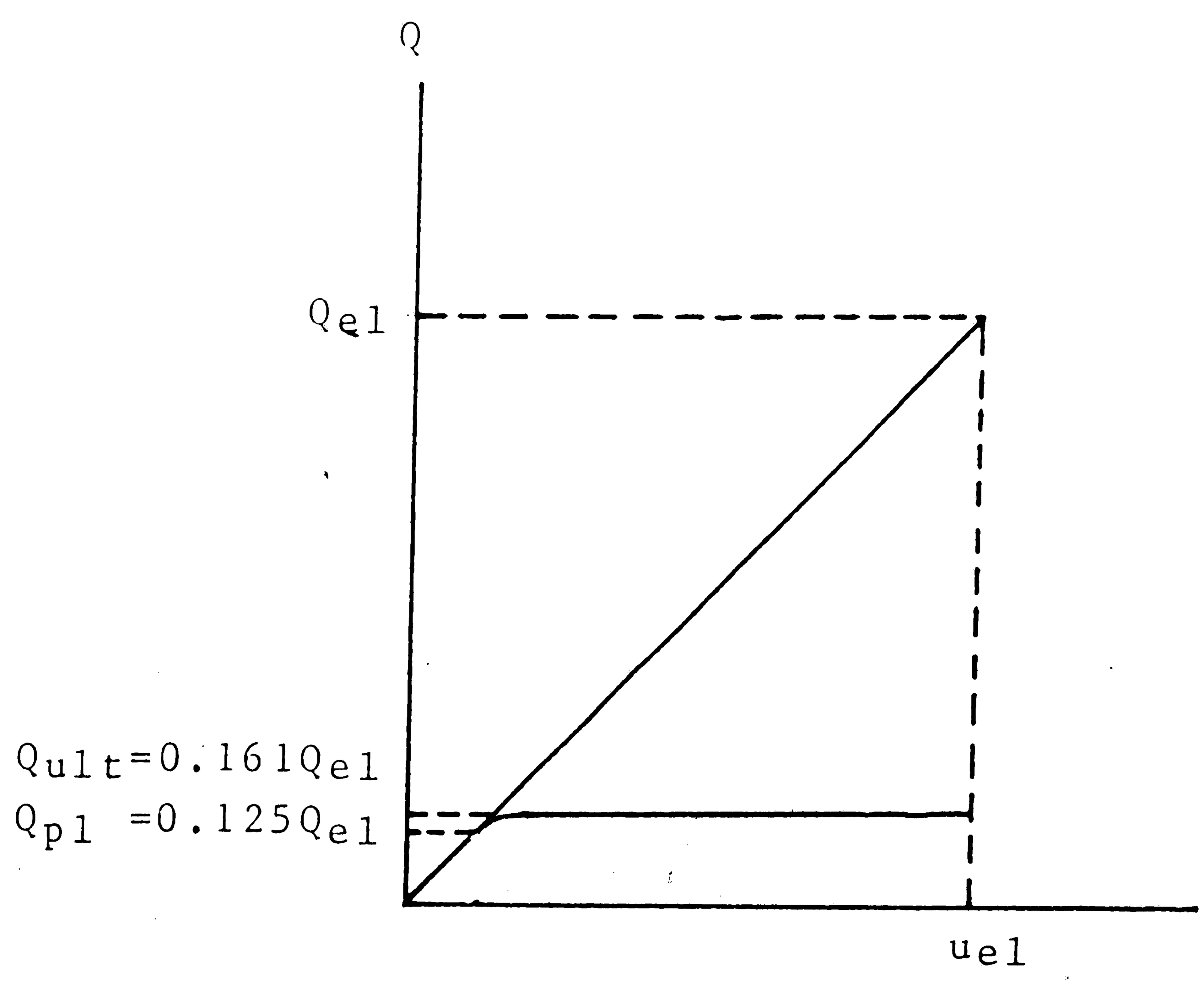


(b) New York

Fig.4-14 Story moment capacity of 15-story OMRSF with DL=75psf, L/B=1.0



(a) OMRSF



(b) SMRSF

Fig.4-15 Ultimate strength of buildings

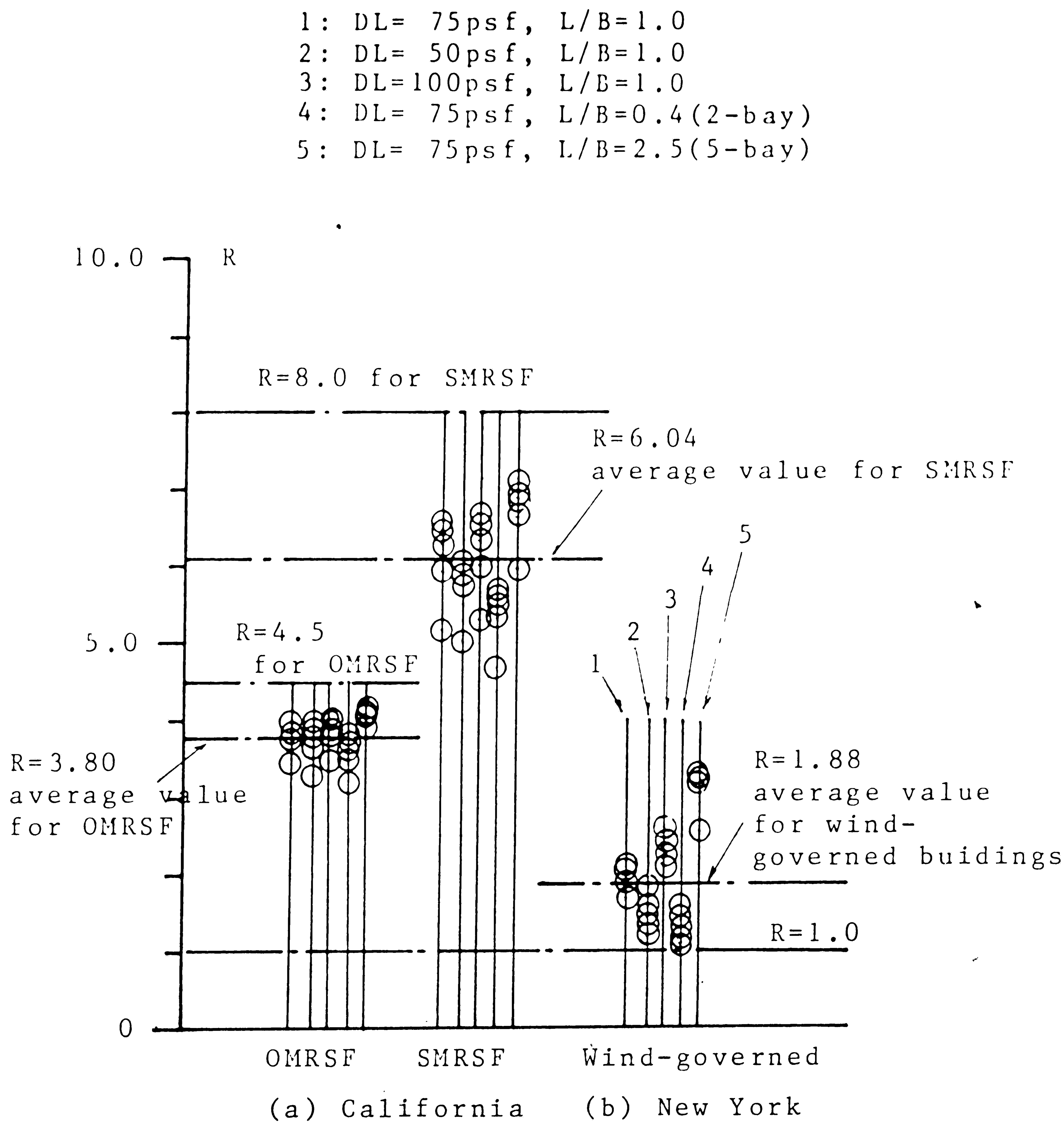


Fig.4-16 Analyzed R factors

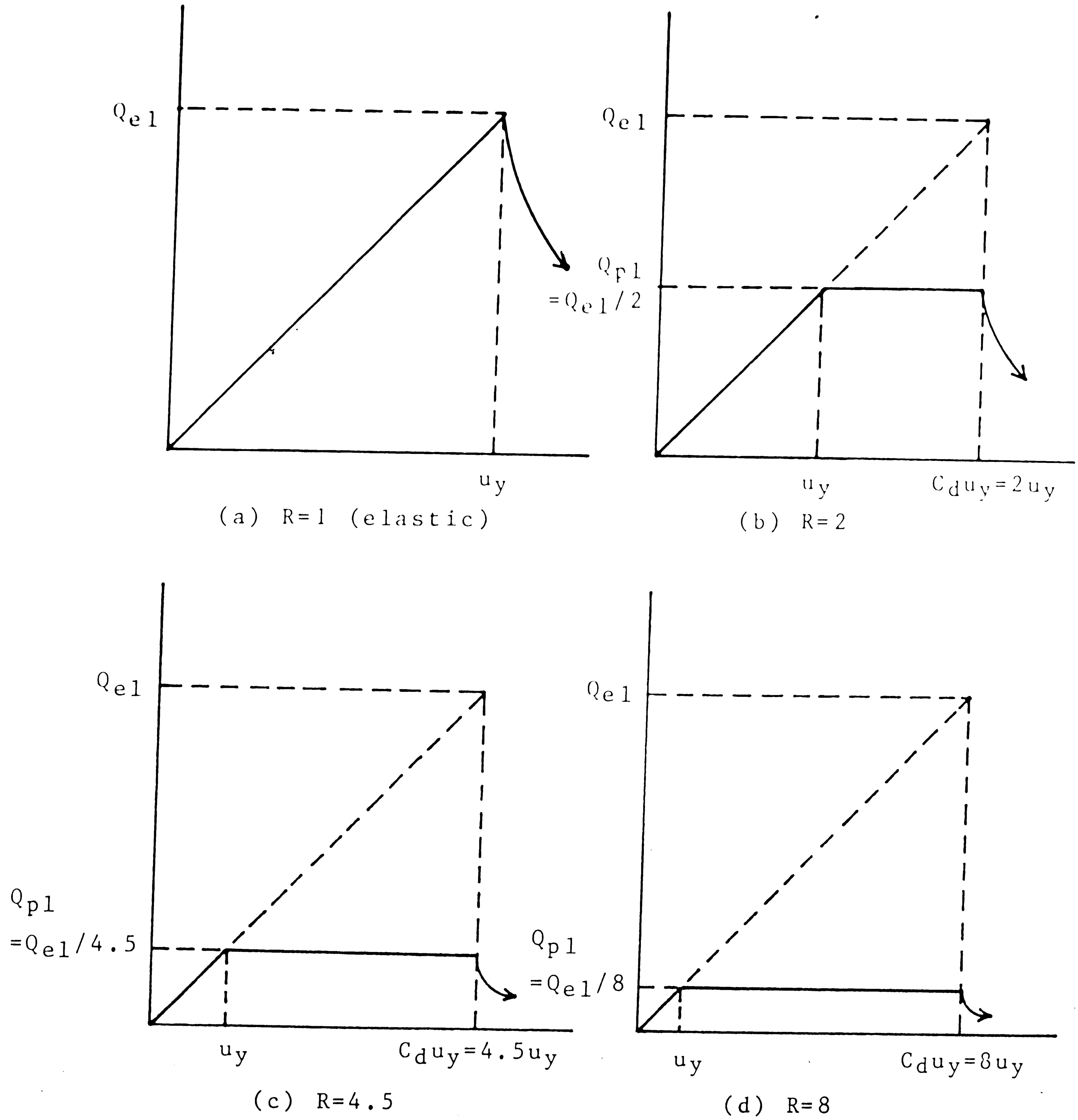


Fig.4-17 Load and deflection relationship for analyzed R factors

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[3.1] Nagata, Masahiro, "*A Study on The Seismic Loads for Plastic Design of Steel Frames*" (in Japanese), Master's Thesis, Osaka university, 1980.

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[4.3] Nagata, Masahiro, "*A Study on The Seismic Loads for Plastic Design of Steel Frames*" (in Japanese), Master's Thesis, Osaka University, 1980.

[4.4] AISC, *Manual of Steel Construction, 8th Edition.* American Institute of Steel Construction, 1980.

[5.1] Chasten, C.P., Fleischman, R.B., Lu, Le-Wu, Driscoll, G.C., "*Semi-Rigid Steel Connections and Their Effects on Structural Frames: Experimental and Theoretical Proposal*", ATLSS Report No. 87-02, Lehigh University, 1987.

7. APPENDIXES

Appendix 1. Trends in High-rise Buildings

The United States has made an important innovation in high-rise buildings using structural systems such as tubular systems and mixed structural systems. Moreover, "High-rise buildings are highly sophisticated engineering projects. Due to the complexity of the structures, the most advanced engineering design techniques are needed in them"[A.1]. By examining this engineering achievement of high-rise buildings and the innovations in the building systems in the United States, we can study the demands on the structural members and the best use of existing materials and new materials. The data for high-rise buildings gathered from the various sources are tabulated in reference[A.2], and they imply the following trends in high-rise buildings.

A1.1 Location of High-rise Buildings

Fig. A1-1 shows the 100 tallest buildings in the world[A.2]. All 100 are taller than 200m in height. Until 1988, the year of the completion of Bank of China, Hong Kong, 12 buildings from the top are all in the United States: 5 in New York, 4 in Chicago, 2 in Houston, and 1 in Seattle. 27 of the 100 tallest buildings are located in New York, 13 are in Chicago, 8 are in Houston, and 8 are in Los Angeles and SanFrancisco, California. Therefore, half of the 100 tallest buildings are concentrated in major U.S. cities and 79 of 100 are in the United States.

Fig. A1-2 to A1-7 show the tallest buildings in major cities in the U.S. and the other countries. Construction of high-rise buildings in the U.S. has two peaks, in the 1930's and the 1970's, while a number of high-rise buildings have been constructed within these 20 years in the high density population areas in the East Asian countries, especially in Japan, Hong Kong, and Singapore. Moreover, it is said that about 60 tall buildings with 40 to 50 stories will be built in the metropolitan area in Tokyo, Japan, early in the 21st Century[A.3].

A1.2 Building Systems of High-rise Buildings

Fig. A1-8 and Table A1-1 show the historical development of high-rise buildings framed by steel, concrete, and mixed structures, respectively[A.2]. Each line indicates the height of the tallest building in the corresponding year. The Empire State Building kept the position of the tallest building in the world for 40 years until the World Trade Center was completed in 1972. In the 1930's high-rise buildings have a uniform building system with the rigid frame and shear wall. A recent development in structural design is the concept of the tubular system introduced by the late Fazlur Khan of Skidmore, Owings & Merrill. At present, 4 of the world's 5 tallest buildings are tubular systems. They are the John Hancock Center, the Sears Tower, the Standard Oil Building in Chicago, and the World Trade Center in New York. "Tubular systems are so efficient that in most cases the amount of structural material used per square foot of floor space is comparable to that used in conventionally framed buildings half the size. The tubular system assumes that the facade structure responds to lateral loads as a closed hollow box beam cantilevering out of the ground. Since the exterior walls resist all or most of the wind loads, costly interior diagonal bracing and shear walls are eliminated."[A.4]

Fig. A1-9 and Table A1-2[A.2,A.5] show the relationship between the number of stories and unit weight of structural steel. It is evident that the tubular system has an advantage in structural efficiency and is close to the optimum structures subjected to not only vertical loads but also lateral loads proposed in reference[A.6].

A1.3 Structural Materials Used in High-rise Buildings

Fig. A1-10 shows the spread of steel-framed buildings compared to that of concrete and mixed structures located in major cities, New York, Chicago, and California (the sum of Los Angeles and San Francisco) and all Japan. In Japan all buildings exceeding 100m in height are steel-framed or mixed structures and none of them are concrete. Japan is located in a high seismic zone, and there is a feeling against concrete on account of its brittle failure manner under cyclic loading and also because of the massiveness of concrete columns in high-rise buildings. In the United States, on the contrary, lots of concrete high-rise buildings with 100m and more in height, even with 200m and more, have been constructed. Although steel is the predominant structural material used in high-rise buildings, there is a trend in recent years that concrete structures, especially mixed structures, are favorably used from the viewpoint of its advantages in rigidity due to high-strength concrete together with modern construction techniques as shown in Figs. A1-11 and A1-12.

Table A1-1 Historical Development of Tall Buildings

Material	No.	Building	City	Year	Height	Memo
Steel	1	Home Insurance	Chicago	1885	55	iron & steel
	2	American Surety	New York	1895	92	First steel
	3	St. Paul	New York	1896	95	
	4	Park Row	New York	1898	118	
	5	Singer	New York	1907	187	
	6	Metropolitan Tower	New York	1909	206	
	7	Woolworth	New York	1913	242	
	8	Chrysler	New York	1929	319	
	9	Empire State	New York	1931	381	
	10	World Trade Center	New York	1972	417	
	11	Sears Tower	Chicago	1974	442	
Concrete	1	Ingalls	Cincinnati	1903	64	
	2	Ilikaii	Honolulu	1963	79	Prestressed
	3	Lake Point Towers	Chicago	1968	196	
	4	One Shell Plaza	Houston	1970	218	Light-weight
	5	Carlton Center	Johannesburg	1973	220	
	6	Water Tower Place	Chicago	1976	262	
	7	1 Wacker Drive	Chicago	1990	295	Under const.
Mixed	1	Palac Kultury I Nauki	Warsaw	1955	241	
	2	Texas Commerce Plaza	Houston	1981	305	
	3	Bank of China	Hong Kong	1988	368	

(Unit of height : meter)

Table A1-2 Unit Weight (U.W.) of Structural Steel

Building	City	Year	Stories	Height	U.W.	Structural System
Empire State Building	New York	1930	102	381	204	frame, shear wall
John Hancock Center	Chicago	1968	100	344	144	Trussed tube
World Trade Center	New York	1972	110	412	179	Framed tube
Sears Tower	Chicago	1974	109	443	160	Bundled tube
Chase Manhattan	New York	1963	60	248	267	Long-span frame
First National Bank	Chicago	1969	60	257	184	
US Steel Building	Pittsburgh	1971	64	256	146	
I.D.S. Center	Minneapolis	1971	57	235	86	Belt truss system
Seagram Building	New York	1957	42	160	136	
Boston Co. Building	Boston	1970	41	183	102	
Civic Center	Chicago	1965	30	202	184	
Alcoa Building	San Francisco	1969	26	121	126	
Low Income Housing	Brockton	1971	10	?	30.5	
Crysler Building	New York	1930	77	319	179	
Esso Building	New York	1945	32	?	132	
UN Secretariat	New York	1950	42	?	155	
Sinclair Oil	New York	1950	27	?	129	
Alcoa Building	Pittsburgh	1951	30	125	125	
641 Lexington Ave.	New York	1955	33	?	92.0	
Socony Mobile	New York	1956	42	174	107	
Corning Glass	New York	1957	26	?	107	
2 Broadway	New York	1957	30	?	81.3	
80 Pine Street	New York	1958	38	?	90.5	
Gateway Center Bldg.	Pittsburgh	1959	22	?	86.6	
United Engrg. Center	New York	1960	20	?	129	
Sperry Rand	New York	1960	43	174	96.8	
Pan. Am. Building	New York	1961	59	246	155	
Chem. Bank NY Trust	New York	1962	50	209	141	
J. C. Penny Bldg.	New York	1963	46	186	112	

(Unit of height, U.W. : meter, kg/m^2)

Table A1-2 Unit Weight (U.W.) of Structural Steel (Continued)

Building	City	Year	Stories	Height	U.W.	Structural System
Connecticut Mutual	Chicago	1967	25	104	109	
437 Madison Ave.	New York	1967	41	?	91.5	
Owens	Toledo	1968	30	?	107	
Burlington House	New York	1968	50	191	98.0	
First National Bank	Seattle	1969	50	185	119	
77 Water Street	New York	1969	29	?	77.4	
2 First National Plaza	Chicago	1970	30	168	103	
One Liberty Plaza	New York	1971	54	227	129	
IBM Building	Chicago	1971	54	212	129	
McGraw-Hill	New York	1971	51	204	126	
First National Bank	Portland	1972	40	163	121	
One Beacon Street	Boston	1972	40	?	107	

(Unit of height, U.W. : meter, kg/m^2)

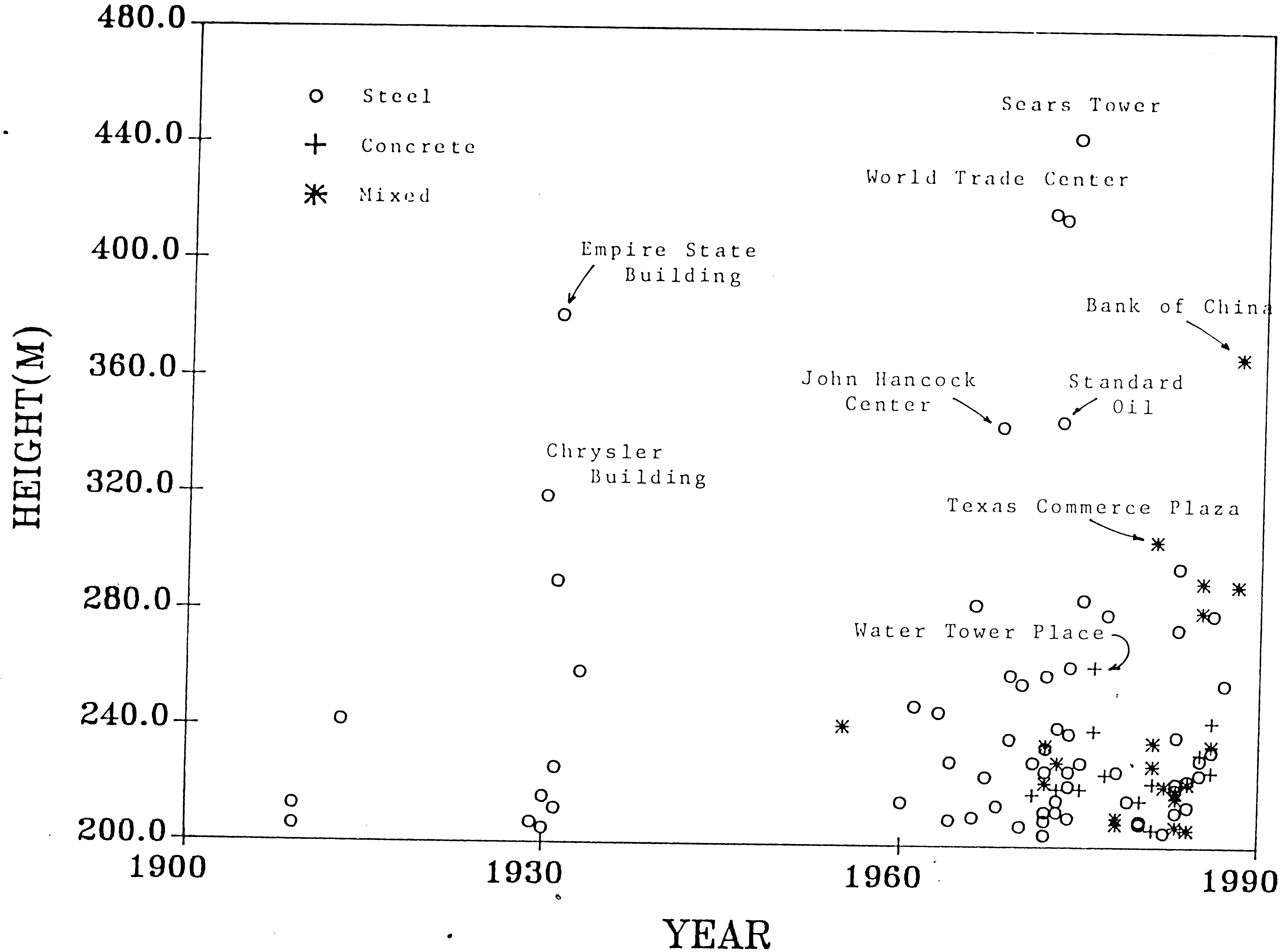

Fig.A1-1 Top 100 tallest buildings in the world

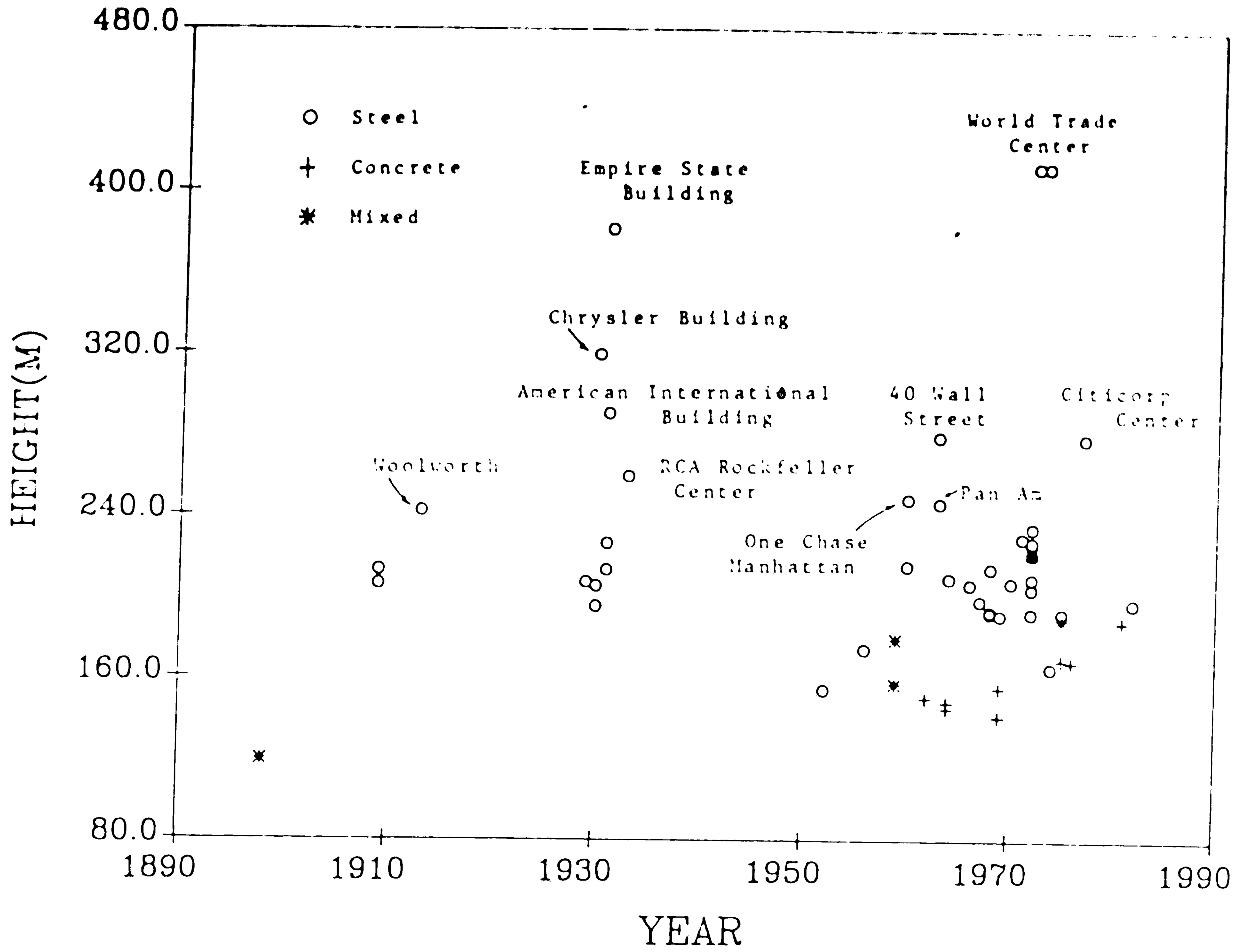

Fig.A1-2 Tall buildings in New York

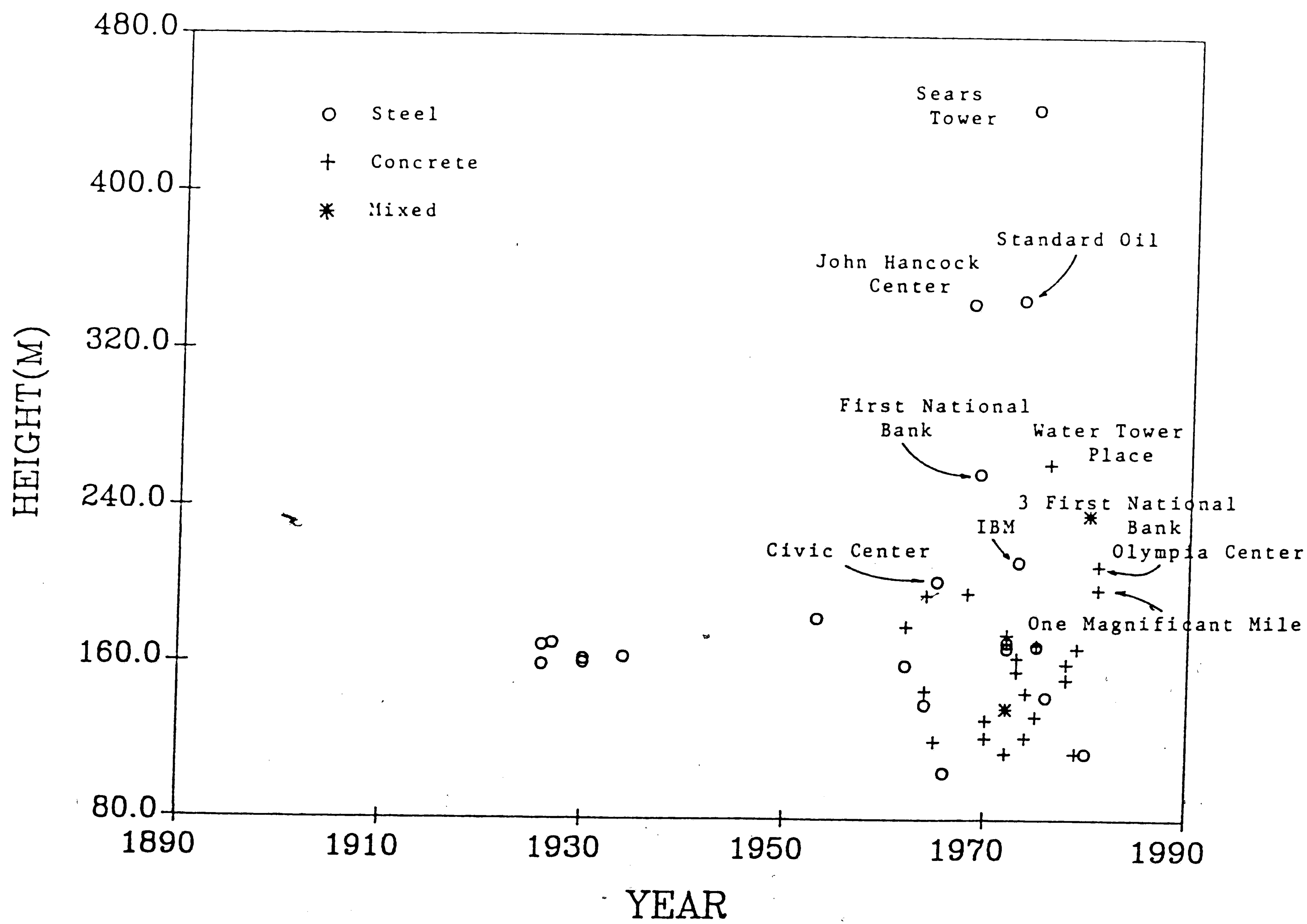

Fig.A1-3 Tall buildings in Chicago

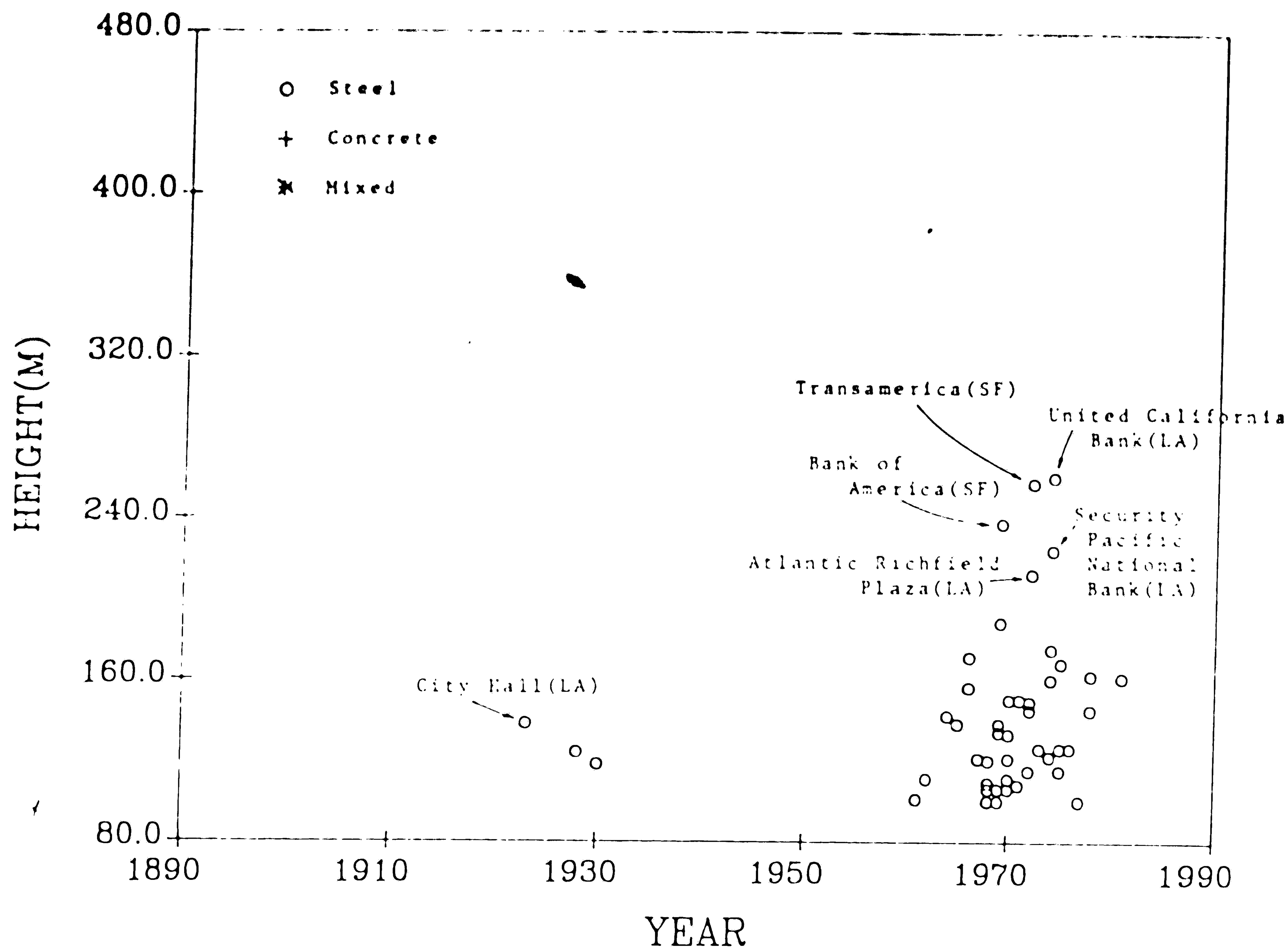

Fig.A1-4 Tall buildings in Los Angeles and San Francisco

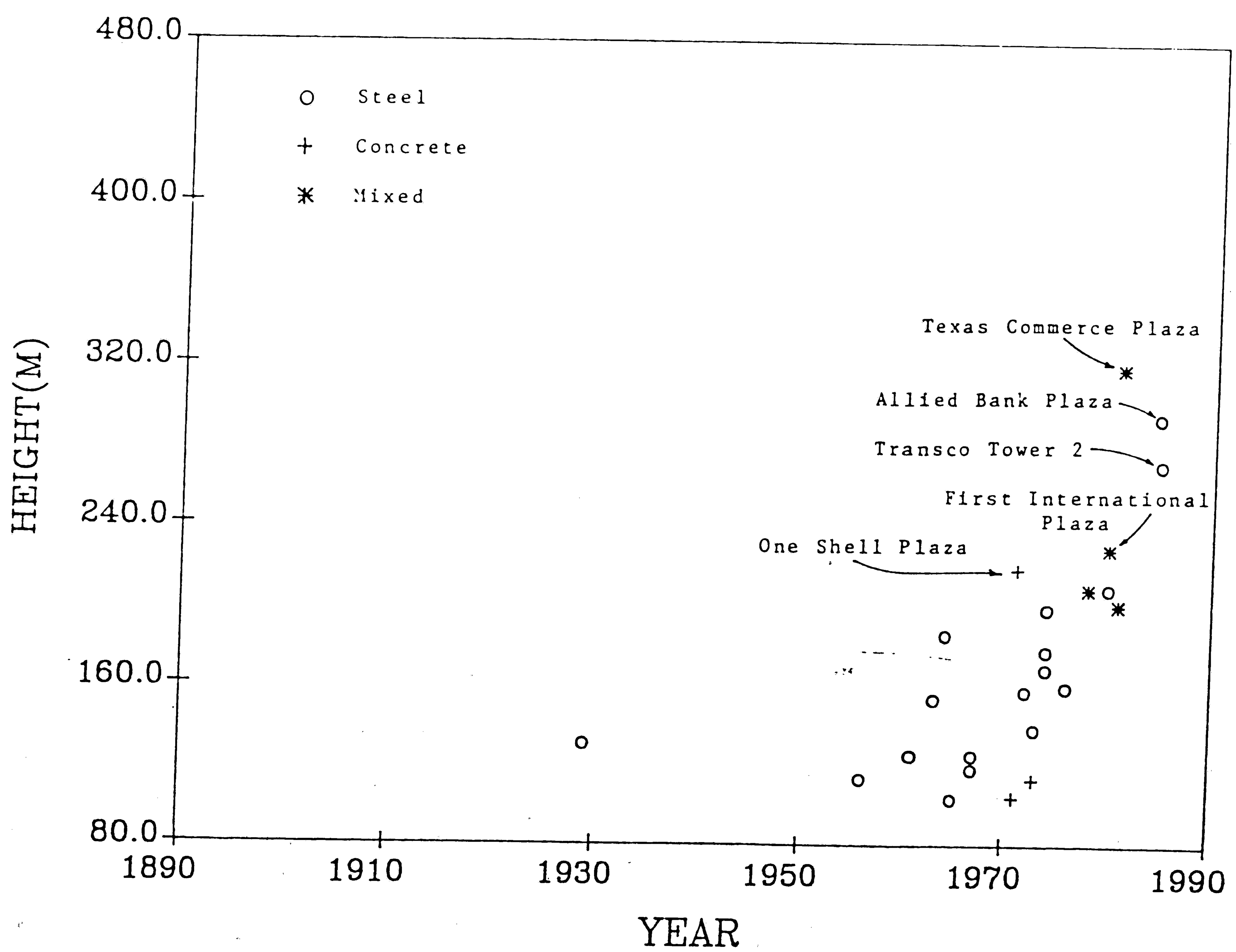

Fig.A1-5 Tall buildings in Houston

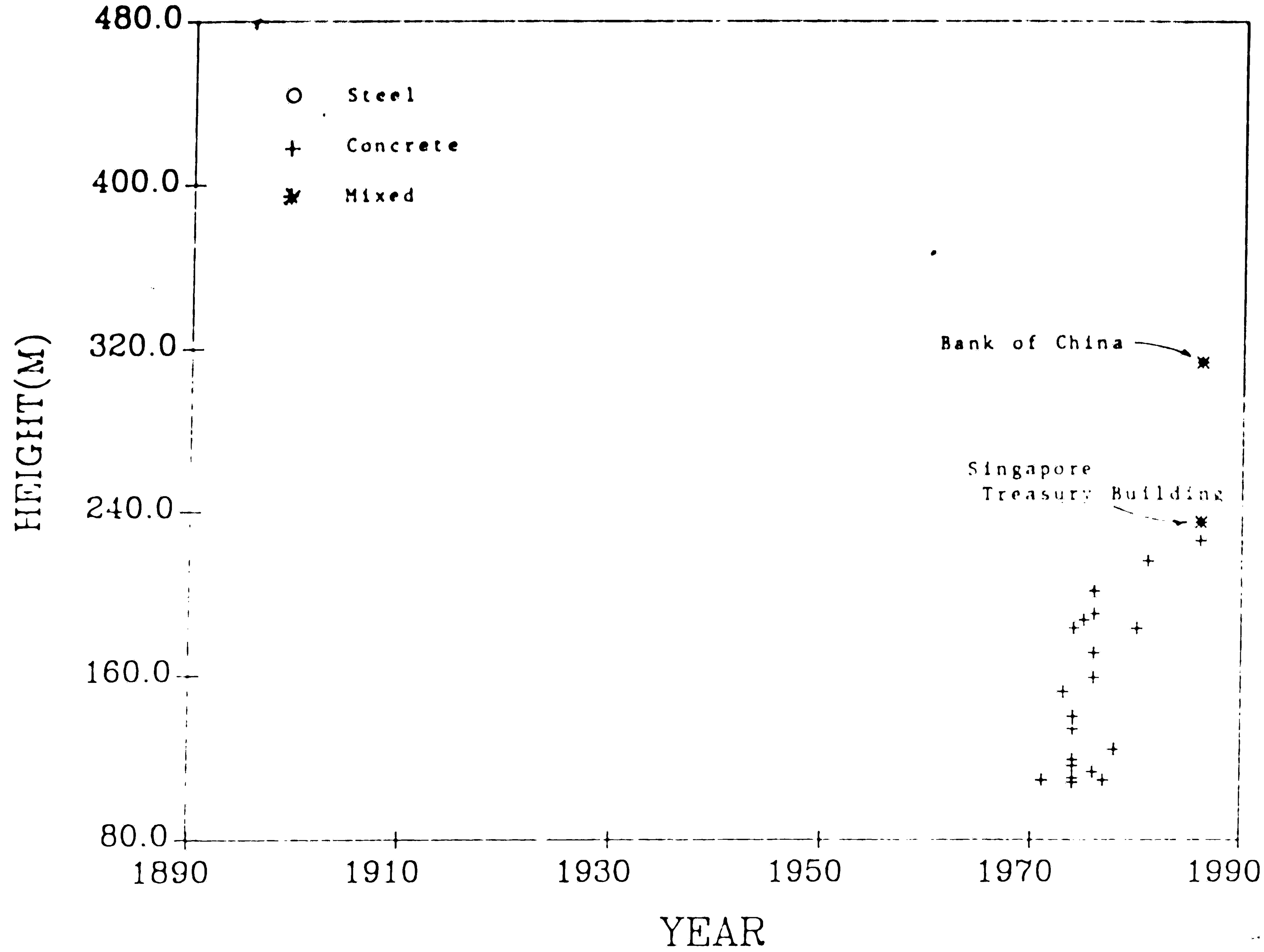

Fig.A1-6 Tall buildings in Hong Kong and Singapore

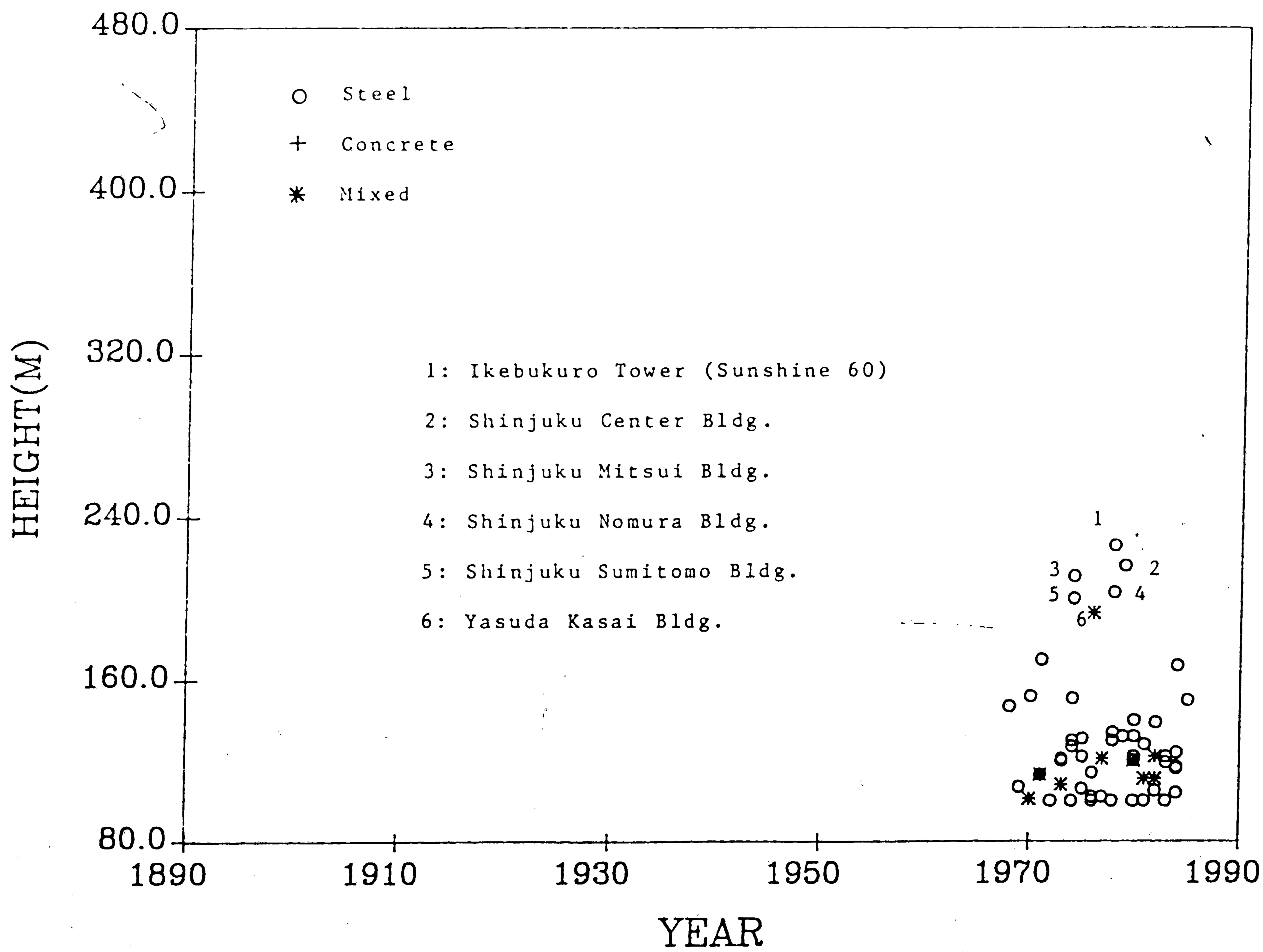

Fig.A1-7 Tall buildings in Japan

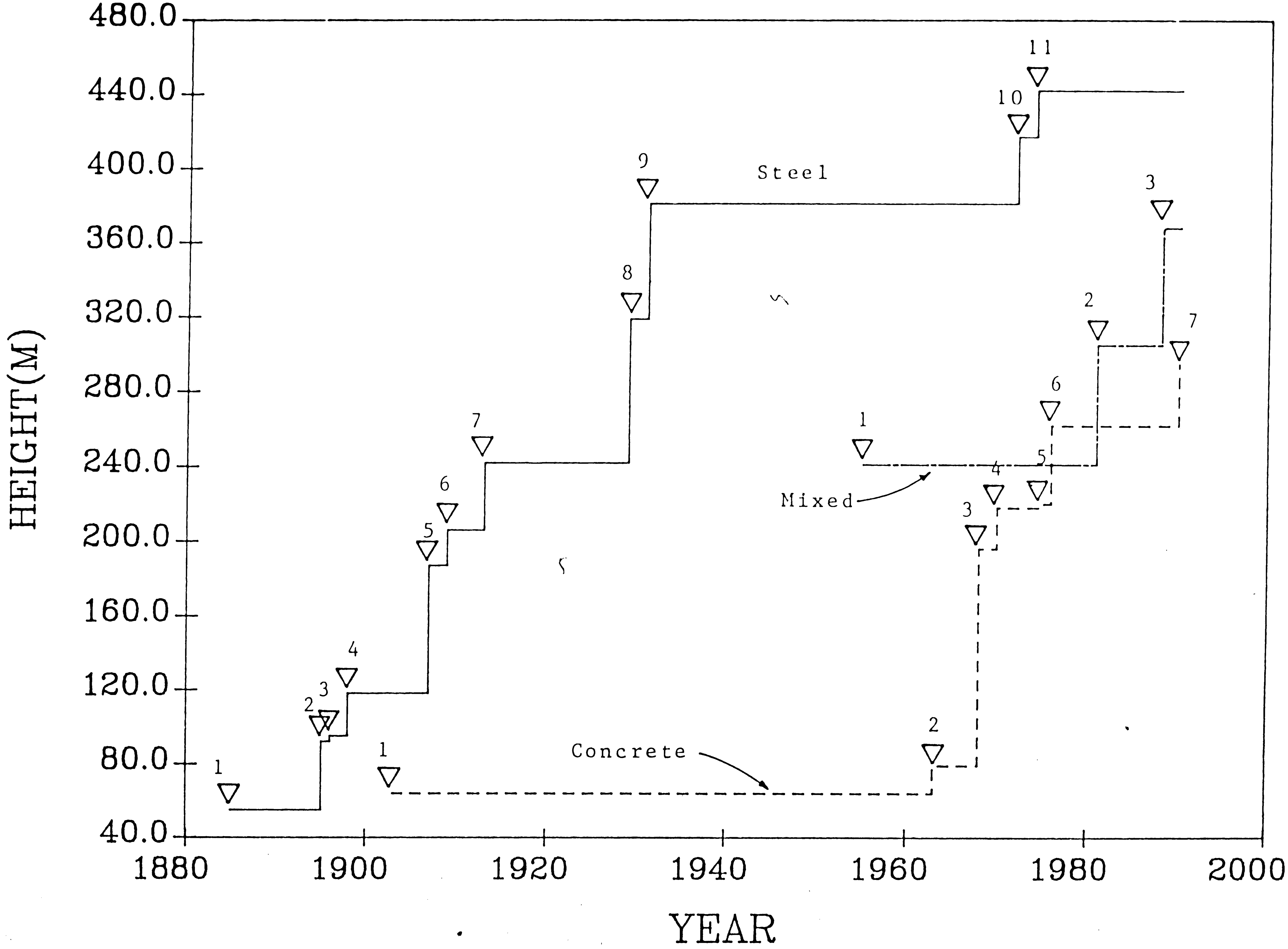

Fig.A1-8 Historical Development of tall buildings

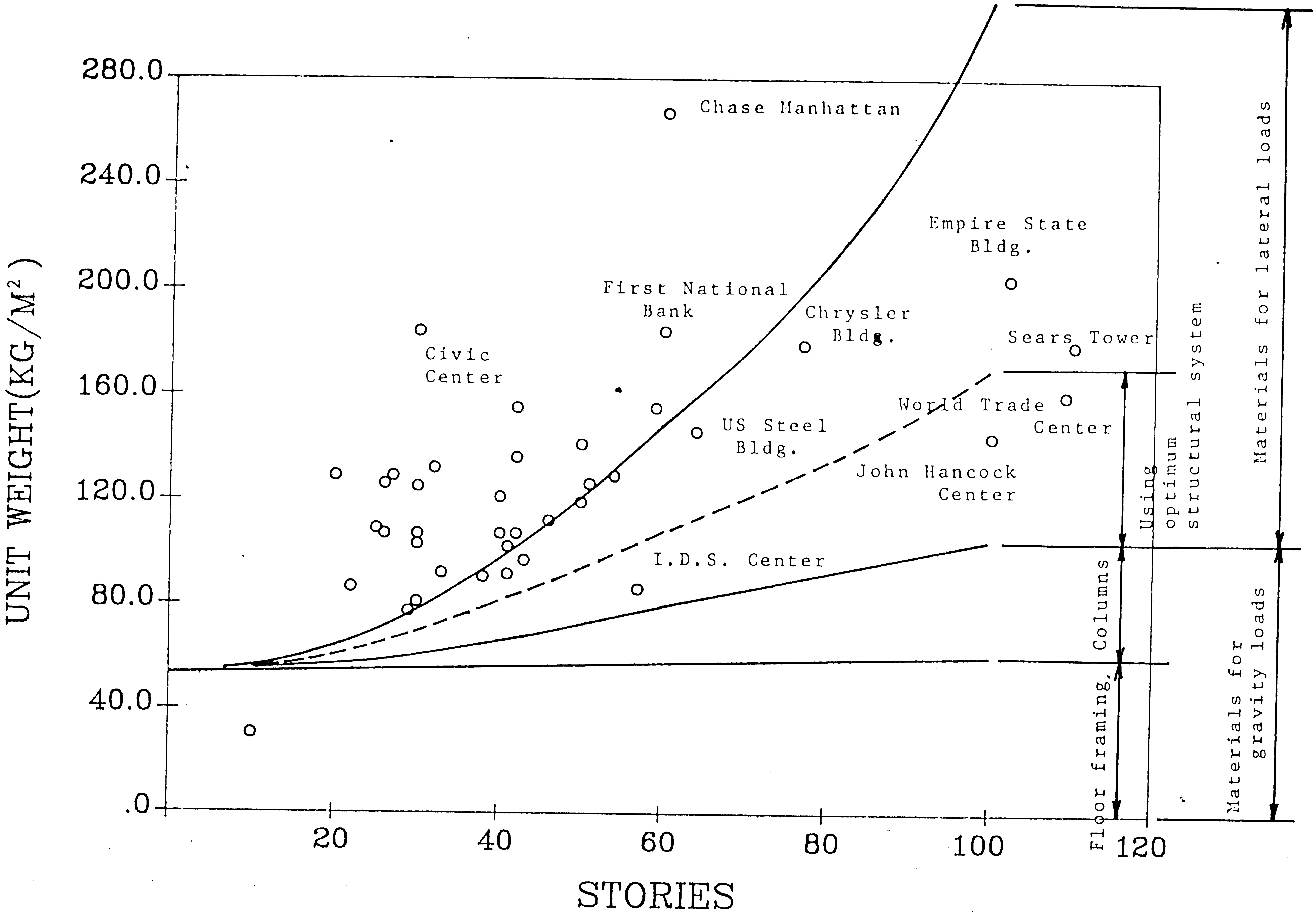

Fig.A1-9 Relationship between the number of stories and unit weight of structural steel

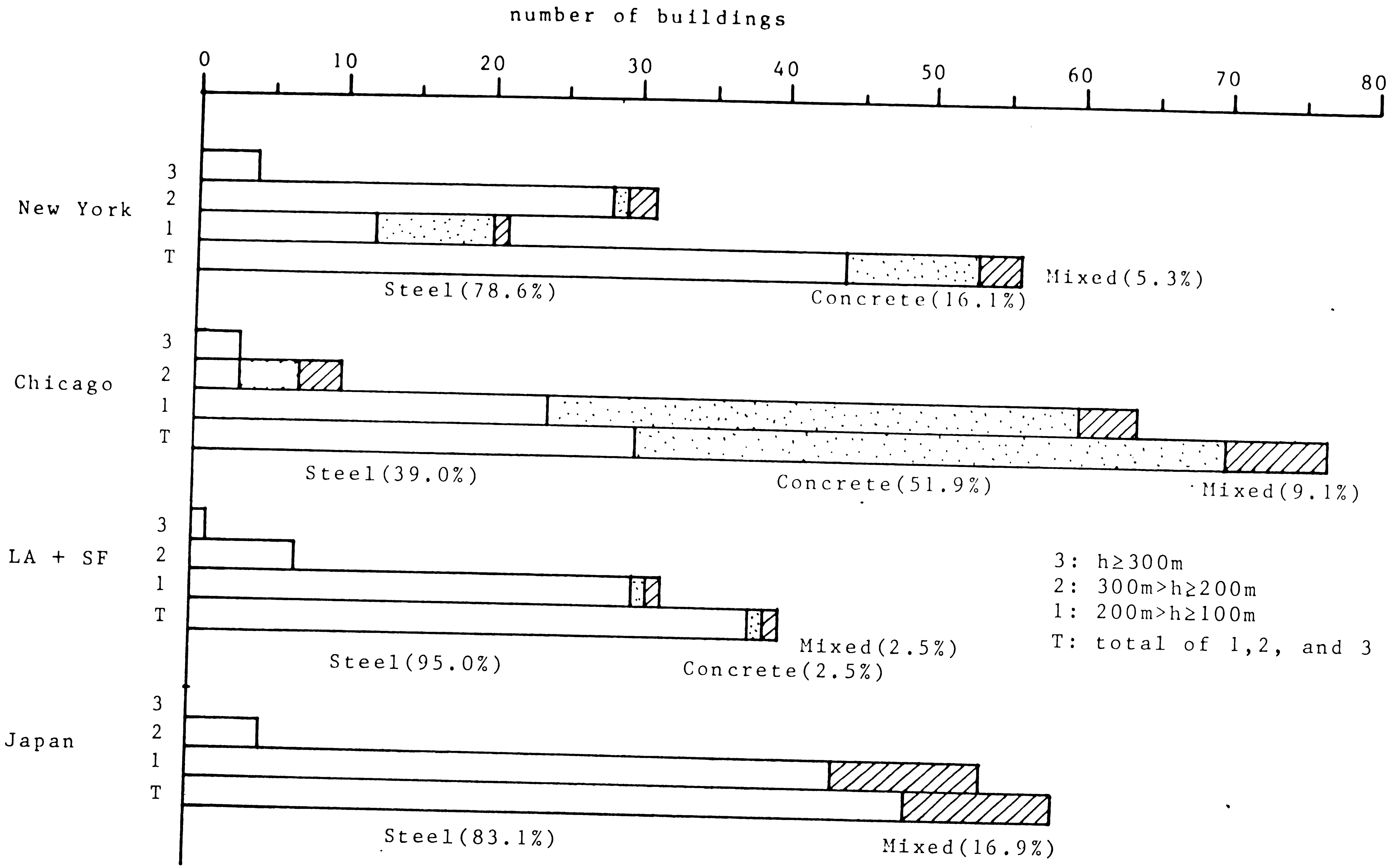

Fig.A1-10 Structural materials used in tall buildings in major cities

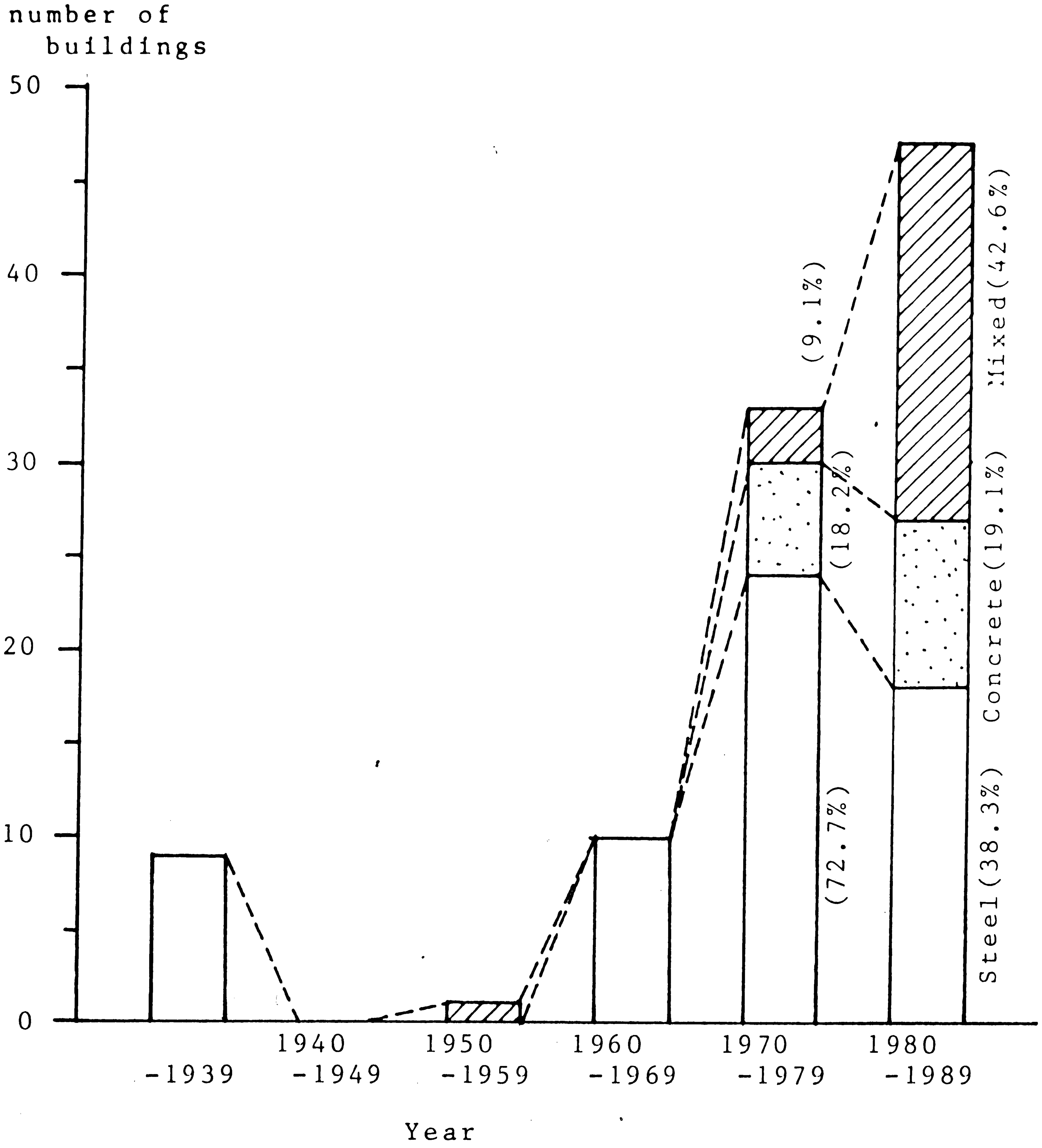

Fig.A1-11 Structural materials used in top 100 tallest buildings in the world

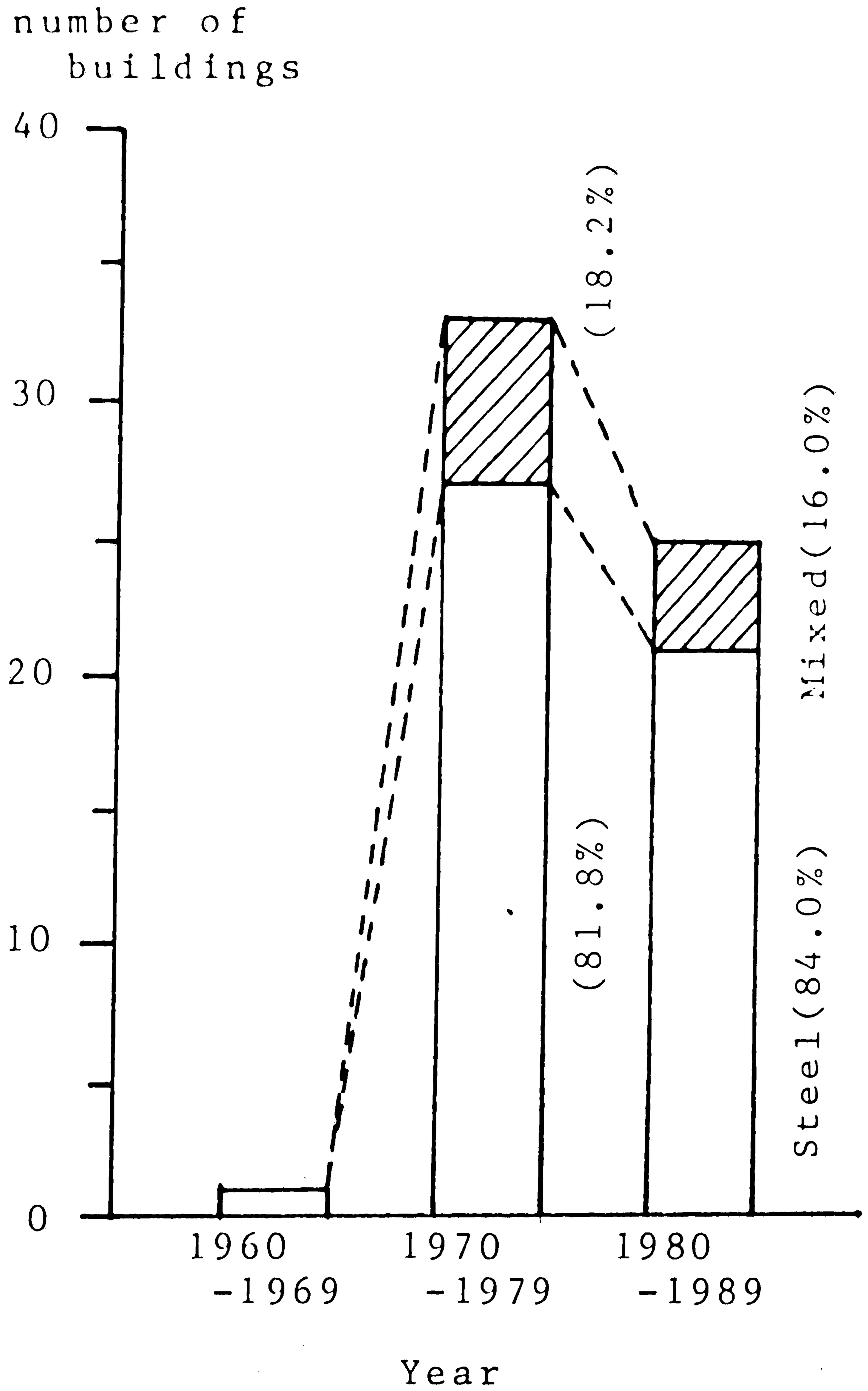

Fig.A1-12 Structural materials used in tall buildings in Japan

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Appendix 2. The Japanese Seismic and Wind Codes

A2.1 The Seismic Design Loads

The seismic design shear at each level is determined by the following formula:

$$Q_j = C_j \cdot W_j \tag{A2.1}$$

in which C_j is the seismic shear coefficient for the j-th story and W_j, the weight of the building above the j-th story. C_j is biven by

$$C_j = Z \cdot R_t \cdot A_j \cdot C_0 \tag{A2.2}$$

where Z: seismic zoning coefficient. The value of 1.0 is used for Tokyo.

R_t: design spectral coefficient. R_t for soil condition 2, which is equivalent to deep stiff soil over rock in the NEHRP, is given by,

$$\begin{aligned} R_t &= 1.0 && (T \leq 0.6 \text{ sec.}) \\ &= 1-0.2\cdot(T/0.6-1)^2 && (0.6 \text{ sec.} \leq T \leq 1.2 \text{ sec.}) \\ &= 0.96/T && (T \geq 1.2 \text{ sec.}) \end{aligned} \tag{A2.3}$$

A_j: lateral shear distribution factor given by,

$$A_j = 1+\left(\frac{1}{\sqrt{\alpha_j}} - \alpha_j\right)\cdot\frac{2T}{1+3T} \tag{A2.4}$$

$$\alpha_j = \frac{W_j}{W}$$

W: total weight of the building,

T: fundamental period of the building in second. $T=0.03\cdot h$ for moment-resisting steel frames, where h is the overall building height in meter.

C_0: standard shear coefficient, which shall not be less than 0.2 and 1.0 for moderate earthquake motions and severe earthquake motions, respectively.

In this seismic design procedure, dual criteria are used: the buildings are designed elastically to resist moderate earthquakes and to resist severe earthquakes without collapse. In the case of severe earthquake motions, the ultimate lateral shear strength of each story shall not be less than the necessary ultimate lateral shear, Q_{un}, determined in accordance with the following formula:

$$Q_{un} = D_s \cdot Q_{ud} \tag{A2.5}$$

where Q_{ud}:lateral seismic shear for severe earthquake motions given by substituting C_0=1.0 into Eq. (A2.2),

D_s: structural coefficient. D_s=0.3 is used for the ductile moment-resisting steel frame and D_s=0.4 is used for the moment-resisting steel frames which are not meeting special detailng requirements, or the braced frames.

The D_s-values stand for the R-factors in the NEHRP provisions and are estimated by using the equal maximum energy assumption.

These code requirements for seismic loads will be reduced to the working load level for comparison. In the elastic design procedure for the moderate earthquakes, the allowable stress may be increased 1/2 above the values in the presence of seismic loading. Then, the design base shear at the working load level, V_w, is,

$$V_w = (2/3) \cdot Q_1 = 0.667 \cdot Q_1, \quad Q_1 = C_1 \cdot W = R_t \cdot C_0 \cdot W \tag{A2.6}$$

$$C_{sw} = V_w / W = 0.667 \cdot R_t \cdot C_0 \tag{A2.7}$$

Substituting C_0=0.2 and R_t-values in Eq. (A2.3) into Eq. (A2.7) gives,

$$\begin{aligned} C_{sw} &= 0.133 && (T \leq 0.6 \text{ sec.}) \\ &= 0.133 \cdot [1 - 0.2 \cdot (T/0.6 - 1)^2] && (0.6 \text{ sec.} \leq T \leq 1.2 \text{ sec.}) \\ &= 0.128/T && (T \geq 1.2 \text{ sec.}) \end{aligned} \tag{A2.8}$$

In the plastic design procedure for severe earthquakes, taking the ratio of plastic moment to yield moment to be $M_p/M_y=1.14$ for rolled steel beams, the base shear capacity requirement at the working load level will be biven by,

$$V_w = \frac{2}{3} \times \frac{1}{1.14} \cdot Q_1 = 0.585 \cdot Q_1, \quad Q_1 = R_t \cdot C_0 \cdot W \qquad \text{(A2.9)}$$

$$C_{sw} = V_w / W = 0.585 \cdot R_t \cdot C_0 \cdot D_s \qquad \text{(A2.10)}$$

Substituting $C_0=1.0$, R_t-values in Eq. (A2.3), and $D_s=0.3$ into Eq. (A2.10) gives the following expression for the ductile moment-resisting steel frames:

$$\begin{aligned} C_{sw} &= 0.176 && (T \leq 0.6 \text{ sec.}) \\ &= 0.176 \cdot [1-0.2 \cdot (T/0.6-1)^2] && (0.6 \text{ sec.} \leq T \leq 1.2 \text{ sec.}) \\ &= 0.169/T && (T \geq 1.2 \text{ sec.}) \end{aligned} \qquad \text{(A2.11)}$$

Substituting $C_0=1.0$, R_t-values in Eq. (A2.3), and $D_s=0.4$ into Eq. (A2.10) gives the one for the non-ductile moment-resisting steel frames:

$$\begin{aligned} C_{sw} &= 0.234 && (T \leq 0.6 \text{ sec.}) \\ &= 0.234 \cdot [1-0.2 \cdot (T/0.6-1)^2] && (0.6 \text{ sec.} \leq T \leq 1.2 \text{ sec.}) \\ &= 0.225/T && (T \geq 1.2 \text{ sec.}) \end{aligned} \qquad \text{(A2.12)}$$

Only Eqs. (A2.11) and (A2.12) are plotted in Fig. A2-1 to compare with the seismic design loads regulated in the NEHRP provisions, because this plastic design procedure is equivalent to that of the NEHRP. In this figure, the fundamental periods determined by the code requirements are also shown with the number of stories. It can be recognized that the Japanese code requirements are almost double of the NEHRP requirements in CA and have conservative values for the design of low- to medium-rise buildings with the period of 0.6 to 1.2 second. Moreover, in the Japanese seismic code, the design fundamental periods of the building are shorter than those of the NEHRP for buildings that have the same dimensions and configurations.

A2.2 The Wind Design Loads

The design wind pressure in kg/m^2 is determined as follows:

$$q = q_0 \cdot Z_w \cdot L \cdot I \tag{A2.13}$$

where q_0: basic velosity pressure (kg/m^2) given by,

$$\begin{aligned} q_0 &= 120 && (0 \leq h \leq 10 \text{ m}) \\ &= 120+8\cdot(h-10) && (10 \text{ m} \leq h \leq 30 \text{ m}) \\ &= 280+1.1\cdot(h-30) && (30 \text{ m} \leq h \leq 230 \text{ m}) \\ &= 500 && (h \geq 230 \text{ m}) \end{aligned} \tag{A2.14}$$

Z_w: zoning factor for wind pressure,

L: structural size factor. The value of 1.0 is used for normal configuration.

I: importance factor. The value of 1.0 is used for normal office buildings.

A2.3 Comparison of Design Loads

A comparison of the design seismic and wind loads in the U.S. and Japan is given in this section. The design loads for buildings located in CA and NY with accordance to the NEHRP provisions have already been investigated in Chapter 3. The design loads for the 15-, 20-, and 30-story buildings in Japan will be added to compare with the above design loads in the U.S.. Buildings are assumed to be moment-resisting steel frames and their configurations and dimensions used in this comparison are as follows:

F (number of story)= 3, 9, 15 for buildings in CA and NY
= 15, 20, 30 for buildings in Japan

L/B (length-to-width ratio) = 1.0 (3-bay)

h (story height) = 12 ft.

l (span length) = 24 ft.

The dead loads are kept at 75 psf and the live load reduction is assumed to be a constant value of 30%. The analysis methods used in this comparison are exactly same as in Chapter 3.

Fig. A2-2 shows the comparison of design seismic and wind loads for the 15-story buildings in the U.S. and in Japan. It presents that the criteria that govern the building design in Japan are the seismic loading and their absolute values are almost double of those in CA and triple in NY.

Figs. A2-3(a), (b), and(c) give the design story shears for the 30-, 20-, and 15-story buildings in Japan. The wind loads govern the design of several stories in the 20-story moment-resiting frame with D_S=0.3 and almost all stories in the 30-story frames with both D_S=0.3 and D_S=0.4.

Fig. A2-4 shows the ratio of seismic-to-wind design loads, Q_{Ej}/Q_{Wj}, at each level for the SMRSFs with 3, 9, and 15 stories in the U.S. and the moment-resisting frames with D_S=0.3 in Japan. In NY all the wind design loads surpass the seismic design loads except the top story of the 3-story SMRSF. In CA, on the contrary, all the seismic loads surpass the wind loads except a few stories of the 15-story SMRSF. It is recognized that the range of the ratio of seismic-to-wind design loads for the 20-story building in Japan is almost same as that for the 9-story building in CA and that for the 30-story building in Japan is also same as that for the 15-story building in CA, although their absolute values are different. This fact implies that the results of this thesis can be applied to the tall buildings in Japan, located in a high seismic zone.

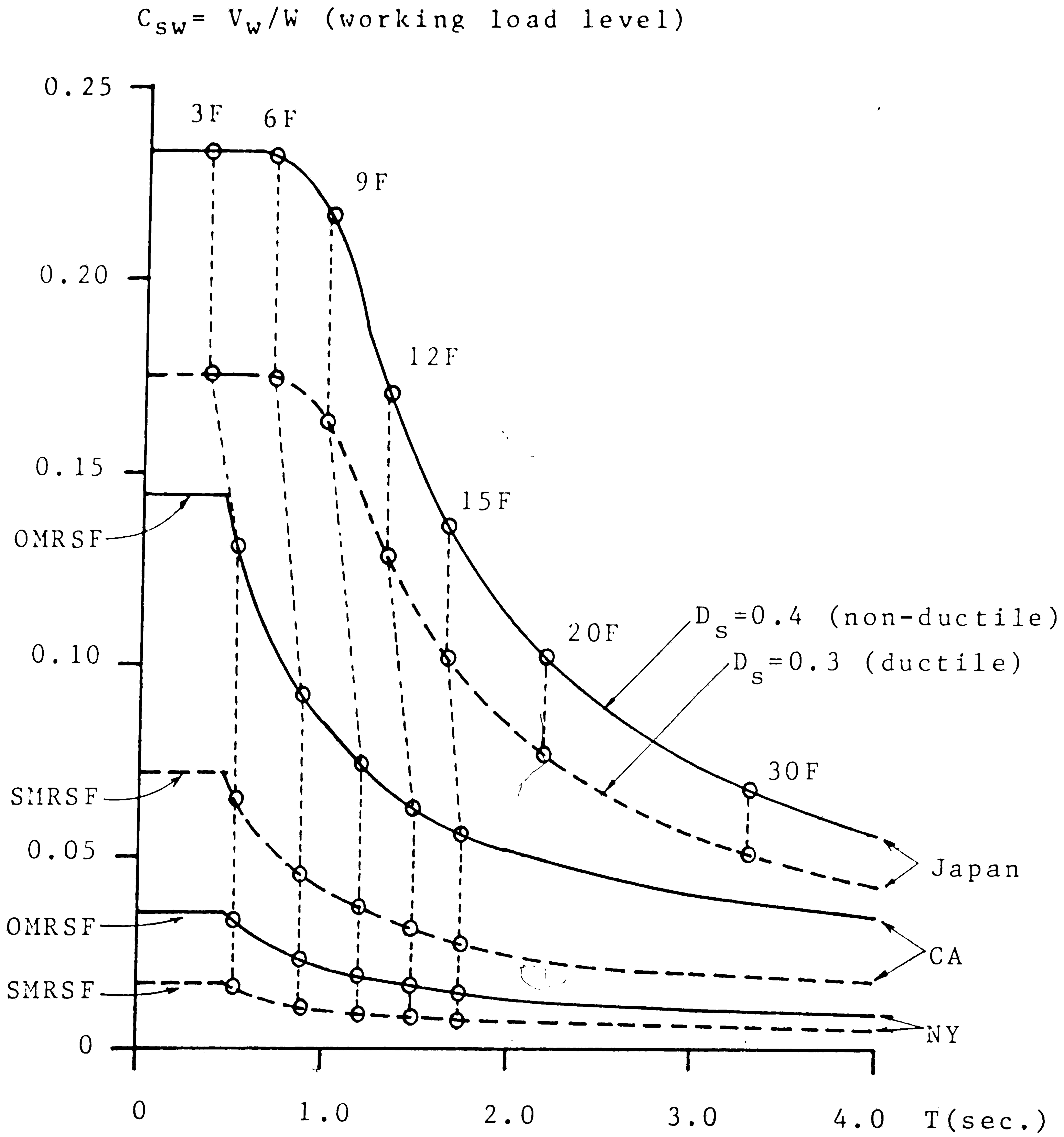


Fig.A2-1 Comparison of design base shear coefficients between the U.S. and Japan

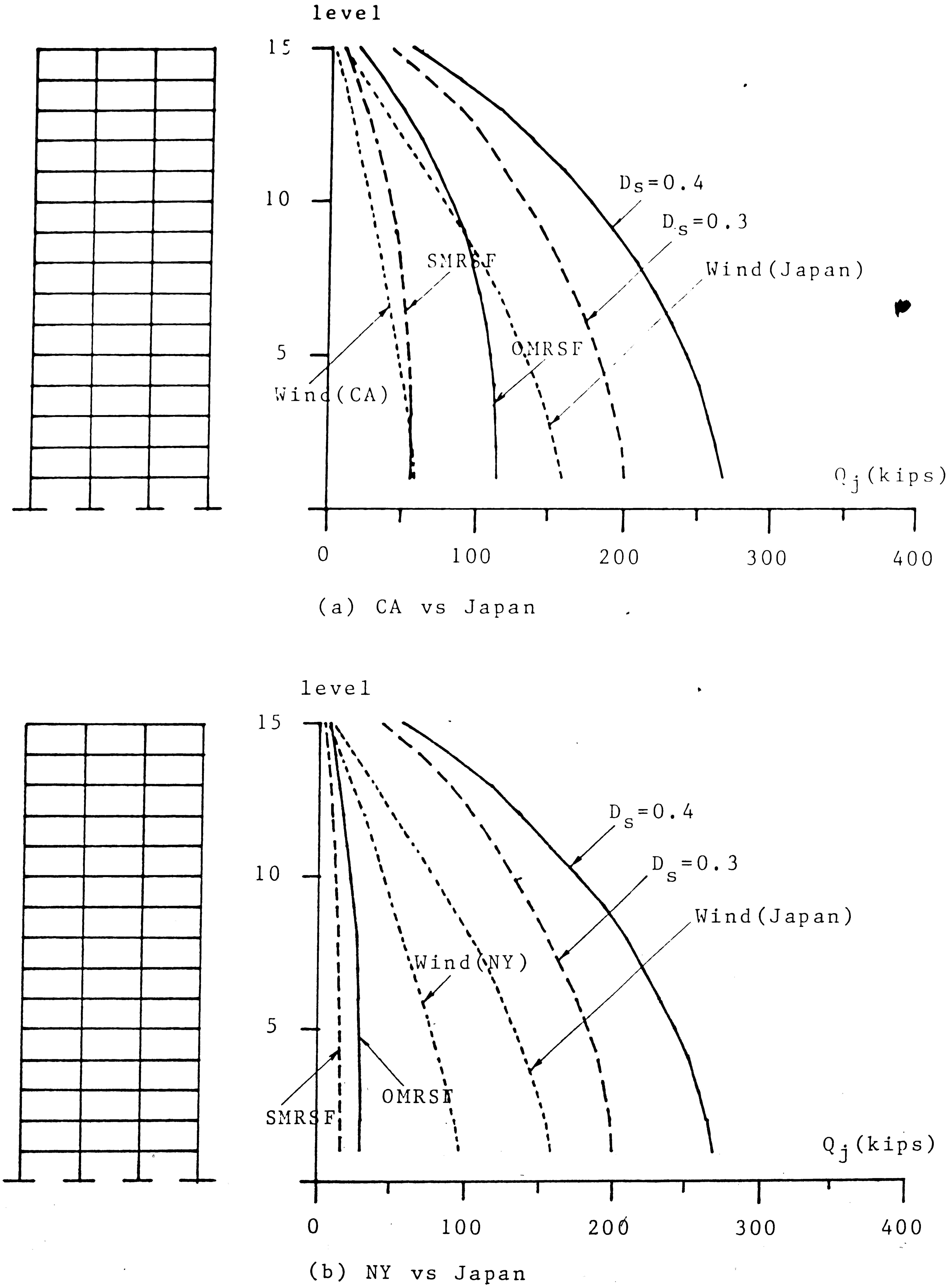
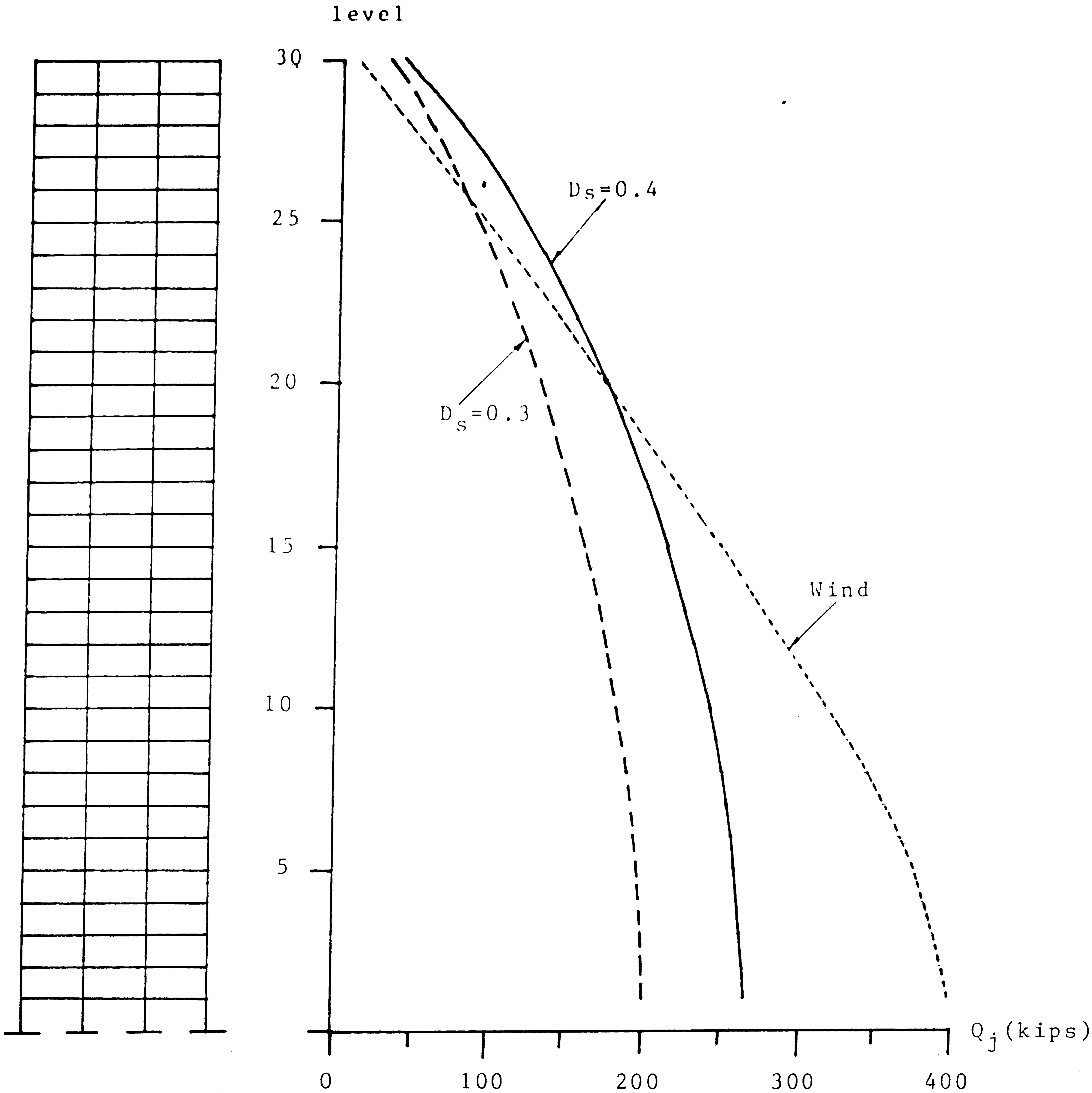
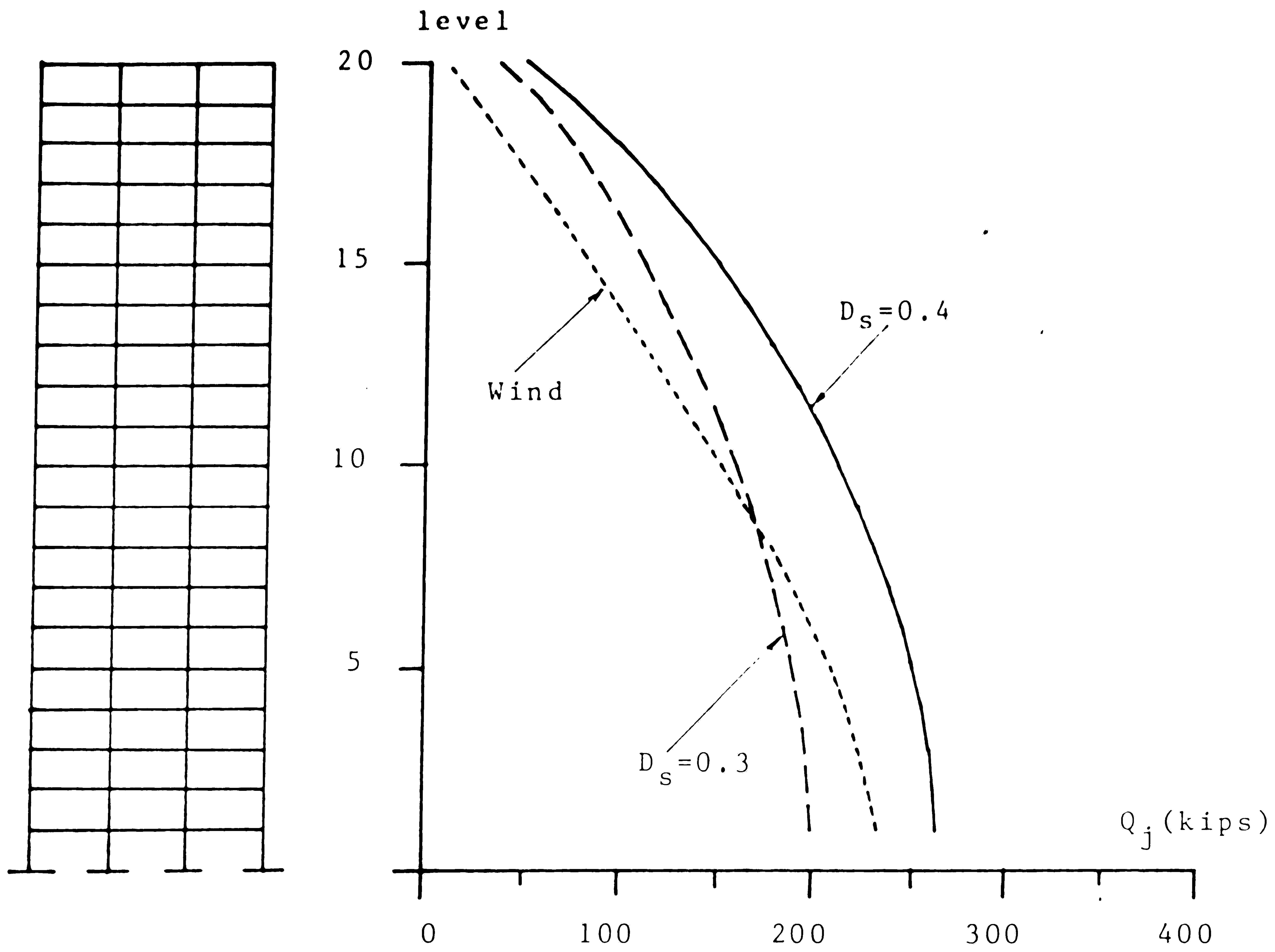


Fig.A2-2 Comparison of horizontal design loads between the U.S. and Japan

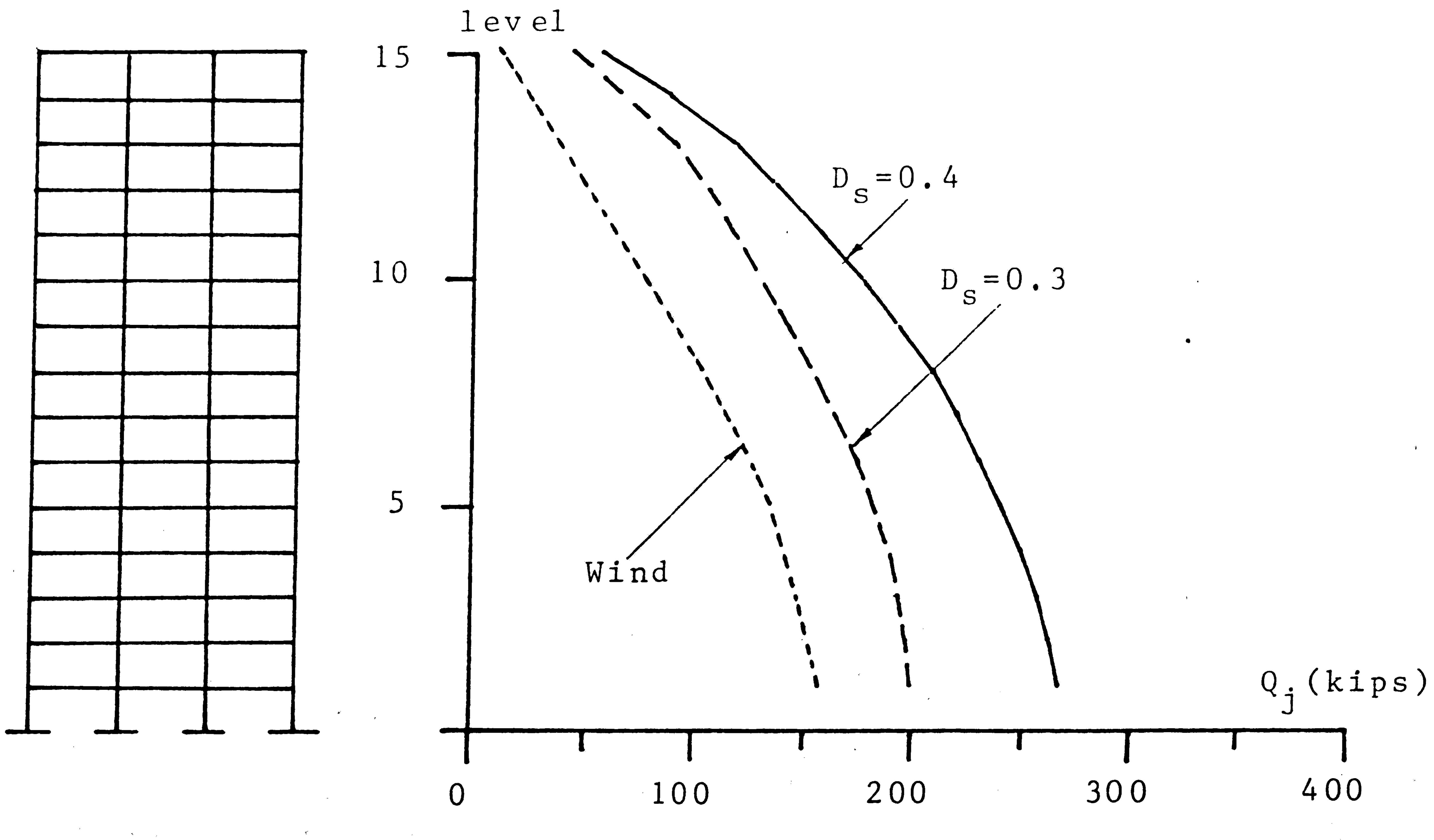


(a) 30-story moment-resisting frames

Fig.A2-3 Horizontal design loads for 15-, 20-, and 30-story buildings in Japan



(b) 20-story moment-resisting frames



(c) 15-story moment-resiting frames

Fig.A2-3 Horizontal design loads for 15-, 20-, and 30-story buildings in Japan

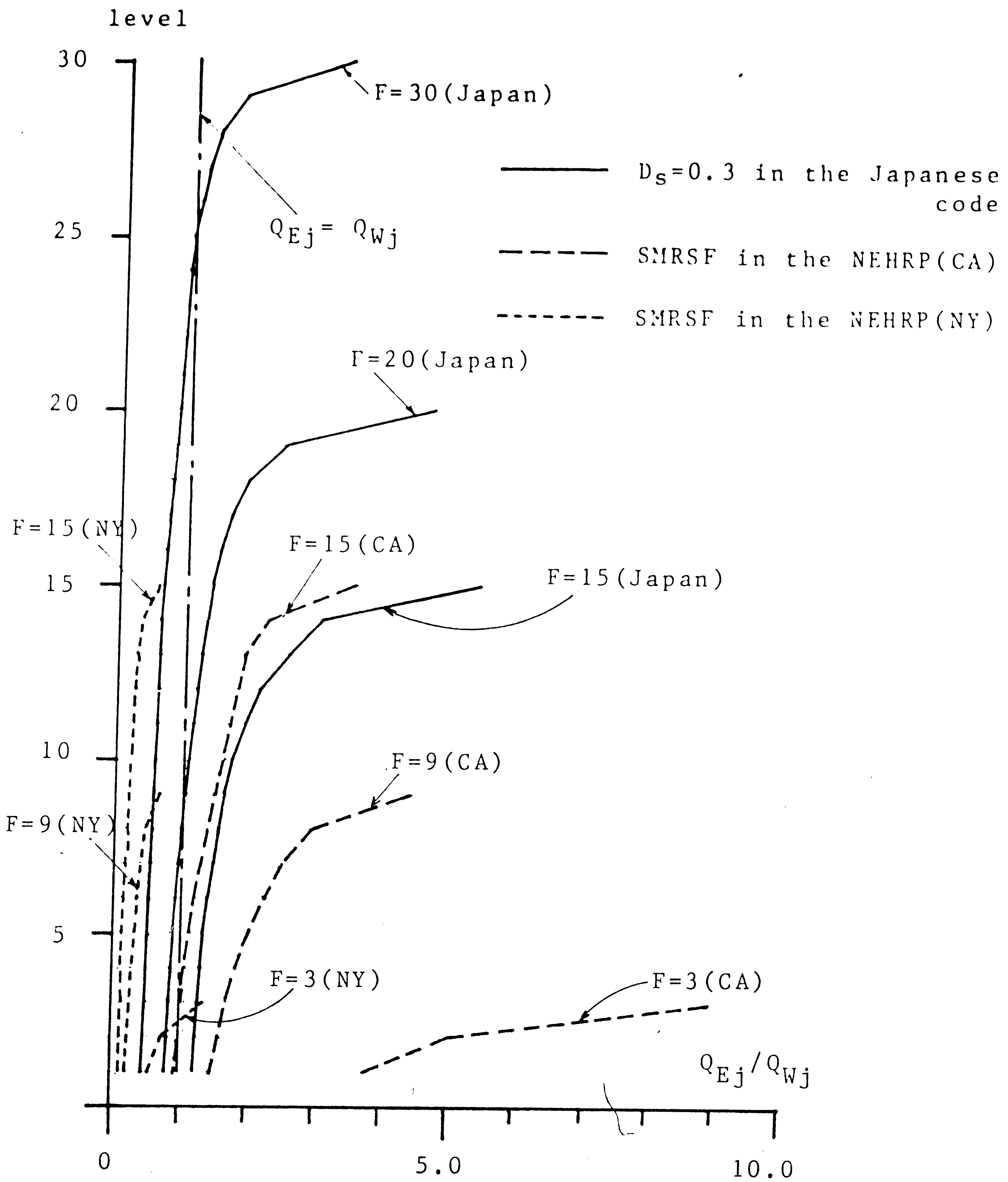


Fig.A2-4 Ratio of seismic-to-wind design loads in the U.S. and Japan

Appendix 3. Ultimate Strength of Buildings Analyzed by Mechanism Method

A3.1 Column Type Sway Mechanism

Referring to Fig. 4-5(a), if a column mechanism occurs at level k, the external work done, W_e, is given by,

$$W_e = \sum_{j=k}^{n} F_j \cdot \theta \cdot h = \theta \cdot h \cdot \lambda_{Ck} \cdot Q_k \quad (A3.1)$$

where λ_{Ck}: load factor for ultimate strength of column type sway mechanism at level k (k=1,2,---,n)

F_j: horizontal force at level j,

Q_k: story shear at level k,

h: story height,

θ: plastic hinge rotation.

The internal work done, W_i, at column hinges is,

$$W_i = \theta \cdot \left(\sum M_{pc} \right) = \theta \cdot C_k \quad (A3.2)$$

where C_k: sum of column plastic moment capacity, M_{pc}, at level k, given by,

$$C_k = 1.18 f_c \cdot [2 \cdot (S-1) \cdot M_{Cij} + 4 M_{Cej}] \quad (A3.3)$$

f_c: shape factor for columns. The value of 1.14 is used for wide flange shapes.

S: number of bays

M_{Cij}, M_{Cej}: yield moment capacity of interior columns or exterior columns.

Using the energy concept, W_e=W_i, and substituting Eq. (A3.3) into Eq. (A3.2) gives the following relationship:

$$\lambda_{Ck} = C_k / (Q_k \cdot h) \quad (A3.4)$$

$$\lambda_C = \text{Min.}[\lambda_{C1}, \lambda_{C2}, \cdots, \lambda_{Cn}] \quad \text{(A3.5)}$$

A3.2 Combined mechanism

Referring to Fig. 4-5(b), under the condition that the combined mechanism is formed above the base to level k, the external work, W_e, is given by,

$$W_e = F_1 \cdot \theta \cdot h + F_2 \cdot \theta \cdot 2h + \cdots + F_{k-1} \cdot \theta \cdot (k-1) \cdot h + \sum_{j=k}^{n} F_j \cdot \theta \cdot kh$$

$$= \theta \cdot h \cdot \lambda_{CBk} \cdot \sum_{j=1}^{k} Q_j \quad \text{(A3.6)}$$

where λ_{CBk} is the load factor for ultimate strength of the combined mechanism at level k (k=2,3,---,n). The computing internal work, W_i, is given by,

$$W_i = \theta \cdot [0.5 \cdot (C_1 + C_k) + \sum_{j=1}^{k-1} B_j] \quad \text{(A3.7)}$$

where B_j: beam plastic moment capacity considering gravity load effects. The plastic hinge at the center of beam in the case of Eq. (A3.8.b) and at the end in the case of Eq. (A3.8.a),

$$B_j = 2S \cdot M_{pBj} \qquad (M_{pBj}/M_{pm} > 4) \qquad \text{(A3.8.a)}$$

$$= 8S \cdot M_{pm} \cdot [\sqrt{(M_{pBj}/M_{pm})} - 1] \quad (M_{pBj}/M_{pm} \leq 4) \qquad \text{(A3.8.b)}$$

$$M_{pm} = w \cdot L^2/16$$
$$M_{pBj} = f_B \cdot M_{Bj}$$

f_B: shape factor for beams. The value of 1.14 is used for wide flange shapes.
M_{Bj}: beam yield moment at level j
M_{pBj}: beam plastic moment at level j

w: vertical distributed load ($w_D + w_L$)

L: span length

From Eqs. (A3.6) and (A3.7),

$$\lambda_{CBk} = [0.5\cdot(C_1+C_k) + \sum_{j=1}^{k-1} B_j] \Big/ \left(\sum_{j=1}^{k} Q_j \cdot h \right) \qquad \text{(A3.9)}$$

$$\lambda_{CB} = \text{Min.}[\lambda_{CB2}, \lambda_{CB3}, \cdots, \lambda_{CBn}] \qquad \text{(A3.10)}$$

A3.3 Beam type sway mechanism

Referring to Fig. 4-5(c), the external work, W_e, is given by,

$$W_e = F_1\cdot\theta\cdot h + F_2\cdot\theta\cdot(2h) + \cdots + F_n\cdot\theta\cdot(nh)$$

$$= \theta\cdot h\cdot[(F_1+F_2+ \cdots +F_n)+(F_2+ \cdots +F_n)+ \cdots +F_n]$$

$$= \theta\cdot h\cdot\lambda_B\cdot \sum_{j=1}^{n} Q_j \qquad \text{(A3.11)}$$

where λ_B is the load factor for ultimate strength. Using Eq. (4.6) and (4.11), the internal work, W_i, is given as follows;

$$W_i = \theta\cdot(0.5C_1 + \sum_{j=1}^{n} B_j) \qquad \text{(A3.12)}$$

From Eqs. (A3.11) and (A3.12),

$$\lambda_B = (0.5C_1 + \sum_{j=1}^{n} B_j) \Big/ \left(\sum_{j=1}^{n} Q_j \cdot h \right) \qquad \text{(A3.13)}$$

8. VITA

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The author was born in Osaka, Japan on February 10, 1955 to the parents Izumi and Fumiko Nagata. He graduated from the Department of Architectural Engineering at Osaka University in 1978. Subsequently, he continued his graduate studies at the same school and majored in steel structures under the instruction of Professor Sadayoshi Igarashi. He was awarded the Master of Engineering degree in 1980.

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[1] Inoue, Kazuo, and Nagata, Masahiro, "*A Study on the Plastic Design of Braced Multi-story Steel Frames - Part 4: on the seismic design load factor of strong column, weak beam, braced frames*", Trans. of the Architectural Institute of Japan, No.283, Sept. 1979.

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