BUILDING CODE PROVISIONS FOR ASEISMIC DESIGN

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This paper undertakes to discuss in a very brief manner the history of earthquake codes in the United States and the development of a proposed Uniform Seismic Code for California by the Structural Engineers Association of California. Although consideration is given in this discussion to seismic development only in the United States, it must be stated at the very outset that the committee which formulated the proposed code for California was aware of the research and excellent theoretical work on earthquake engineering accomplished, and of the seismic codes established, in other parts of the world. The writers acknowledge the influence of these studies upon the deliberations of the California committee, and they are especially pleased to express appreciation of the work in the seismic field of the host country for the Second World Conference on Earthquake Engineering.

The year 1925, because of the impetus given by the Santa Barbara earthquake, may be considered as marking the real beginning of earthquake studies and research in the United States. It was in this year that, by direction of the United States Congress, the United States Coast and Geodetic Survey was given the responsibility to make investigations and reports on seismology. The work of the U. S. Coast and Geodetic Survey, in particular the publishing of strong motion earthquake records, was to have a significant influence on the development of codes.

The first edition of the Uniform Building Code of the Pacific Coast Building Officials Conference (now known as the International Conference of Building Officials) was published in 1927 and contained in the appendix a chapter on earthquake provisions, planned for optional use.

In 1928, the California State Chamber of Commerce recognized the need for a building code which would afford protection against earthquake damage through the inclusion of a section on seismic design. This project evoked wide-spread and active interest and stimulated studies and research by engineers on the subject of earthquake resistant design. The resulting requirements for the design of structures in seismic areas provided for greater security and sounder investment to the public than had been previously offered.

The first mandatory seismic codes used to any extent in the United States were published in 1933 following the March 10, 1933, Long Beach (Southern California) earthquake.(1) As a consequence of the Long Beach earthquake, two California State laws were passed.

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The Field Act, which placed complete control of structural design over public school construction in the Division of Architecture, California State Department of Public Works, authorized the Division of Architecture to approve or review all public school plans and specifications and to furnish general supervision of the work of construction. This law became effective as an emergency measure on April 10, 1933. The Division of Architecture at the same time adopted Rules and Regulations, including requirements for seismic analysis. The excellent leadership of the Division of Architecture was an influential factor in stimulating a genuine interest in seismic design and in a better understanding of design for seismic loading.

The second law, the Riley Act, which became effective May 26, 1933, made provision for design and construction to resist seismic or wind forces and was formulated for more general application than the Field Act which applied to schools only. Exempted from the Riley Act were (1) buildings located outside the limits of an incorporated area and not intended for occupancy by human beings, and (2) dwellings outside the limits of an incorporated area and planned for not more than two families. The Riley Act specified a seismic design coefficient of 2% of the design load which for most situations would mean approximately 2.5% of the design dead load. (Minor modifications of this legislation have been made from time to time.)

In addition to the State legislation, local codes were adopted in 1933 by the County of Los Angeles, the City of Los Angeles, the City of Long Beach, and other municipalities primarily in Southern California.

While there was general agreement on basic seismic matters among most California structural engineers, the development of codes after 1933 progressed along somewhat different lines in Northern and Southern California. The principal seismic codes in use following 1933 will be discussed later in the paper.

In 1957, the Seismology Committee of the Structural Engineers Association of California, at the direction of its Board of Directors, and under the chairmanship of William T. Wheeler, undertook to develop a uniform seismic code which would, in the main, resolve the important differences in the several codes used in seismic areas of the United States and particularly in California. To accomplish this objective, committee members of the Central, Northern, and Southern Associations of the Structural Engineers Association of California worked for two years, pooling their knowledge and experience in seismic matters in order to develop a uniform code.

The Committee in its early deliberations soon realized that, because of the many unknowns involved, a seismic code should be concerned primarily with establishing certain minimum ground rules and general provisions. It was expected that these would provide as uniform a degree of earthquake protection as possible for the dynamic response of structures which vary because of differences in soils, plan, size, height, and type of construction. It was concluded that it would be unreasonable and economically unjustifiable to attempt to impose a design planned to achieve structures which would survive even the strongest earthquake without any damage. Therefore, the Committee adopted as one of its objectives the development of a seismic code which would confine its provisions to limiting the extent and type of property damage which endangers health and safety. It was also agreed that a commentary on the Code, known as a "Manual of Practice," should complement it, presenting items of detail to supplement and explain its broad considerations.

The Committee was cognizant of some, if not most, of the many variables which can influence the dynamic response of a structure to an earthquake ground motion. It realized that mere compliance with seismic code requirements would not of necessity produce a sound earthquake resistant design since the failure of an engineer to understand and appreciate fully the dynamic effects of earthquakes could result in an inadequate design. The Seismic Code of the Structural Engineers Association of California and the Manual of Practice cannot and do not attempt to provide the specific answers to seismic problems which vary from structure to structure. If the variables and their influences were known, structures could be designed in accordance with the principle of mechanics, and the Code would probably take a different form. The development of design cannot therefore be entirely analytical but must depend upon the designing engineer's experience, skill, and sound judgment in earthquake matters. It is expected that the responsible engineer will realize the dynamic nature of the problem, the complex distortions which a structure may undergo, and the related phenomena, all of which cannot be covered in specific detail by a building code. Therefore, the design of many seismic structures should become the responsibility of those experienced in the art of seismic design. This responsibility is properly assigned to those who are equipped with the knowledge and understanding of seismic research work, of special design techniques, and of the damage effects of earthquakes, particularly for the locality involved, which in our case is the United States. These concepts are expressed in the following excerpts from the foreword of the Manual of Practice:

"This manual is not intended to be a compendium of all knowledge on earthquake design, nor is it a handbook for the inexperienced engineer. The earthquake phenomenon is complex in its simplest manifestations and codes providing minimum design criteria are simplified of necessity. The 'Recommended Lateral Force Requirements' are not intended to be applied as a substitute for sound engineering judgment. The understanding, analyses, and good judgment of the experienced structural engineer in the field of engineering seismology must be relied upon for the sound and economical design and construction of specific structures. This despite the recognition of spectacular advances in knowledge in recent years, particularly in the theoretical understanding of the earthquake phenomenon."

In formulating the Structural Engineers Association of California Seismic Code, study groups from the three regional Associations were delegated the tasks of developing background material through consideration of all available research and earthquake damage studies, and of making recommendations which would become the springboard for discussion of the Committee as a whole. Separate study groups were assigned the subjects of: Base Shear, Structural Frame, Shear Distribution, Diaphragms, Torsion, Overturning, Foundations, Set-Back, and Drift.

Inasmuch as the work of the study groups was carried on to a large extent by correspondence, and since there was an inter-relationship of the various topics, a liaison member for each of the three Associations--Northern, Central and Southern--was appointed to see that pertinent information was continually circulated among Committee members in order to avoid later conflict. When it was felt that the work of the study groups had progressed to the point of making specific recommendations, a meeting of the entire Committee was called in order to consider the findings of the various study groups.

The Committee was originally made up of sixteen members and was supplemented later by twenty members appointed to study groups. Many well qualified individuals were consulted from time to time, and thus specialized knowledge was given consideration in the deliberations and conclusions of the Committee.

In order that a sound and logical code might be developed, the Chairman cautioned the Committee not to be influenced by repeated statements which may have become cloaked with apparent authority and which tended towards a bias from which an expedient solution might be rationalized but without proper basis..

In undertaking to develop the Structural Engineers Association of California Code, the Committee gave consideration to the seismic portions of the following codes:

1. Uniform Code of the International Conference of Building Officials adopted in 1946. (This was the same code as adopted by the City of Los Angeles in 1943 and the California State Division of Architecture in 1953.)

2. Joint Committee of San Francisco Code, published in the 1952 Transactions of the American Society of Civil Engineers.[2]

3. City of San Francisco Code, adopted in 1954.

4. City of Los Angeles Code, adopted in 1957.[3]

The Base Shear "C" coefficients, in per cent of dead load "W", established by these codes are as follows:

1. Uniform Code:

$$C = \frac{60}{N + 4.5}$$

where N = number of stories above the story under consideration. Thus for a one story building, N = 0 and C = 13.3%; for a 13 story building, N = 12 and C = 3.65%; for a 21 story structure, N = 20 and C = approximately 2.5% which is the minimum coefficient permitted by the California Riley Act.

2. Joint Committee Code: C varies from 6% to 2%. Since this code uses dead load plus 1/4 live load, the coefficient for dead load only will vary from approximately 6.6% to 2.2%.

3. City of San Francisco Code: C varies from 7.5% to 3.5% based on dead load plus some percentage of live load; for dead load only, C would be approximately 8.2% to 3.8%.

4. City of Los Angeles:

$$C = \frac{4.6S}{N + 0.9(S-8)}$$

where S = the total number of stories and N = number of stories above the story under consideration. For 1 to 13 stories S is used as 13. Thus, for this range,

$$C = \frac{60}{N + 4.5}$$

and C = 13.3% for 1 story structures; 3.65% for a 13 story structure as discussed under Item 1, Uniform Code. For structures over 13 stories, say 30 stories, C = 2.8%.

It may be of interest to discuss briefly the development of the City of Los Angeles Code value of

$$C = \frac{4.6S}{N + 0.9(S-8)}$$

Prior to 1957, the City of Los Angeles had a height limit of 150 feet, with a maximum of 13 stories above the ground. In 1957, the height limit of 150 feet was removed and the Structural Engineers Association of Southern California was asked to develop and present for consideration by the City building authorities a revised seismic code for use in connection with structures of unlimited height. The Association decided to retain the original code value of

$$C = \frac{60}{N + 4.5}$$

up to 13 stories and to develop a formula for C which in the taller structures would approach triangular distribution for the shear load diagram. It should be noted that this type of formula not only sets the base shear value but also establishes the shear distribution.

Triangular distribution

$$F_x = \frac{Vw_xh_x}{\sum wh}$$

was first suggested in the Joint Committee report. For uniform story masses and uniform story heights, the force diagram values of F_x for the loading diagram will vary uniformly from zero at the base to a maximum at the top and is a true triangle. It is noted that the formula

$$C = \frac{60}{N + 4.5}$$

is a hyperbolic function with the constant "4.5" forming one asymptotic line and the constant "60" representing the degree of curvature or general configuration of the hyperbola. For the formula

$$C = \frac{4.6S}{N + 0.9(S-8)}$$

S is taken as 13 for structures 1 through 13 stories and in this range yields

$$C = \frac{60}{N + 4.5}$$

For structures above 13 stories, the objective of engineers of the Los Angeles City Building Department who developed the formula was to approach triangular distribution values in taller structures by using only that portion of the hyperbolic function that is essentially linear throughout the range where such values are desired. The resultant values of C achieved the original objective quite satisfactorily for structures above approximately 13 stories.

To this point we have discussed the base shear seismic coefficients as used by existing United States codes which vary from 13.3% down to approximately 2.5% of dead load. Many, if not most, of the Committee members were thoroughly familiar with the long and heated debates which had taken place over the years as to the magnitude of the seismic factor to be used and the degree of damage to be guarded against. It was known that the magnitude advocated by some engineers often varied with their degree of understanding of the problems involved. In addition, the use of the dynamic approach was questioned by those who advocated one seismic coefficient for all structures.

The Committee realized that the lateral forces, arrived at from the base shear coefficients specified by a code, do not necessarily represent the actual inertia forces which might be acting during an earthquake. It was also understood, from elastic analyses that have been made, that for a strong earthquake the stresses may be of greater magnitude than those indicated by the code requirements. However, it was concluded by the Committee that, from the study of earthquake damage in the United States and from other findings, the range of coefficients in existing codes was realistic and in accord with the objectives set by the Committee.

On the basis of damage studies and the continuing research on the spectral response of resonators,(4) it was agreed that the time period of the responding structure is a significantly important parameter in determining the base shear coefficient. It was concluded that formulas using the number of stories as the variable for a measure of period did not encourage development of research in the area of determining realistic periods. In electing to compute the fundamental period in terms of the building dimensions, the Committee members, even those who originally developed the empirical formula, were aware of the short-comings of this method. However, it was felt that if the Code were based on a period concept, new and better methods of determining periods of a structure could and would be developed.

It was the opinion of the Committee that base shear coefficients, predicated on a dynamic approach in its static equivalents, should include the consideration, insofar as practical, of the response performance of different types of construction along with considering the varying degree of flexibility as well as the damping, ductility, and energy absorption capacity of various types of construction. It was expected that this approach, plus the consideration of earthquake damage studies, could result in as uniform a degree of earthquake protection as possible.

The Base Shear Study Group and the Structural Frame Study Group separately had concluded that the use of a single formula for computing the coefficient "C" for all types of structures, did not give effective consideration to completely different construction categories. In considering the various categories, they recommended a modifying factor for the coefficient "C", where "C" would be increased or decreased on the basis of the structure's energy absorption capacity and ductility; credit would also be given to structures having a second line of resistance. These recommendations were studied by the Committee, and after long and penetrating discussion, it was decided to use a factor to modify "C" in order to provide a coefficient which would vary with the type of construction. In other words, while the total base shear "V", induced into the base of a structure from an earthquake ground motion, had previously been established as $V = CW$, the new proposal would establish $V = KCW$ where K represents the modifying influence of the response performance of different types of construction with varying degrees of damping, stiffness, ductility, and energy absorption.

Base Shear coefficient "C" formulas have been stated as

$$C = \frac{60}{N + 4.5}$$

for the Uniform Code and

$$C = \frac{4.6S}{N + 0.9(S-8)}$$

for the City of Los Angeles Code. It should be noted that in using these formulas base shear coefficients vary with the number of stories. For the Joint Committee Code, the formula for C is 1.5/T and for the City of San Francisco Code, the formula for C is 2.0/T. For the City of San Francisco Code, periods of 0.27 seconds and 0.57 seconds established the coefficient cut-off values of 7.5 and 3.5% respectively. It was considered that the plateau values which established maximum and minimum "C" coefficients, with a narrow range of periods for the transition, would not provide a consistent relationship to express adequately the response performance of structures varying in periods from 0.10 seconds to 2.0 seconds and over. The Base Shear Study Group, after considerable study, made several proposals for determining "C" as a function of the period "T", and the formula finally accepted by the Committee was

$$C = \frac{5}{\sqrt[3]{T}}$$

with a maximum basic cut-off coefficient of 10%.

Figure No. 1 shows the curves for four of the codes discussed, including the Structural Engineers Association of California Code. It is believed that the new Structural Engineers Association of California Code lessens or eliminates some, if not many of the objections to the other codes. Figure No. 2 shows a nomograph for the determination of Period and Coefficient "C" with

$$T = \frac{0.05H}{\sqrt{D}} \quad \text{and} \quad C = \frac{5}{\sqrt[3]{T}}$$

Mention has been made that the response of structures having varying degrees of damping, stiffness, and energy absorption would be recognized in the final evaluation of a base shear coefficient "C". This recognition, in part, is accomplished through the use of a modifying factor designated "K". In its early deliberations, the Committee considered numerous types of construction and types of structures. However, in order to reduce the complexity of the problem, it was decided to limit K values to four types of building construction which are most prevalent. It was agreed that further studies would be necessary in order to consider any other satisfactory structural framing system not presently covered by the Code.

"K" values were established for the various types of structures and bracing systems as follows:

(a) K = 1.0 is established for structures having a complete "Vertical Load Space Frame" designed to carry all vertical loads, dead and live, and with shear walls or other vertical resisting bracing systems which are designed to resist all lateral forces. The vertical load space frame is not required to be designed to resist any percentage of lateral forces; however, while vertical load space frames need not be designed as moment resisting for lateral loads, it is anticipated that many engineers will take advantage of the frame to achieve a certain nominal amount of lateral stiffness.

(b) K = 1.33 is established for "Box Type" structures which do not have a complete vertical load space frame capable of carrying all vertical loads. Vertical and lateral loads may be carried by various combinations of load-carrying framing, bearing walls, shear walls and vertical bracing systems without a self-contained vertical load carrying frame. Stated another way, if some of the vertical loads were carried by or relied on being carried by bearing walls, the structure would not have a complete framing system for vertical loading.

Structures under (a) and (b) above are limited to 13 stories or not to exceed 160 feet. Periods for these structures may be computed from the building dimensions as previously stated,

$$T = \frac{0.05H}{\sqrt{D}}$$

The Code states that for the purpose of computing "C", the value of "T" need not be less than 0.10 seconds and the maximum basic value of C for one and two story buildings need not exceed 10%, thus KC has a maximum of 10% x 1.33 for the Box System described under (b).

(c) K = 0.80 is established for a structure having a complete Moment Resisting Space Frame capable of resisting at least 25% of the total base shear "V". Even though the frame is capable of taking 25% of the total lateral load under this category, 100% of "V" must be designed into the total structure with all of the lateral load resisting systems participating in accordance with the principle of relative rigidities. The 100% lateral force resistance may be taken by shear walls, vertical bracing or various combinations of vertical lateral load resisting systems. The Committee felt that the 25% capacity in the moment resisting space frame would, through the use of the ductile framing, become an important second line of defense in the event of an unusually strong earthquake. For the 100% lateral resistance, periods may be computed from the building dimensions, and this same period should be used for computing the 25% on Moment Resisting Space Frames; however, the design of the frame does not require consideration of rigidities other than those of the bare frame.

(d) K = 0.67 is established for structures in which the total lateral force "V" is capable of being resisted by a Moment Resisting Space Frame in which the frame has the necessary ductility. Under this system of framing, T may be used as 0.10 seconds per story provided the frame is not enclosed by or adjoined by more rigid elements which would tend to prevent the frame from initially resisting the lateral forces. When designing tall slender structures under this category, it is important that the engineer control the lateral deflections or drift of a story relative to its adjacent stories.

"K" values noted in paragraphs (c) and (d) apply to structures of all heights having a moment resisting space frame. However, it should be noted that for structures over 13 stories or 160 feet a moment resisting space frame is mandatory, and the appropriate K values of 0.80 and 0.67 apply. The moment resisting space frame is required to be made of a ductile material or a ductile combination of materials. "The necessary ductility shall be considered to be provided by a steel frame with moment resisting connections or by other systems proven by tests and studies to provide equivalent energy absorption."

Figure 3 shows curves for the four values of K discussed above.

To this point consideration has been given to base shear coefficient in terms of the response of the total structure (resonator) to a random and chaotic ground motion. The Base Shear Committee prepared lateral force calculations for three types of construction: Uniform Stiffness, Uniform Drift, and Bending Type. The spectrum used was for a ground motion modified from the strongest motion recorded in the El Centro earthquake in 1940. Results were obtained from the first mode only, and base shear coefficients were determined as a function of the period of the structure. The Committee recognized the influence on the period of a structure, of mass distribution and mass used for structural and non-structural purposes. It was concluded that if the computed base shear used depends in a large measure on the natural period of a structure, some reasonable method of calculating the period must also be used. It should be noted that insofar as periods are concerned, while arbitrary methods are set forth for obtaining the period "T", the Code provides that "Properly substantiated technical data for establishing the period T for the contemplated structure may be submitted." It is considered that the two empirical formulas for period

$$T = \frac{0.05H}{\sqrt{D}}$$

for all structures other than a complete moment resisting space frame, for which T = 0.10 seconds per story, are probably sufficiently conservative, so that when used in connection with the formula

$$C = \frac{5}{\sqrt[3]{T}}$$

minor differences in the period of the structure will not greatly influence the value of "V". In this connection it may be noted that a conservative value of T is the minimum value because under the inverse relationship between T and C, the maximum values of "C" are thus developed.

In establishing base shear coefficients which are viewed as arbitrary values, the Committee considered among other things the geological conditions in seismic areas, the soils on which the foundation of the structure is located, the rocking or yielding of the foundations, and the influence of near and distant epicenters with respect to various types of structures. The Committee recognized the importance of soils as a variable in establishing base shear coefficients. Needless to say, many earthquake damage studies, charts, and diagrams, as well as reviews of research and theoretical work were discussed in detail before finally establishing the Code's "KC" coefficients. It remains to be acknowledged that the intangible factor of engineering judgment, of necessity, influenced the ultimate decision.

The subject of the distribution of base shear into equivalent static forces to be concentrated at each story has been discussed briefly under the subject of Base Shear. Admittedly, if the deflected position of the structure were computed at any given instant, the equivalent static forces would follow. The concept here is that the dynamic force (in the elastic range) attributed to any mass is proportional to the mass and its deflection. It was agreed that this would be a reasonable position if the period were computed from a deflected position. It is not the purpose of this paper to discuss the many proposals for approximating the distribution which were made by the Committee. Therefore, only the final Code requirements are given. In the main, Triangular Distribution,

$$F_x = \frac{Vw_xh_x}{\sum wh}$$

as previously discussed, is specified for most structures with the following exceptions: (1) one and two story buildings shall have uniform distribution, and (2) where the height to depth ratio of a lateral force resisting system is equal to or greater than five to one, 10% of the total force "V" shall be considered as concentrated at the top story; the remaining 90% shall be distributed as provided by the formula

$$F_x = \frac{Vw_xh_x}{\sum wh}$$

Exception (2) was, in the main, planned to give consideration to structures in which the total lateral resistance is designed around the structure's center service core. Although the Committee recognized the comparative low polar moment of inertia and the possible significant accidental torsional stresses for this type of design, it was felt that exception (2) was probably reasonably sound since this type of structure will deflect primarily in bending. Under the bending type of deflection, the forces move further up the building than is indicated by the triangular distribution formula.

The Code provides that "Every building shall be designed and constructed to withstand minimum total lateral seismic forces assumed to act non-concurrently in the direction of each of the main axes of the building in accordance with the following formula: V = KCW." The general provisions of the Code provide that the lateral "force shall be assumed to come from any horizontal direction."

Having established the criteria for the response of the total structure (resonator) to an unknown earthquake ground motion recognized to be chaotic and random, the Committee next took under consideration the problems of the design details which tie a structure together in such a manner as to cause it to vibrate as a unit.

To tie a structure together in a horizontal plane, the Code provides that floors and roofs, as well as horizontal trussing systems, acting as diaphragms, shall be designed for a minimum value of 10% applied to loads tributary from that story unless a greater coefficient of KC is required by the basic seismic formula V = KCW. The minimum basic coefficient of 10% for diaphragms was predicated in part on studies which indicated that the shear coefficient for the load of each story does not necessarily decrease as rapidly as the total seismic shear coefficient at the story as given by the basic seismic formula. The Code further provides that the horizontal shear in any horizontal plane shall be distributed to the vertical resisting elements in proportion to their rigidities. Diaphragms for design purposes are also required to have sufficient strength and stiffness to redistribute effectively the shears other than those developed from the design load or masses at the story under consideration. Examples of such other shears are the accidental torsional shears, and shears in stories at set-backs and at other points where the vertical resisting elements are discontinuous. The Committee recognized that diaphragms, including equivalent bracing systems, of given strength, could have varying degrees of flexibility and that the engineer should take into account the load-deflection characteristics of a particular diaphragm system in considering his design. It was agreed that for certain structures, and particularly for tall structures, a stiff diaphragm should be required. The Diaphragm Study Group recommended that stiff diaphragms should be considered as those having load-deflection characteristics no more limber than those of commonly used diaphragms capable of sustaining, with adequate rigidity, the applied loads, vertical and lateral, to which they may be subjected. Examples of acceptable diaphragms are 2000 psi, reinforced concrete floor slabs, or all welded type of steel decks which utilize two sheets of light gage material with one in a continuous plane and for which satisfactory test results have been recorded.(5)

The distribution of horizontal shears to the vertical lateral load resisting systems in proportion to their relative rigidities presumes, among other things, that the diaphragm system is sufficiently rigid to accomplish this distribution. Experience and research has pointed out the difficulty of accurately estimating the stiffness of a number of types of lateral load resisting systems. Admittedly, it is possible by analytical methods to over-estimate the stiffness of a vertical lateral load resisting system. Thus the designer has the problem of assigning shear to the resisting systems, giving consideration to the flexibility or stiffness of the horizontal diaphragm and to the possibility of either over or under-estimating the rigidities of the vertical resisting elements. The Committee recognized that research and studies as to the deflections of diaphragms and vertical resisting elements should continue.

Previous codes in the United States have not made provision for including the effect of torsion other than for a specifically built-in eccentricity. The Torsion Study Group recognized the importance of accidental torsion, caused in part by unbalanced dead loads, live loads, openings in floors, the application of forces, and many other factors.(6) The Study Group's report stated that, in addition, there may be torsional forces induced from floors above or below the one under consideration. Further, differences between the assumed and true stiffness of resisting walls may be considerable. To provide for these additional forces, a conservative "C" factor for diaphragms should be required. The Code, in order to allow for accidental torsion which may result from various situations, provides ". . . . . the shear resisting elements shall be capable of resisting a torsional moment assumed to be equivalent to the story shear acting with an eccentricity of not less than five per cent of the maximum building dimension at that level."

Seismic codes prior to 1952 recognized the "overturning" problem to a varying degree, and as a result, designs for seismic overturning moments may have been handled in several different ways by designers. Following 1952, the San Francisco and Los Angeles Codes developed positive requirements for computing overturning moments.

The Committee agreed that for tall buildings the first mode is most important for overturning. The seismic coefficients and shear distribution, in part, take into account the higher modes for local shears so that the coefficients would be overly liberal for computing overturning. Accordingly, the Code introduced a modifying factor reducing the overturning moments with an increase in the period of the structure. The Code provides that the modifying factor J shall be not less than 0.33 nor more than 1.00, where

$$J = \frac{0.5}{\sqrt[3]{T^2}}$$

Thus structures with periods approximately two seconds and over will have a reduction of 2/3, with no reduction permitted for structures with a period of approximately 0.35 seconds or less. Possibly of greatest concern to the Committee was the influence of overturning moments on the foundations. Much of the background for the requirements on overturning in the Structural Engineers Association of California Code will be covered in a paper, "Design Criteria for Shear and Overturning Moment," to be presented at this conference by John E. Rinne.

The Study Group on Foundations, while submitting several reports, concluded that it did not expect to develop results that could be codified at this time. Due to the fact that earthquakes vary and that each earthquake will further vary at different site locations, more records (e.g. data for building coupling, etc.) are needed for study by the profession before any general conclusions may be drawn. The Study Group stressed that it was important for the structural engineer, in collaboration with a soils consultant, to give studied consideration to the type of foundations which would be best suited for any major structure. Also such special consideration is needed for other structures located on sites with unusual soil conditions. It was suggested that the Code might provide a nominal increase for the seismic soil bearing values on most soils but exclude those in the classification of loose sand, silt, and other loose materials on which no increase would be permitted. Deviations from these proposed arbitrary code values could be permitted on the basis of special foundation investigations made by a recognized soils consultant. The Study Group stressed the importance of considering most carefully possible wide differences in settlements resulting from soil bearing values used for normal loading when compared to bearing values used for short time seismic loading.

Provision has been made in the Code requiring the interconnecting of piles by ties capable of carrying tension and compression, unless it can be demonstrated that equivalent restraint can be effected by other approved methods.

The Set-Back Study Group presented a report which the Committee considered dealt with the problem of set-backs in a logical and detailed manner. The complicated dynamic situations which are inherent in developing code requirements for the many probable combinations of set-backs were discussed. Because of the difficulties involved in trying to present the detailed information in code form, it was decided to defer action on this report until a later time. Meanwhile, the Committee decided to adopt the provisions of the Los Angeles City Code which are, in general, similar to the San Francisco City Code requirements. The Code as adopted stipulates that buildings having set-backs with plan dimensions 75% or more of the corresponding base plan dimensions need not be treated as set-backs. Where the tower (set-back) dimensions are less than 75% of the base dimensions, the tower portion is required to be designed as a separate structure unless the seismic coefficient at the base of the tower is greater for the tower when considered as part of the overall structure. The Manual of Practice will present the report of the Set-Back Study Group.

The Drift Study Group concluded that research was not sufficiently advanced or comprehensive to justify specific detailed requirements for the Code. It was pointed out that, over the years, drift has been a part of design left to the judgment of the designing engineer and that drift limitations should vary for types of construction and also for the special features in a given type of structure. In general, drift requires control when (a) wind causes sway and results in discomfort to occupants of a building, and (b) when an earthquake might cause brittle non-structural elements and fragile materials, such as glass, to fail and become hazardous. It was agreed that Drift provisions should be concerned primarily with lateral wind forces which are of frequent occurrence as compared to infrequent strong earthquakes. It was felt that when drift due to wind forces is kept within the comfort zone, the earthquake drift will probably be within the safety zone, except in special cases. With these conclusions accepted, the Committee did not write into the Code any specific drift requirements but did include the general statement that "Lateral deflections or drift of a story relative to its adjacent stories shall be considered in accordance with accepted engineering practice." It is known that engineering considerations would require the limiting of story drift for tall slender structures, particularly for lateral wind forces. In addition, for this type of structure, it would be most important to use a method of analysis which would take into account the lengthening and shortening of columns. The Committee took the position that the

Code requirement on drift is intended as a caution to the engineer to consider the magnitude of lateral deflections insofar as these deflections may be a factor in achieving a satisfactory structure over and above consideration of strength requirements alone. The Committee realized that damage control due to deflections could be handled in several different ways. For example, brittle elements may require a design with sufficient clearance to accommodate calculated deflections, and exterior wall panels and glazing can be mounted with sufficient clearance to allow for design deflections. (Of particular interest on this subject is a paper, "Drift Limitations Imposed by Glass," to be presented at this conference by J. F. Meehan.)

Of necessity, this Code. like any building code, will undoubtedly be improved as it bears the test of time. The Committee feels that further study is particularly necessary with regard to periods of buildings, torsion effects on buildings, set-backs, drift, effect of foundation conditions for building behavior during earthquakes, and behavior of non-symmetrical buildings and structures.

At the time this paper was prepared, the Code was in the process of being adopted by the City of Los Angeles and the County of Los Angeles. Approval has also been given by the Engineering Subcommittee of the International Conference of Building Officials. (See Appendix A and B.) This acceptance of the Code attests to the achievement by those who worked on the project of their objective to develop provisions which, as Prof. R. W. Clough states in his paper, "Dynamic Effects of Earthquakes,"[7] relate "the practical requirements of a building code to the essential features of dynamic theory."

* * *

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* * *

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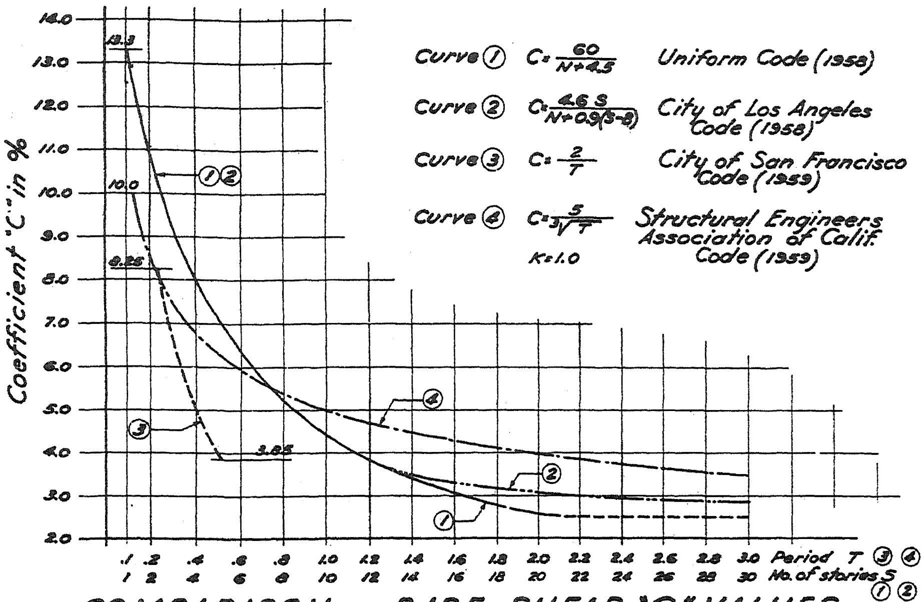
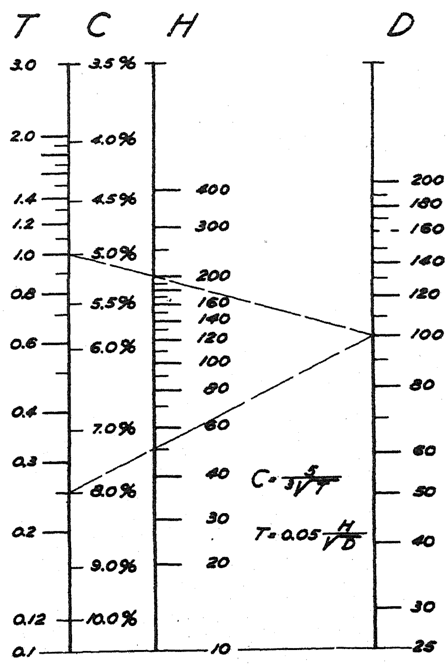


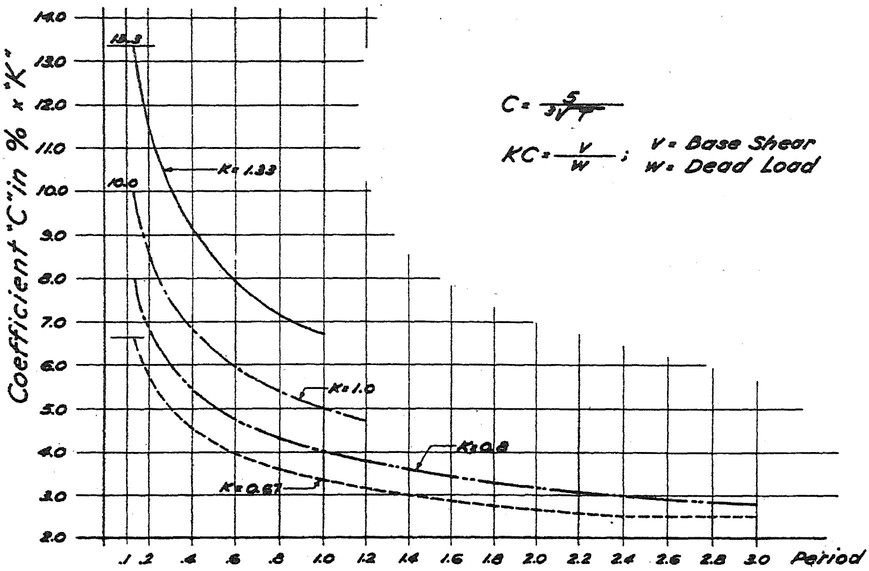
COMPARISON OF BASE SHEAR "C" VALUES

FIGURE 1



NOMOGRAPH For PERIOD "T" and COEFFICIENT "C"

FIGURE 2



KC VALUES
FIGURE 3

APPENDIX A

RECOMMENDED LATERAL FORCE REQUIREMENTS

SEISMOLOGY COMMITTEE

STRUCTURAL ENGINEERS ASSOCIATION OF CALIFORNIA

Refer to Section 2312. The following provisions are suggested for inclusion in the Uniform Building Code by cities located within an area subject to earthquake shocks:

Sec. 2312. (a) General. These lateral force requirements are intended to provide minimum standards as design criteria toward making buildings and other structures earthquake-resistive. The provisions of this Section apply to the structure as a unit and also to all parts thereof, including the structural frame or walls, floor and roof systems, and other structural features.

The provisions incorporated in this Section are general and, in specific cases, may be interpreted as to detail by rulings of the Building Official in order that the intent shall be fulfilled.

Every building or structure and every portion thereof, except Type V buildings of Group I occupancy which are less than twenty-five feet (25') in height, and minor accessory buildings, shall be designed and constructed to resist stresses produced by lateral forces as provided in this Section. Stresses shall be calculated as the effect of a force applied horizontally at each floor or roof level above the foundation. The force shall be assumed to come from any horizontal direction.

(b) Definitions. The following Definitions apply only to the provisions of this section.

SPACE FRAME is a three dimensional structural system composed of interconnected members, other than shear or bearing walls, laterally supported so as to function as a complete self-contained unit with or without the aid of horizontal diaphragms or floor bracing systems.

SPACE FRAME - VERTICAL LOAD-CARRYING: a space frame designed to carry all vertical loads.

SPACE FRAME - MOMENT RESISTING: a vertical load-carrying space frame in which the members and joints are capable of resisting design lateral forces by bending moments. This system may or may not be enclosed by or adjoined by more rigid elements which would tend to prevent the space frame from resisting lateral forces.

BOX SYSTEM is a structural system without a complete vertical load-carrying space frame. In this system the required lateral forces are resisted by shear walls as hereinafter defined.

SHEAR WALL is a wall designed to resist lateral forces parallel to the wall. Braced frames subjected primarily to axial stresses shall be considered as shear walls for the purpose of this definition.

(c) Symbols and Notations. The following symbols and notations apply only to the provisions of this Section.

C	=	Numerical coefficient for base shear as defined in Section 2312(d)1.
C_p	=	Numerical coefficient as defined in Section 2312(d) 2. and as set forth in Table No. 23-D.
D	=	The dimension of the building in feet in a direction parallel to the applied forces.
F_a	=	Allowable axial stress.
f_a	=	Computed axial stress.
F_b	=	Allowable bending stress.
f_b	=	Computed bending stress.
F_p	=	Lateral forces on the part of the structure and in the direction under consideration.
F_x	=	Lateral force applied to a level designated as x.
H	=	The height of the main portion of the building in feet above the base.
h_x	=	Height in feet above the base to the level designated as x.
J	=	Numerical coefficient for base moment as defined in Section 2312(h).
K	=	Numerical coefficient as set forth in Table 23-C.
$\sum wh$	=	Summation of the product of all $w_x \cdot h_x$ for the building.
M	=	Overturning moment at the base of the building or structure.
N	=	Total number of stories above exterior grade.
T	=	Fundamental period of vibration of the building or structure in seconds in the direction under con-

sideration.

V = Total lateral load or shear at the base.

W = Total dead load.

EXCEPTION: W shall be equal to the total dead load plus 25 per cent of the floor live load in storage and warehouse occupancies.

W_p = The weight of a part or portion of a structure.

W_x = That portion of W which is located at or is assigned to the level designated as x.

(d) Minimum Earthquake Forces for Buildings. 1. Total lateral force and distribution of lateral force. Every building shall be designed and constructed to withstand minimum total lateral seismic forces assumed to act non-concurrently in the direction of each of the main axes of the building in accordance with the following formula:

$$V = KCW$$

The value of K shall be not less than that exhibited in Table 23-C. The value of C shall be determined in accordance with the following formula:

$$C = \frac{0.05}{\sqrt[3]{T}}$$

EXCEPTION: C = 0.10 for all one and two story buildings. T is the fundamental period of vibration of the structure in seconds in the direction considered. Properly substantiated technical data for establishing the period T for the contemplated structure may be submitted.

In the absence of such data, the value of T shall be determined by the following formula:

$$T = \frac{0.05H}{\sqrt{D}}$$

EXCEPTION: T = 0.10N in all buildings in which the lateral resisting system consists of a moment-resisting space frame which resists 100% of the required lateral forces and which frame is not enclosed by or adjoined by more rigid elements which would tend to prevent the frame from resisting lateral forces.

For the purpose of computing C the value of T need not be less than 0.10 seconds.

The total lateral force "V" shall be distributed over the height of the building in accordance with the following formula:

$$F_x = \frac{V\, w_x\, h_x}{\sum wh}$$

EXCEPTION 1: One and two story buildings shall have uniform distribution.

EXCEPTION 2: Where the height to depth ratio of a lateral force resisting system is equal to or greater than five to one, 10 per cent of the total force "V" shall be considered as concentrated at the top story. The remaining 90 per cent shall be distributed as provided for in the above formula.

At each level designated as x, the force F_x shall be applied over the area of the building in accordance with the mass distribution on that level.

2. Lateral force on parts or portions of buildings or other structures. Parts or portions of buildings or structures and their anchorage shall be designed for lateral forces in accordance with the following formula:

$$F_p = C_p\, W_p$$

The values of C_p are in Table 23-D. The distribution of these forces shall be according to the gravity loads pertaining thereto.

3. Pile foundations. Individual pile footings of every building or structure shall be so interconnected by ties each of which can carry by tension and compression a horizontal force equal to 10% of the larger pile cap loading unless it can be demonstrated that equivalent restraint can be provided by other means.

(e) Distribution of Horizontal Shear. Total shear in any horizontal plane shall be distributed to the various resisting elements in proportion to their rigidities considering the rigidity of the horizontal bracing system or diaphragm as well as the rigidities of the vertical resisting elements.

(f) Drift. Lateral deflections or drift of a story relative to its adjacent stories shall be considered in accordance with accepted engineering practice.

(g) Horizontal Torsional Moments. Provisions shall be made for the increase in shear resulting from the horizontal torsion due to an eccentricity between the center of mass and the center of rigidity. Negative torsional shears shall be neglected. In addition, where the vertical resisting elements depend on diaphragm action for shear distribution at any level, the shear resisting elements shall be capable of resisting a torsional moment assumed to be equivalent to the story shear acting with an eccentricity of not less than five per cent of the maximum building dimension at that level.

(h) Overturning. Every building or structure shall be designed to resist the overturning effects caused by the wind forces and related requirements set forth in Section 2307, or the earthquake forces specified in this section, whichever governs.

EXCEPTION: The axial loads from earthquake force on vertical elements and footings in every building or structure may be modified in accordance with the following provisions:

(1) The overturning moment (M) at the base of the building or structure shall be determined in accordance with the following formula:

$$M = J \Sigma F_x h_x$$

where

$$J = \frac{0.5}{\sqrt[3]{T^2}}$$

The required value of J shall be not less than 0.33 nor more than 1.00.

(2) The overturning moment (M_x) at any level designated as x shall be determined in accordance with the following formula:

$$M_x = \frac{H - h_x}{H} M$$

At any level the overturning moments shall be distributed to the various resisting elements in the same proportion as the distribution of the shears in the resisting system. Where other vertical members are provided which are capable of partially resisting the overturning moments, a redistribution may be made to these members if framing members of sufficient strength and stiffness to transmit the required loads are provided.

Where a vertical resisting element is discontinuous, the overturning moment carried by the lowest story of that element shall be carried down as loads to the foundation.

(i) Set-Backs. Buildings having set-backs wherein the plan dimension of the tower in each direction is at least 75 per cent of the corresponding plan dimension of the lower part may be considered as a uniform building without set-backs for the purpose of determining seismic forces.

For other conditions of set-backs the tower shall be designed as a separate building using the larger of the seismic coefficients at the base of the tower determined by considering the tower as either a separate building for its own height or as part of the overall structure. The resulting total shear from the tower shall be applied at the top of the lower part of the building which shall be otherwise considered separately for its own height.

(j) Structural Frame. Buildings more than 13 stories or one hundred and sixty feet (160') in height shall have a complete moment resisting space frame capable of resisting not less than 25 per cent of the required seismic load for the structure as a whole. The frame shall be made of a ductile material or a ductile combination of materials. The necessary ductility shall be considered to be provided by a steel frame with moment resistant connections or by other systems proven by tests and studies to provide equivalent energy absorption.

(k) Design Requirements. 1. Combined axial and bending stresses in columns forming a part of a space frame. Maximum allowable extreme fiber stress in columns at intersection of columns with floor beams or girders for combined axial and bending stresses shall be the allowable bending stress for the material used. Within the center one-half of the unsupported length of the column, the combined axial and bending stresses shall be such that

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \quad \text{is equal to or less than 1.}$$

When stresses are due to a combination of vertical and lateral loads, the allowable unit stresses may be increased as specified in Section 2302.

2. Building separations. All portions of structures shall be designed and constructed to act as an integral unit in resisting horizontal forces unless separated structurally by a distance sufficient to avoid contact under deflection from seismic action or wind forces.

3. Minor alterations. Minor structural alterations may be made in existing buildings and other structures, but the resistance to lateral forces shall be not less than that before such alterations were made, unless the building as altered meets the requirements of this section of the Code.

4. Unreinforced Masonry. All elements within the structure which are of masonry or concrete and which resist seismic forces or movement shall be reinforced so as to qualify as reinforced masonry or concrete under the provisions of Chapters 24 and 26.

5. Combined Vertical and Horizontal Forces. In computing the effect of seismic force in combination with vertical loads, gravity load stresses induced in members by dead load plus design live load, except roof live load, shall be considered.

TABLE 23-C

HORIZONTAL FORCE FACTOR "K" FOR BUILDINGS OR OTHER STRUCTURES[2]

Type or Arrangement of Resisting Elements	Value of K[1]
All building framing systems except as hereinafter classified.	1.00
Buildings with a box system as defined in Section 2312(b).	1.33
Buildings with a complete horizontal bracing system capable of resisting all lateral forces, which system includes a moment resisting space frame which, when assumed to act independently, is capable of resisting a minimum of 25% of the total required lateral force.	0.80
Buildings with a moment resisting space frame which when assumed to act independently of any other more rigid elements is capable of resisting 100% of the total required lateral forces in the frame alone.	0.67
Structures other than buildings and other than those listed in Table 23-D.	1.50

(1) 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.

(2) Where wind load as set forth in Section 2307 would produce higher stresses, this load shall be used in lieu of the loads resulting from earthquake forces.

TABLE 23-D

HORIZONTAL FORCE FACTOR "C_p" FOR PARTS OR PORTIONS OF BUILDINGS OR OTHER STRUCTURES

PART OR PORTION OF BUILDINGS	Direction of Force	Value of C_p
Exterior bearing and nonbearing walls, interior bearing walls and partitions, interior nonbearing walls and partitions over ten feet (10') in height, masonry fences over six feet (6') in height.	Normal to flat surface	0.20
Cantilever parapet and other cantilever walls, except retaining walls	Normal to flat surface	1.00
Exterior and interior ornamentations and appendages	Any direction	1.00
When connected to or a part of a building: towers, tanks, towers and tanks plus contents, chimneys, smokestacks, and penthouses. Elevated tanks plus contents not supported by a building.	Any direction	0.20[1]
When resting on the ground: tank plus effective mass of its contents.	Any direction	0.10
Floors and roofs acting as diaphragms[2]	Any direction	

(1) When H/D of any building is equal to or greater than 5 to 1 increase value by 50%.

(2) Floors and roofs acting as diaphragms shall be designed for a minimum value of C_p of 10% applied to loads tributary from that story unless a greater value of C_p is required by the basic seismic formula V = KCW.

APPENDIX B

ALLOWABLE UNIT STRESSES FOR CONCRETE, STEEL AND WOOD

of the

UNIFORM BUILDING CODE of the INTERNATIONAL CONFERENCE OF BUILDING OFFICIALS

Some of the pertinent basic allowable unit stresses for concrete, steel, and wood are tabulated below; except as noted, these stresses may be increased 33-1/3 per cent for wind or seismic forces. Wind and seismic forces for design purposes are considered separately and are not additive.

The tabulated allowable unit stresses should not be used without a thorough review of the building code and particularly the specific section of the code which applies.

Unfortunately, limitation of space precludes the inclusion of allowable unit stresses for gypsum, masonry, light gage steel, and other structural materials; of values for limit or ultimate design; of allowable unit stresses for high strength materials; and of special design techniques and applications.

ALLOWABLE BASIC CONCRETE STRESSES

General:

In determining the ratio of the modulus of elasticity of steel to concrete, the modulus for steel is 30,000,000, and for standard weight concrete the modulus is assumed equal to 1000 f'_c; where f'_c is the ultimate compressive strength of the concrete as determined by testing 6 inch diameter by 12 inch long cylinders at the age of 28 days.

Reinforcing Steel:

The most commonly used steel conforms to ASTM A15

	Intermediate Grade	Structural Grade
Ultimate tensile strength psi =	70,000 to 90,000	55,000 to 75,000
Yield point, min., psi =	40,000	33,000
Elongation in 8" min., per cent =	$\frac{1,100,000}{\text{Tensile strength}}$ (But not less than 12%)	$\frac{1,200,000}{\text{Tensile strength}}$ (But not less than 16%)
Allowable tensile working stress psi	**20,000**	**18,000**

Deformations on this steel conform to ASTM A 305.

ALLOWABLE UNIT STRESSES IN CONCRETE

DESCRIPTION		ALLOWABLE UNIT STRESSES					
		For any strgth of concrete in accordance with Sec. 2606 $n = \frac{30,000}{f'_c}$	Maximum value psi	For strength of concrete shown below: f'_c = 2000 psi, n = 15	f'_c = 2500 psi, n = 12	f'_c = 3000 psi, n = 10	f'_c = 3750 psi, n = 8
Flexure: fc - extreme fiber stress in:							
Compression	fc	0.45f'c		900	1125	1350	1688
Tension in plain concrete footings	fc	0.03f'c		60	75	90	113
Shear: v (as a measure of diagonal tension)							
Beams with no web reinforcement	c	0.03f'c	90	60	75	90	90
Beams with longitudinal bars and with either stirrups or properly located truss bars, but not both	v	0.08f'c	240	160	200	240	240
Beams with longitudinal bars and a combination of stirrups and truss bars (the latter bent up suitably to carry at least 0.04f'c)	v	0.12f'c	360	240	300	360	360
Flat slabs at distance d from edge of column, capital, or drop panel	Vc	0.03f'c		60	75	90	100
Footings	Vc	0.03f'c	75	60	75	75	75
Reinforced concrete shear walls		0.05f'c		100	125	150	187
Bond: u							
Deformed bars							
Top bars	u	0.07f'c	245	140	175	210	245
In 2-way footings (except top bars)	u	0.08f'c	280	160	200	240	280
All others	u	0.10f'c	350	200	250	300	350
Plain bars (must be hooked)							
Top bars	u	0.03f'c	105	60	75	90	105
In 2-way footings (except top bars)	u	0.036f'c	126	72	90	108	126
All others	u	0.045f'c	158	90	113	135	158
Bearing: fc							
On full area	fc	0.25f'c		500	625	750	938
On one-third area or less	fc	0.375f'c		750	938	1125	1405

Stresses in the above Table are used by a large number of cities in most of the Western States.

B-2

Column Formula:

Spirally reinforced columns (closely spaced spirals enclosing a circular concrete core reinforced with longitudinal steel).

$$P = A_g\,(0.225f'_c + f_s\,P_g)$$

A_g = gross area of column

f'_c = compressive strength of the concrete (ultimate)

f_s = nominal working stress in vertical column reinforcement, to be taken at 40% of the minimum specification value of the yield point; viz., 16,000 psi for intermediate grade and 20, 000 for rail or hard grade steel.

P_g = ratio of the effective cross-sectional area of the vertical reinforcement to the gross area, A_g. P_g shall not be less than 1% nor more than 8%.

Unsupported length not greater than 10 times the least lateral dimension.

Tied Columns:

The maximum permissible axial load on columns reinforced with longitudinal bars and separate lateral ties shall be 80% of that given for spirally reinforced columns. P_g shall not be less than 1% nor more than 4%.

Long Columns:

Columns with height (h) divided by least dimension (d) ratio greater than 10 are defined as long columns. In long columns subjected to definite bending stresses, the ratio of height to width shall not exceed 20. Special designs are required for long columns.

ALLOWABLE STRUCTURAL STEEL STRESSES in Pounds per Square Inch

Tension:

Structural steel, net section	20, 000 psi
Rivets, on area based on nominal diameter	20, 000 psi
Bolts, on nominal area at root of thread	20, 000 psi
Butt welds, section through throat	20, 000 psi

Compression:

Columns, gross section.

For axially loaded columns with value of L/r not greater than 120	$17{,}000 - 0.485\,\frac{L^2}{r^2}$
For axially loaded columns (Main members) with values of L/r greater than 120	$\left[\frac{18000}{1+\frac{L^2}{18000r^2}}\right]\left[1.6 - \frac{L}{200r}\right]$
For axially loaded columns (bracing and other secondary members) with L/r greater than 120	$\frac{18000}{1+\frac{L^2}{18000r^2}}$

L is the unbraced length of the column and r is the corresponding radius of gyration, both in inches.

For main compression members, the ratio of L/r shall not exceed 180 and for bracing, struts and similar members 200.

Plate girder stiffeners, gross section	20, 000 psi
Webs of rolled sections at toe of fillet	24, 000 psi
Butt welds section through throat (crushing)	20, 000 psi

Bending:

Tension on extreme fibers of rolled sections, plate girders, and built-up members,	20, 000 psi
Compression on extreme fibers of rolled sections, plate girders, and built-up members.	
With $\frac{Ld}{bt}$ not in excess of 600,	20, 000 psi
With $\frac{Ld}{bt}$ in excess of 600,	$\frac{12{,}000{,}000}{\frac{Ld}{bt}}$

in which L is the unsupported length and d the depth, of the members; b is the width, and t the thickness, of its compression flanges, all in inches.

Shearing:

Rivets	15, 000
Pins, and turned bolts in reamed or drilled holes	15, 000
Unfinished bolts	10, 000
Webs of beams and plate girders, gross section	13, 000
Weld Metal	
On section through throat of fillet weld, or on faying surface area of plug or slot weld	13, 600
On section through throat of butt weld	13, 000

(Stress in a fillet weld shall be considered as shear on the throat, for any direction of applied stress. Neither plug nor slot welds shall be assigned any values in resistance to stresses other than shear.)

Bearing:

	Double Shear	Single Shear
Rivets	40,000	32,000
Turned bolts in reamed or drilled holes	40,000	32,000
Unfinished bolts	25,000	20,000
Pins	32,000	
Contact Area		
Milled Stiffeners and Other Milled Surfaces	30,000	
Fitted Stiffeners	27,000	

Structural Steel shall conform to American Society for Testing Materials Specification A-7 which includes the following requirements:

Tensile strength, psi	60,000 to 72,000*
Yield point, min., psi	33,000
Elongation in 8 in., min. per cent	21
Elongation in 2 in., min. per cent	22

*The upper limit of 72,000 psi may be increased by 3,000 psi for material over 1-1/2 inches in thickness.

ALLOWABLE BASIC WOOD STRESSES
West Coast Practice

Douglas Fir used in vast majority of West Coast Structures: Basic Wood Stresses.

Description of type of stress	Allowable unit stresses in pounds per square inch for grade of lumber noted (West Coast Lumber Inspection Bureau, 1956, as modified for UBC) Douglas Fir Pacific Coast Region.	
	Select Structural Grade	Construction Grade
Bending and Tension Parallel to Grain	1,710 psi	1,350 psi
Compression Parallel to Grain	1,260	1,080
Compression Perpendicular to Grain	375	350
Horizontal Shear	110	110
Modulus of Elasticity	1,600,000	1,600,000

Column Stress $P/A = \frac{.3E}{(L/D)^2}$ (Max. L/D = 50)

General:

Stresses (including fastenings) are based on "Normal" loadings - the usual live loads in buildings, etc.

Stresses may be increased	15%	for snow loads
	25%	for 7 days loads (roof L.L. in most of California)
	33-1/3%	wind or earthquake (except as noted)
	100%	impact

Nails: (In many areas no increase is permitted for wind or earthquake.)

Nail Lateral Strengths: Driven perpendicular to grain of wood	6 penny	(2" long)	70 lbs.
	8 "	2-1/2"	100 "
	10 "	3"	120 "
	12 "	3-1/4"	130 "
	16 "	3-1/2"	160 "
	20 "	4"	190 "

Bolts:

Bolt values vary with diameter, length of bearing in main member, and angle to the grain.

Bolts in Double Shear: Typical bolt values in pounds per bolt.

Length of Bolt in Main Member	Loads Parallel to Grain 1/2"Ø	3/4"Ø	1"Ø	Loads Perpendicular to Grain 1/2"Ø	3/4"Ø	1"Ø
1-5/8"	910	1390	1860	430	540	660
3-5/8"	1160	2570	3990	920	1210	1460
5-1/2"		2600	4610		1750	2210
7-1/2"		2600	4630		1690	2820

Plywood Diaphragms:

Values of shear per foot allowed with common (1-5/8") width of joists or studs, for wind or earthquake.

Nails provided at all edges - blocking required at joints.

Plywood panel joints parallel to the diaphragm width shall be staggered.

		Nails 6" o.c.	4" o.c.	3" o.c.
5/16" Plywood	6 penny nails	250#/ft.	375#/ft.	420#/ft.
3/8" Plywood	8 " "	360	530	600
1/2" Plywood	10 " "	425	640	730

Nail spacing along the boundaries of the diaphragm and along shear wall connections shall not exceed two-thirds of the above values.

Diagonal Sheathing Diaphragms:

1 x 6 bds. common construction, 2 - 8 penny nails per contact, 3 - 8 penny nails at ends of boards,

Shear 300 lbs/lin. ft. for wind or earthquake

Special construction - nails as calculated.

Diaphragms:

Diaphragms are limited as to size and shape by code requirements.