"provisions in any one code is an amalgam of theory, approximations, simplifications, judgment and experience."

Aseismic Design Provisions of Selected Building Codes*

by

Jose Ma. de Castro & Ulpiano Ignacio, Jr.**

INTRODUCTION

A building code is a set of legal requirements which are intended to provide assurance that no major failures of structures and consequent injuries or loss of life will take place under expected loads.

The foregoing objective is easily met for normal service loads which occur frequently throughout the life of the structure. A wealth of experience exists from which reliable code standards are derived. In contrast, strong earthquakes occur infrequently. As a result, aseismic design provisions in building codes do not always provide a comparable degree of assurance.

In this paper, five different codes will be reviewed with respect to aseismic design provisions. The codes are: the National Structural Code of the Philippines (NSCP, 1986) (Ref. [1]); the 1978 Applied Technology Council's Tentative Provisions for the Development of Seismic Regulations for Buildings, ATC 3-06 (Ref. [2]); the New Zealand Code of Practice for General Structural Design and Design Loadings for Buildings, NZS 4203; 1984 (Ref. [3]); the Code of Technical Requirements for Construction of Buildings in Seismic Zones, Yugoslavia, 1981 (Ref. [4]); and the Criteria for Earthquake Resistant Design for Structures, IS: 1893-1975 (Ref. [5]). Whenever possible, the review will include comparisons with pertinent NSCP, 1986 provisions.

In addition, fairly extensive discussion and commenting of NSCP, 1986 will be made.

The paper will be concluded with some recommendations which, it is hoped, will introduce some improvements to the NSCP, 1986.

UNDERLYING PHILOSOPHY OF THE CODE

NSCP, 1986

Since NSCP, 1986 earthquake provisions were derived from the American Uniform Building Code of 1982 which in turn was probably taken from the 1975 Recommended Lateral Force Requirements of the Seismology Committee of the Structural Engineers Association of California (SEAOC) (Ref. [6]), it is inferred that the latter's underlying philosophy is accepted by NSCP, 1986.

---

*Paper read at the National Society for Seismology & Earthquake Engineering Induction Ceremonies and Lectures, July 18, 1987.

**Professors of Civil Engineering, U.P. College of Engineering, Diliman, Quezon City, Philippines.

Structures designed in conformity with the SEAOC recommendations "should, in general, be able to:

1. Resist minor earthquakes without damage.
2. Resist moderate earthquakes without structural damage, but with some nonstructural damage.
3. Resist major earthquakes, of the intensity or severity of the strongest experienced in California without collapse, but with some structural damage as well as nonstructural damage.

Implicit in this philosophy is that the structure remains essentially elastic during minor and moderate earthquakes, and, although strained into the inelastic range during major earthquakes, remains stable.

NZS 4203:1984

The very first section governing design for earthquakes states: "The main elements of a building that resist seismic forces shall, as nearly practicable as possible, be located symmetrically about the centre of mass of the building." The need for symmetry is succinctly stated in the commentary to this code: "For high buildings, *symmetry is one of the most basic requirements in achieving a structure of predictable performance."*

As a general rule, NZS 4203:1984 requires that a "building as a whole and all of its elements that resist seismic forces or movements, or that in case of failure are a risk to life, shall be designed to possess ductility . . ." This in effect is similar in philosophy to that of the 1975 SEAOC.

Yugoslav Code, 1981

The Yugoslav Code is based on a design philosophy that earthquake-resistant structures should provide safety against human injuries and losses and ensure minimum damage of installations and equipment. Continuous operation of structures of vital importance with an adequate and economically acceptable construction cost should also be achievable under the code.

Earthquake-resistant buildings should be designed according to the following safety criteria:

1. For slight earthquake effects, which can be expected to occur more frequently, structural and non-structural elements should not suffer any damage.
2. For earthquakes of moderate intensity, structural elements can suffer slight damage without exhausting their capacity for post-elastic behavior. This corresponds to the design earthquake.
3. For infrequent strong earthquakes, the structure or parts of them should not fail. Technically repairable and economically justified damage should be expected in most structures . This would be the maximum expected earthquake.

The Yugoslav code requires structures to have energy absorption and dissipation capacity. This means structures should have efficient strength and ductility. Of the two, ductility is considered more important since the seismic forces usually cause the structure to work in the post-elastic range possible only in structures with sufficient ductility.

Thus, the Yugoslav Code has virtually the same basic philosophy as NSCP, 1986.

IS:1893-1975

IS:1893-1975 endeavors to ensure that as far as possible, structures are able to respond, without structural damage, to shocks of moderate intensity and without total collapse to shocks of heavy intensities.

ATC 3-06

ATC 3-06 aims "to establish design and construction criteria for buildings subject to earthquake motions in order to minimize the hazard to life and improve the capability of essential facilities to function during and after an earthquake."

"The design earthquake motions . . . are selected so that there is a low probability of their being exceeded during the normal lifetime expectancy of the building. Buildings and their components and elements which are designed to resist these motions and which are constructed in conformance with these requirements for framing and materials . . . may suffer damage but should have a low probability of collapse due to seismic-induced ground shaking."

SOME FUNDAMENTAL PROBLEMS

A structure subjected to earthquake motion will vibrate with the ground. The aim of a designer is to provide sufficient strength and deformation capability to withstand these vibrations.

While the mathematical theory of vibrations, structural analysis and structural design are well known and readily available, they are applicable to idealized models. For real structures and real earthquake motions, these analytical techniques are difficult, if not impossible, to apply.

The major obstacle is the nature of earthquakes. Perhaps the most serious problem is the present inability to predict the characteristics of earthquakes which may occur in the future. It is conventionally assumed that for structural design purposes their characteristics will be similar to those for which instrumental records are available. Even if this assumption could be satisfied, the nature of earthquake motions presents further difficulties. These motions involve simultaneous translations and rotations in three dimensional space often in chaotic fashion. Contemporary conventional structural analyses do not readily handle this general condition.

The response to earthquake motions of structures presents additional difficulties. Current design procedures implicitly assume that nonlinear, ductile behavior will take place during major earthquakes. Yet, by and large, contemporary structural analysis techniques do not readily include this explicitly. Further, simplifications of the structural frames to be used in the analysis do not allow accounting for complicated load transmission paths introduced by elements as stairs, "nonstructural" partitions, etc.

A major factor in the dynamic analysis of structures is the amount of damping present. Estimating the damping in bare frames is daunting enough. The problem becomes even more intractable when nonstructural elements have to be accounted for.

How then do codal provisions for aseismic design account for all of these factors in a manner which is simple enough to be used in routine design and yet is adequate to provide the desired structural response?

Because of the different approach taken by ATC 3-06 and its departures from usual seismic design procedures, it is not always possible to make direct comparison with other codes. Hence, a separate discussion of the ATC 3-06 provisions will be made as necessary.

All formulas and tables referred to in this paper are consolidated into appendices. This was resorted to in order not to burden the reader with a large mass of information which only needs to be referred to casually. Numbering of pertinent formulas and tables are prefaced with a capital letter which identifies the particular appendix referred to. Thus "Eqn. (B-2)" refers to Eqn. 2 of Appendix B.

ANALYSIS TECHNIQUES

Most building code methods of analysis include most, if not all, of the following considerations:

1. the seismicity of the area where the structure is located,
2. the effect of the type of soil at the site on soil-structure interaction,
3. the response spectra for structures in the area,

4. the expected behavior of the type of structure to be used on the basis of previous performance record of that particular type of structure,
5. the degree of hazard to life that the collapse of the structure being designed poses.
6. the importance of the structure particularly with regard to post-earthquake operations.
7. the portion of the mass which participates in the fundamental mode of vibration of the structure.

At present, two major analysis techniques are available: equivalent static force analysis or dynamic analysis. There are also two different methods of dynamic analysis: spectral modal analysis and computation of structural response through numerical integration. Depending upon the code, one or more of the methods are recommended for use.

NSCP, 1986

NSCP, 1986 permits both static and dynamic analyses. However, the detailed provisions dealing with the equivalent static force technique impliedly indicates preference for this method. Dynamic analysis is permitted in the paragraph entitled "Alternate Determination and Distribution of Seismic Forces."

Comment: NSCP, 1986 does not have a detailed set of requirements for implementing a dynamic analysis. It is desirable to have these explicitly laid out in order to give the user of the code guidance in carrying out such an analysis.

NZS 4203:1984

NZS 4203:1984 recognizes both the equivalent static force technique and the dynamic analysis method.

The equivalent static force is the permitted analysis method. Dynamic analysis is to be resorted to, if special conditions warrant such analysis.

The dynamic analysis method specified is the modal superposition technique. NZS 4203:1984 contains detailed provisions governing the use of the technique. The calculation of response through the use of numerical integration is specifically ruled out as the primary dynamic analysis technique "because numerical integration response analysis has been insufficiently calibrated for code purpose". However, numerical integration response analysis is allowed for use in obtaining "additional information on building behaviour, particularly in the post-elastic range, to supplement that obtained by spectral modal analysis."

Yugoslav Code, 1981

The Yugoslav Code also permits two basic approaches to the determination of seismic effects: the method of equivalent static loads and dynamic analysis. The latter method is required by the code for special structures called "out of category" structures and for prototypes of prefabricated buildings to be produced in large numbers. For structures constructed in areas of seismic intensities VIII and IX (Mercalli, Cancani, Sieberg Scale) dynamic analysis supplemented by experimental confirmation of dynamic characteristics used in the analysis is required.

IS 1893-1975

This code makes the choice of analysis method dependent upon the height of the building and the seismic zone of its location. Thus:

1. The equivalent static load method of analysis is permitted for determining seismic forces for buildings not exceeding 40 m in height.
2. For buildings between 40 m and 90 m in height, spectral modal analysis is recommended although for zones I to III (where earthquakes up to Modified Mercalli Intensities of V to VII may occur), the equivalent static load method is permitted:

3. For buildings in excess of 90 m in height in zones other than I & II, detailed dynamic analysis shall be made based on expected ground motion. For such buildings in zones I & II, the spectral modal analysis is recommended.

ATC 3-06

The equivalent static force method and the modal superposition method are recognized by ATC 3-06. However, ATC 3-06 has very detailed provisions explicitly tying the choice of analysis technique to the seismicity of the site, the occupancy type of the structure, and the performance record of the type of structure to be used.

ATC 3-06 recognizes that most major damage is caused by shaking with high energy content repeated over a substantial period of time rather than by a few isolated "spikes" of extremely high accelerations. Two index quantities, A_A and A_V, are used to characterize the expected ground motions of the site. A_A, the effective peak acceleration divided by the acceleration of gravity is the index quantity which characterizes the short period components of earthquake motion while A_V, velocity-related acceleration coefficient, characterizes the long period components. ATC 3-06 has two maps which divide the U.S.A. into zones with one for each index. After locating a site from these maps, the seismicity index and the values for A_A and A_V are obtained through the use of Table E-1.

The seismic hazard exposure group for the building which involves the occupancy type and risk or importance of the building is picked off from Table E-2.

From Table E-3, knowing the seismicity index and the seismic hazard exposure group, the seismic performance category of the building is obtained. The seismic performance category measures "the degree of protection provided for the public and building occupants against the potential hazards resulting from the effects of earthquake motions on the building". There are four seismic performance categories: A, B, C, and D.

Category A buildings can use any framing system and need not be analyzed for seismic forces on the building as a whole.

Category B buildings can also use any framing system but must be analyzed at least by the equivalent static force method.

Regular buildings of category C must be analyzed at least by the equivalent static force method. For buildings higher than 160 ft, types 2, 3 and 4 framing systems (Table E-4) may be used with some specified restrictions. Type 3 framing systems should not exceed 240 ft in height. Where irregularities exist, spectral modal analysis is preferred.

Category D buildings have the same requirements as category C except that the 160 ft limit is reduced to 100 ft and the 240 ft limit is reduced to 160 ft.

EQUIVALENT STATIC FORCE ANALYSIS

During an earthquake, inertia forces are generated throughout the structure, the sum of which results in a resisting base shear at the base of the structure.

In the equivalent static force method, the base shear is first computed and then from this, the lateral force at each floor is calculated according to the distribution stipulated by the code. The pattern of force distribution approximates the inertia force distribution pattern of the first or fundamental mode of vibrations.

It must be emphasized that the design forces calculated by this method are not the same as the actual forces developed during an earthquake.

Base Shear Calculation

The base shear force is computed as a fraction of the total weight of the building using code stipulated formulas.

Most code formulas for base shear calculations are developed by starting with a plot showing the maximum acceleration induced by earthquake ground motions acting on several single degree of freedom oscillators having the same damping factor but having different natural periods of vibration. Figure 1 (Ref. 8) shows this process graphically. From several such plots of major earthquakes a mean curve showing the ratio of the mean maximum acceleration to the acceleration of gravity is developed as shown in the middle curve of Figure 2 (Ref. 7). Adjusting for the combined effects of different modes of vibration of a multi-story structure, ductility, etc., a plot of base shear coefficient (the ratio of the maximum base shear force to the weight of the structure) vs. natural period of vibration is developed. The stippled band in the lowermost portion of Figure 2 is the range of base shear coefficients evolved for NSCP, 1986.

A multiple factor formula is commonly used, i.e., the base shear coefficient is the product of several factors.

NSCP, 1986 – The NSCP, 1986 formula considers the different items previously listed. The formula used is:

$$V = ZIKCSW$$

The product of Z, C, and S may be looked upon as the basic base shear coefficient reflecting items 1., 2., and 3. of the foregoing.

Z, the zone coefficient is intended to reflect the seismicity of the location of the structure being designed as characterized by peak accelerations which may be expected based on historical records. In previous editions of NSCP, the Philippines was divided into three zones. The Moro Gulf Earthquake of August 17, 1976 showed that the zoning needed further refinement. Pending such additional study, NSCP, 1986 now uses a single zone with $Z = 1$ throughout the Philippines.

C, the seismic coefficient, accounts for the combined effects of the different vibrational modes of the building as well as the amount of damping and ductility present. It is a function of the natural period of the structure, T, and is given by Equation (A-2) of Appendix A.

T, in NSCP, 1986, can be computed by three different equations. The first is the Rayleigh formula, Eqn. (A-3), which is the preferred procedure and is general. Unfortunately, this procedure requires the input of member properties which are unknown initially. Thus, most designers opt to use the alternative formulas, Eqn. (A-4) or Eqn. (A-5), to arrive at some preliminary sizes for use in the Rayleigh formula. Eqns. (A-4) and (A-5) should be used only under the conditions prescribed by NSCP, 1986 for the results to be reliable.

S is a coefficient which relates site conditions affecting resonance to the structural response.

Comments: This factor is dependent upon the ratio of the natural period of the structure, T, to the characteristic site period, T_S. NSCP, 1986 refers to a publication, UBC Standard No. 23-1, which was not available for reference during the preparation of this paper. However, Appendix B to the Commentary to the 1975 SEAOC Requirements (Ref. 6) requires a dynamic shear beam analysis for the characteristic site period, T_S, to be made considering any non-uniform material properties as revealed by the profile from a geotechnical investigation of the site. Several usable mathematical models are given in this Appendix but some use soil properties which are not normally determined in usual geotechnical investigations.

The product of CS should not exceed 0.14.

Certain structural systems possess less ductility than others and, therefore, to attain the same performance as ductile systems during an earthquake, would have to be designed for larger forces. To provide some assurance that all structural systems will have reasonably similar performance during an earthquake, different design force levels are assigned for different structural systems. The K coefficient was evolved for this purpose. Table (A-1) lists the different structural systems covered by NSCP, 1986. K takes into account, implicitly, the fourth and fifth items in the list of considerations of NSCP, 1986.

I was introduced to account for the sixth item in the list of items to be considered. It assigns higher forces for essential structures housing facilities intended to remain operational during and

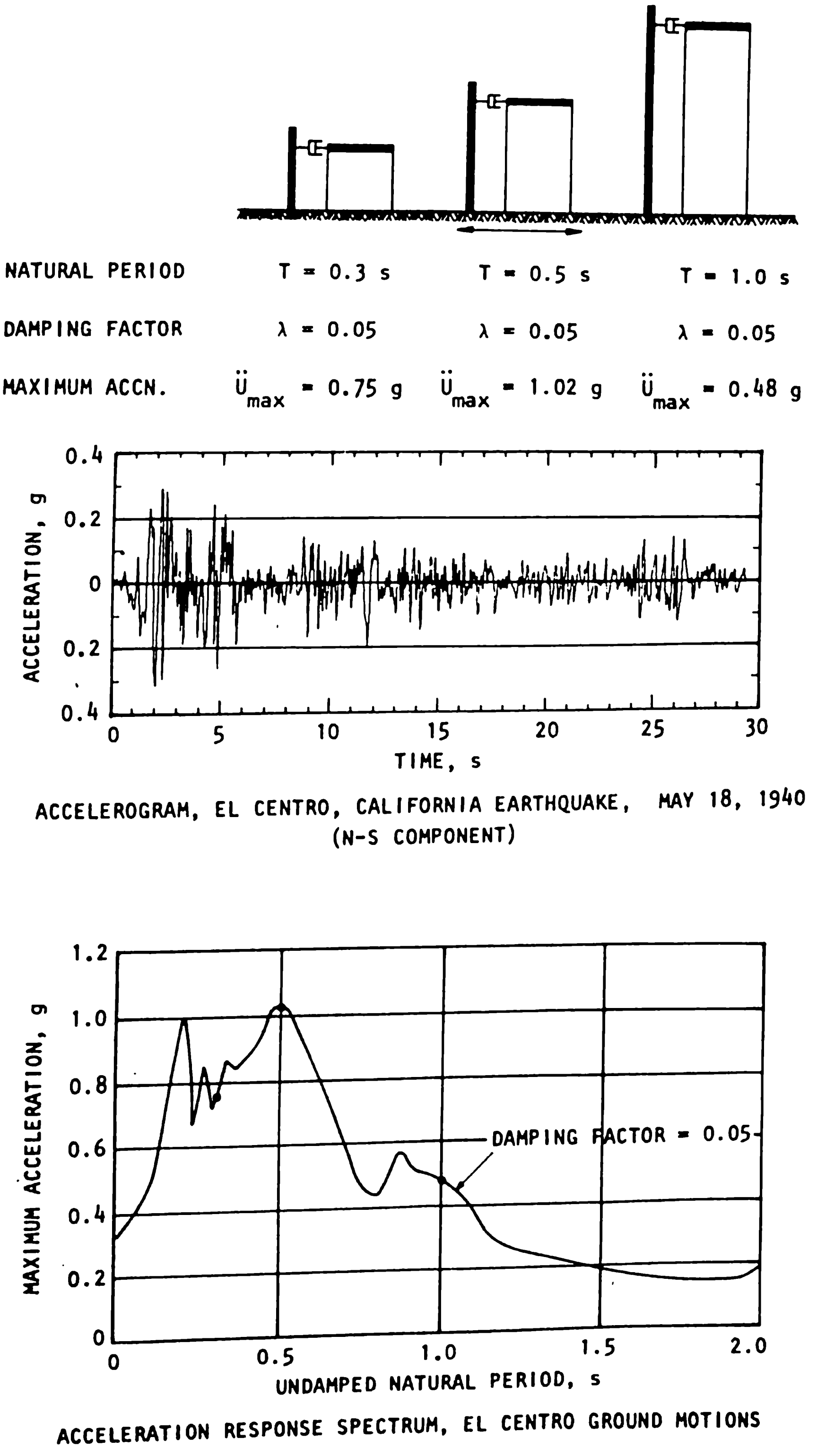

Figure 1. Evaluation of acceleration response spectrum.

after major earthquakes. The introduction of this factor was the result of the 1971 San Fernando (California, U.S.A.) earthquake when hospitals, communication facilities, vital highways, etc. were rendered non-functional due to collapse or serious damage caused by the earthquake.

NSCP, 1986 further requires the product of ZIKCS to be greater than or at least equal to 0.015.

Comments: It is interesting to note that a building with an unrealistically long period of 7.5 sec would have a C of 0.0243. For Z = 1 and assuming that I and S are at their minimum permissable values of 1 and K at its minimum value of 0.67, the product of ZIKCS will be 0.016, 6 percent more than this limit. It would seem, therefore, that for more realistic values of T, this limit will never be reached.

W is intended to define what portion of the mass of the structure participates during the vibration due to earthquake motions.

Comments: NSCP, 1986 defines W as the dead load including partition loading. This definition effectively ignores the fact that items considered as live load in vertical load analysis which are securely bolted down or attached to the structure such as water tanks, machinery, book stacks, etc. do contribute to the vibrating mass.

NZS 4203:1984 – NZS 4203-1984 also uses a multi-factor formula (from Eqns. (B-1) and (B-2):

$$V = CRSMW_t$$

The factor C is the basic base shear coefficient analogous to the product ZCS of NSCP, 1986. As such, it already includes the seismicity of the site, the response of the structure and the effect

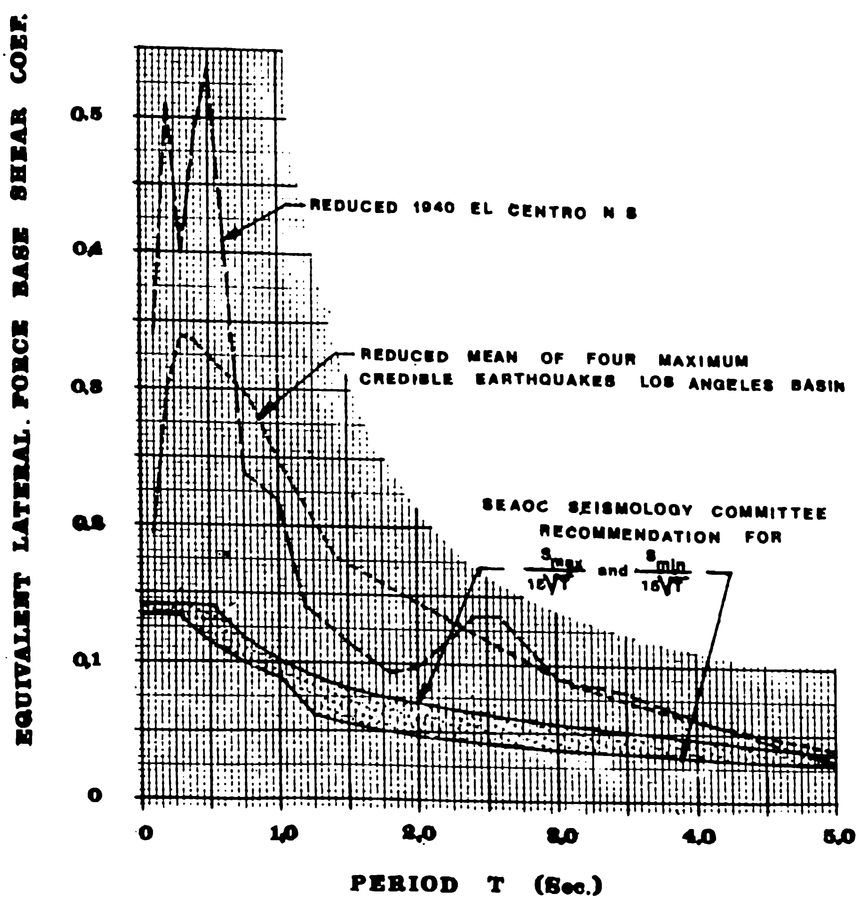

Figure 2. Base Shear Coefficient

of the type of soil. Instead of being given by a formula or a set of formulas, C is simply picked off as the appropriate plot from Figure 3 (Ref. 3). Three plots are provided, one for each of the three seismic zones into which New Zealand is divided. Two types of soil conditions are covered: flexible soils and rigid and intermediate soils. Unlike NSCP, 1986 which requires special geotechnical tests to determine the soil effects, NZS 4203:1984 uses the undrained shear strength and thickness of the soil supporting the building (Appendix B) to define soil types.

The value of T for use in Figure 3 is computed using a variation in form of the Rayleigh Formula (Eqn. (B-3)). For "reasonably regular" buildings, T may be obtained from an approximate relation (Eqn. (B-4)).

Minimum values of C as indicated in Figure 3 occur at T_S of 1.2 seconds.

Analogous to the I factor in NSCP, 1986, NZS 4203:1984 incorporates what is termed the risk factor, R. The number of categories, all carefully described, are somewhat more in NZS 4203: 1984 as shown in Table B-1 than in NSCP, 1986.

The structural factor, S, is similar to the factor K of NSCP, 1986 but is much more comprehensive in scope as shown in Table B-2.

Closely associated with the structural factor is the material factor, M, Table B-3. This factor reflects the energy dissipating capability of the material concerned. A special table for SM is provided for timber (Table B-3a).

The product of CRSM need not be more than 4.8CR for steel and prestressed concrete nor 4CR for structures of other materials but should be at least equal to 0.04.

Yugoslav Code, 1981 for base shear computations, uses a multi-product formula resulting from Eqns. (C-1) and (C-2):

$$S = K_c K_s K_d K_p G$$

K_c serves the same purpose as the importance factor, I, of NSCP, 1986. Table C-1 gives the values of K_c for different structural types.

K_s, the seismic intensity coefficient, modifies the base shear according to the seismicity of the area. It has the same function as the Z coefficient of NSCP, 1986. Values are found in Table C-2.

K_d, termed the dynamic coefficient, corresponds to the combined effect of the structural response coefficient, C, and the soil coefficient S of NSCP, 1986. Table C-3 gives the values of this coefficient. The natural period of the building, T, should be computed from a dynamic analysis or from approximate formulas derived from dynamic analysis.

K_p, serves the same purpose as the structural performance factor, K, of NSCP, 1986. Values are obtained from Table C-5.

The product of $K_c K_s K_d K_p$ should not be less than 0.02.

G is the sum of dead load, probable live load, snow load and weights of permanently attached pieces of equipment.

A unique feature of the Yugoslav Code is the specification of a vertical seismic force calculated in accordance with Eqn. (C-6).

IS:1893-1975 uses for the computation of the base shear the formula:

$$V_B = C\alpha_h W$$

C, the spectral response factor is obtained from Figure 4.

T, the natural period of the building used in determining C, is obtained either by approximate formulas identical to those used by NSCP, 1986 (Eqns. (D-2) & (D-3)), by rational mathematical analysis, or by experimental determinations from similar buildings.

α_h, the seismic coefficient, is determined by either the seismic coefficient method (Eqn. (D-4)) or by the response spectrum method (Eqn. (D-5)). In both methods, a soil factor, an importance factor and the seismic characteristics of the site are involved.

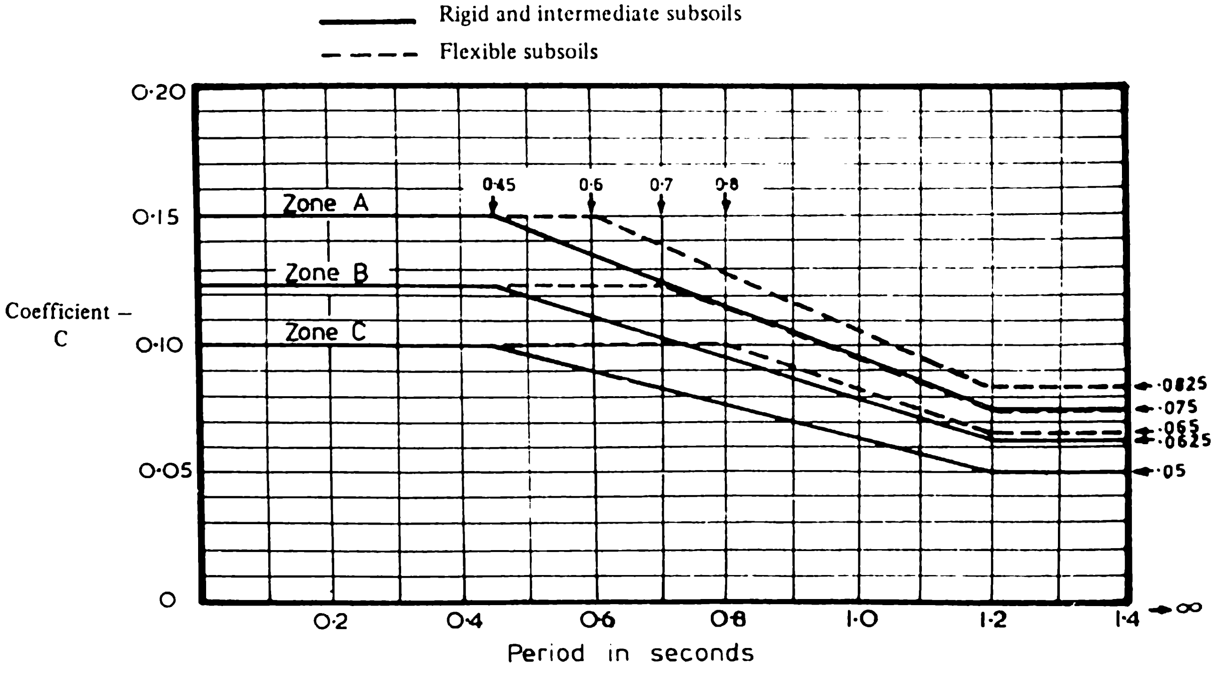

Figure 3. Basic Seismic Coefficient C

ATC 3-06 uses the following formula for base shear computations when the equivalent static load method is permitted:

$$V = C_sW$$

with

$$C_s = [1.2A_vS] \; / \; [RT^{2/3}]$$

The base shear formula is thus another multi-factor formula involving A_v the velocity-related acceleration coefficient, S the soil coefficient, R the response modification factor which depends upon the structural type and the natural period of the building T.

T is to be obtained using established methods of mechanics. One method specifically mentioned in the commentary is Rayleigh's method. In no case should T exceed 1.2 T_A. Alternatively, approximate formulas (Eqns. (E-5) and (E-6)) may be used.

Base shear obtained by the equivalent lateral force method may be modified according to the chapter on soil-structure interaction.

DYNAMIC ANALYSIS

Dynamic analysis provides a more rigorous means of determining the distribution of lateral forces. It gives a good picture of which features of a structure are most vulnerable. However, the degree of accuracy is highly dependent on the mathematical model used, i.e., how well the model corresponds to the real structure, whether the response spectrum or time history used reflects the most probable future earthquake motion that may be expected for the site of the structure, etc. Thus dynamic analysis cannot possibly provide a complete solution to the problem of determining earthquake forces accurately.

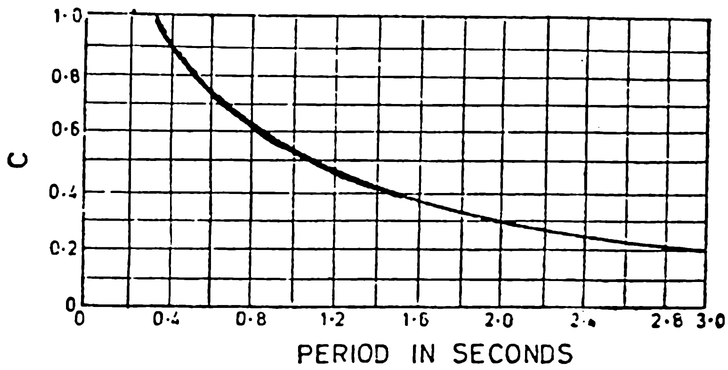

Figure 4. C Versus Period

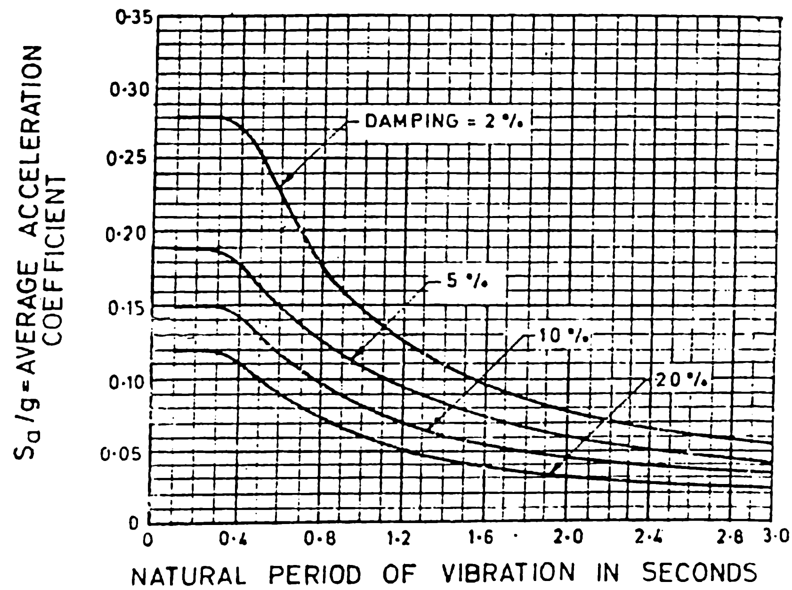

Figure 5. Average Acceleration Spectra

NSCP, 1986

NSCP, 1986 requires the use of dynamic analysis for irregular structures.

Comment: NSCP, 1986 provides no guidance as to how dynamic analysis is to be carried out.

NZS 4203:1984

The spectral modal analysis is used with the structural design spectrum obtained for each mode using the expression KC where C is obtained from Figure 3 and K is a scaling factor so chosen that the computed base shear V is not less than $0.9C_dW_t$. The results of the modal analysis are derived from an elastic model and from a response spectrum set up for the entire mass of the structure rather than for the individual masses. Hence, the base shear will be overstated particularly in a structure where inelastic deformations are permitted during an earthquake. This is the reason for using a scaling factor.

Yugoslav Code, 1981

Although dynamic analysis is recommended for some building types and is required for special structures, no provisions which could guide the designer are given as to how the analysis is to be conducted.

Where a dynamic analysis is not conducted, an approximate method is suggested. A maximum relative displacement between floors for elastic behavior not to exceed $h_1/350$ where h_1 is the height between floors in cm is used to calculate elastic internal forces of the building. A maximum relative displacement of $h_1/150$ is then used to calculate internal forces of the building assuming inelastic behavior.

IS:1893-1975

Spectral modal analysis is the dynamic analysis procedure specified in this code. A detailed procedure for computing the floor loads and the shears are given (Eqns. (D-7) and (D-8)).

ATC 3-06

Spectral modal analysis is the dynamic analysis procedure specified in the code. A detailed procedure for computing the base shear and floor loads are given (Eqns. (E-9) and (E-12)). ATC 3-06 specifies further that the analysis "shall include, for each of two mutually perpendicular axes, at least the lowest three modes of vibration or all modes of vibration with periods greater than 0.4 second, whichever is greater, except that for structures less than three stories in height, the number of modes shall equal the number of stories". Results from the modal analysis may be modified in accordance with a chapter on soil-structure interaction.

VERTICAL DISTRIBUTION OF BASE SHEAR

When a mass is subjected to acceleration, the inertia force developed is equal to the mass times acceleration. For elastic systems, the acceleration is proportional to the displacement. Therefore, the inertia forces on an elastic structure are proportional to the product of the masses times their respective displacements. If, then, the displacement patterns are known, the pattern of distribution of forces is also known.

For a uniform building frame consisting of beams and columns vibrating in the first or fundamental mode, the deflection curve is very close to a straight line, i.e., zero at the bottom varying linearly to a maximum at the top. Hence, the forces must similarly be distributed with a zero force at the bottom increasing linearly to a maximum at the top — the so-called inverted triangular distribution.

There are, however, buildings whose lateral force resisting systems' deflection curves deviate significantly from a triangular pattern. For these, the base shear distribution also deviates from the triangular distribution.

NSCP, 1986

Eqn. (A-10) of NSCP, 1986 distributes the base shear in an inverted triangular distribution. The quantity F_t provides a correction for structures whose deflection curve deviates from the straight line pattern discussed in the foregoing. F_t is a function of the fundamental period. This is reflected in Eqn. (A-9) which also provides a cut-off value of T at, and below which, F_t is not needed.

For irregular structures, NSCP, 1986 requires the use of dynamic analysis in determining the distribution of the lateral forces.

Comments: NSCP, 1986 does not provide any definition or description of what features distinguish a "regular" structure from an irregular one. Neither does it describe the manner in which the "dynamic characteristics of the structure" are to be determined.

NZS 4203:1984

The base shear as determined by equivalent static force method is required to be distributed among the different floors in accordance with the straight-line distribution as given by Eqn. (B-5). Where the height to depth ratio of the lateral force resisting system is ≥ 3, the 0.1V is to be concentrated at the top with the remainder of the base shear distributed according to Eqn. (B-5).

When the base shear has been determined by the spectral modal method, the shear at any level is taken as the square root of the sum of the squares of the modal shears at that height. This shear should not be less than 0.8 of the values computed by the equivalent static force method.

Yugoslav Code, 1981

For buildings up to five stories high, the Yugoslav Code uses the same linear distribution of forces that NSCP, 1986 uses with some slight modifications in the formulas. (Eqn. (C-3)).

For buildings higher than five stories, 15 percent of the base shear is applied at the top of the building with the remainder of the base shear distributed among the floors in the usual linear fashion, i.e., according to Eqn. (C-3).

IS:1893-1975

Unlike the other codes, a second degree parabolic distribution (Eqn. (D-6) is used. No force at the top floor, F_t, is required.

Comment: This distribution is probably an attempt to approximate the displacement curve, hence the distribution of forces, in flexural cantilever. The other codes used F_t to produce this effect.

ATC 3-06

ATC 3-06 distributes the base shear vertically according to the relation:

$$F_M = (w_x h_x{}^k)/(\Sigma\, w_x h_x{}^k)$$

with k = 1 for buildings with T $\leq$0.5 sec
k = 2 for buildings with T $\geq$2.5 sec
for buildings with 0.5 $<$ T $<$ 2.5 sec,
linear interpolation between k = 1
and k = 2 should be used.

HORIZONTAL SHEAR DISTRIBUTION

The design of any structure must include means for distributing shears not only among the different resisting elements at a given story but also for distributing shears at setbacks and other discontinuities of vertical elements as well as provide a means for interaction between shear walls and moment frames.

NSCP, 1986

The total story shear is to be distributed to the various elements of the structural resisting system in that story in proportion to their individual stiffnesses and including the degree of rigidity of horizontal bracing and diaphragms. Any other elements which are not intended to be part of the lateral force resisting elements but which have sufficient rigidity to affect the behavior of the system must be "considered and designed for".

NZS 4203:1984

There is no provision describing the manner in which the story shear is to be distributed among the different elements of the lateral force resisting system at that story.

Comment: Since the routine use of computers for structural analysis is prevalent in New Zealand, the distribution among the different resisting elements becomes automatic during the analysis. This probably explains this "omission".

Yugoslav Code, 1981

The procedure of distributing the story shear among the different resisting elements for that story is not specified in this code.

IS:1893-1975

No provisions governing the distribution of story shear among resisting elements are given.

ATC 3-06

The story shear, defined as

$$V_x = \Sigma F_i$$

is distributed among the various vertical resisting elements in proportion to their respective stiffnesses.

TORSION

When an eccentricity exists between the center of mass of a structure and its center of rigidity at any level, then a horizontal torsional moment will develop during an earthquake. Where the center of mass and the center of rigidity coincide or are separated only by a small eccentricity, then a minimum torsional moment to account for possible larger "accidental" eccentricities is often required by codes to be designed for.

NSCP, 1986

NSCP, 1986 requires that any horizontal torsional moment which exists at any level should be designed for. The minimum design horizontal torsional moment is to be calculated by using an eccentricity of 5 percent of the maximum building dimension at that level.

Comment: NSCP, 1986 does specify how this eccentric shear is to be applied.

NZS 4203:1984

A torsion equal to the force at any level applied in turn at each of two points 0.1 b from the center of mass at that level and on either side of it measured perpendicular to the direction of loading is required. The dimension, b, is the building dimension perpendicular to the direction of loading.

The use of spectral modal analysis for evaluation of torsion is suggested as an alternative to the foregoing for structures which are reasonably regular.

However, for structures which are irregular and more than four stories high, a three-dimensional modal analysis is required.

If a modal analysis is made to determine torsional moments, the first three modes are required as a minimum for symmetrical or moderately unbalanced structures. For three-dimensional modal analysis, four modes are required to be included – two translational, and two torsional. The minimum accidental eccentricity of 0.1b on either side of the center of mass is considered as the lower limit of eccentricity. In no case is the computed torsional moment to be less than that as determined by the static method of analysis.

No mention is made as to the manner in which the distribution of these effects to the resisting elements is to be carried out.

Comment: See comment in DISTRIBUTION OF HORIZONTAL SHEAR of the foregoing.

Yugoslav Code, 1981

The torsional moment at each story is given according to Eqn. (C-4). No procedure for distributing the resulting torsional shear among the different elements of the lateral force resisting system is specified.

IS:1893-1975

No provisions for torsion for equivalent static analysis is given.
For dynamic analysis, the torsion would be automatically taken into account.

ATC 3-06

The torsional moment associated with the story shear at any level should be distributed according to the stiffnesses of the resisting elements of the structure. In no case should the eccentricity used in computing the torsion be less than 5 percent of the dimension of the building perpendicular to the applied shear measured each way from the center of mass.

LATERAL FORCES ON COMPONENTS AND ELEMENTS OF STRUCTURES

Falling debris have a great potential of injuring or killing people who are trying to get out of the building or who are in the immediate vicinity of the building during an earthquake. Codal provisions attempt to eliminate or at least minimize the possibility of such debris being created by an earthquake.

NSCP, 1986

NSCP, 1986 provides that all parts, elements, or components of structures, whether structural or non-structural, along with their anchorages, shall be designed for lateral forces in accordance with the formula:

$$F_p = ZIC_pW_p$$

The values of C_p are given in Table (A-3). In addition, numerous requirements with respect to anchorages are given throughout NSCP, 1986.

NZS 4203:1984

NZS 4203:1984 provides very detailed requirements for the design of elements and non-structural components of the structure. The basic formula, Eqn. (B-6) is the same as that used by NSCP, 1986.

$$F_p = C_pW_p$$

However, Tables B-4 and B-5 provide a more comprehensive set of constants for use in evaluating C_p than those available in NSCP, 1986.

Yugoslav Code, 1981

Structural and non-structural parts or components of structures and anchorages of equipment are designed for forces calculated in accordance with Eqn. (C-5).

IS:1893-1975

This particular Indian Standard does not have any provisions on this. However, detailed design recommendations are found in another Indian Standard-Code of Practice for Earthquake Resistant Design and Construction of Buildings, First Revision, IS:4326-1976 (Ref. 9).

ATC 3-06

ATC 3-06 devotes one entire chapter of the code to the design levels of forces acting on architectural, mechanical and electrical components and systems and their attachments. The provisions are too many to include in this paper. Suffice it to say that the items involved are very comprehensive.

DRIFT & BUILDING SEPARATIONS

Excessive sway of structures needs to be prevented. In addition, particularly when brittle components are framed by the structural frame, the interstory relative displacements or drifts must be controlled to prevent crushing of these components. Finally, buildings must have adequate separation so that damage due to the buildings hammering against each other will not take place. This last mode of damage was estimated to be responsible for 40 percent to 60 percent of the major damage in the Mexico Earthquake of 1985 (Ref. 8).

NSCP, 1986

NSCP, 1986 requires that lateral deflections or drift of any story relative to its adjacent story should not exceed 0.005h. The displacements calculated using the codal floor loads are multiplied by 1/K, which ratio should not be less than 1.0, before checking against maximum drifts permitted by the code.

Comment: The 1/K multiplied is needed in order to anticipate the larger displacements which will take place during the excursion into the inelastic range by a ductile structure under a major earthquake's ground motion.

NZS 4203:1984

Computed deformations resulting from application of forces are to be multiplied by the factor K/(SM). When the equivalent static force method is used, K is taken as 2; otherwise, if modal analysis is used, K is taken as 2.2.

Each building is required to be separated from its neighbor through *a clear space from the property line* either 1.5 times the deflections computed according to the method of the previous paragraph or 0.002 times the building's height whichever is greater nor less than 12 mm.

Parts of buildings or buildings on the same site separated from each other are required to have a minimum clear space from each other at least 1.5 times the sum of their computed deflections according to the method described in the first paragraph of this section or 0.004 times their height whichever is larger but not less than 25 mm.

Interstory deflections computed in accordance with the method described in the first paragraph of this section are required not to exceed 0.006 times the story height where non-structural elements are not separated as described in the next paragraph nor 0.010 times the story height multiplied by a zone factor of 1 for seismic zone A; 5/6 for seismic zone B; and 2/3 for seismic zone C.

Non-structural elements, particularly brittle elements, should be so separated from the structure that when the structure deforms to twice the deformations computed according to the method of the first paragraph of this section, no impact will take place.

Yugoslav Code, 1981

The code specifies a maximum horizontal deflection:

$f_{max} = H/600$ where H = height of structure.

If soil conditions warrant, a special investigation should be made to determine magnification of the foregoing.

IS:1893-1975

The maximum horizontal relative displacement between any two floors is required to be not more than 0.004 times the interfloor height. Additional limits are given in Ref. 9.

ATC 3-06

The design story drift, Δ, is computed as the difference between the deflections, δ_x of the story at the top and the story at the bottom of the given story. δ_x, in turn, is computed by the formula:

$$\delta_x = C_d \delta_{xE}$$

where C_d is the deflection amplification factor from Table E-xx
δ_{xE} is the deflection from an elastic analysis using the seismic resisting system and design seismic forces.

Additional increases are also specified in the separate chapter on soil-structure interaction previously mentioned.

DESIGN PRINCIPLES

Practically all codes either explicitly or implicitly assume that for major structures, the lateral force resisting system has significant ductility. The means by which these are attained vary from code to code.

However, it is recognized that some structural systems which will have limited or little ductility will also be used. It is the function of the building codes to provide guidance also for the design of such structures.

Structural Systems

Structural systems in use can vary from vertical load carrying systems which utilize bearing walls exclusively without any complete space frames (consisting of beams and columns), to, systems consisting of space frames supporting vertical loads often combined with walls. Thus, lateral earthquake forces are resisted in a variety of ways depending upon the structural system used.

NSCP, 1986 classifies structural systems in Table A-1 into:

1. Structures consisting primarily of vertical load carrying ductile space frames capable of resisting the entire lateral earthquake force.
2. Dual bracing systems comprised of substantially complete vertical load carrying, ductile, moment resisting space frames and shear walls or braced frames. The dual system is required to resist the entire lateral force with distribution among the different resisting elements in accordance with their individual stiffnesses. In addition, the shear walls or braced frames acting independently are required to be capable of resisting the entire lateral force. The ductile space frame, in turn, is required to be able to resist at least 25 percent of the entire lateral load as a reserve in the event of damage to a portion of the shear wall system.

   Comments: The portion of the code requiring the shear walls or braced frames to be capable of independently resisting 100 percent of the lateral earthquake forces does not make clear what deformation pattern is to be used in apportioning the lateral forces among the different levels of the shear walls or braced frames. This is of interest to the structural engineer since a shear wall acting by itself will deflect like a flexural cantilever beam. On the other hand, the frame-shear wall combination at the onset of the earthquake will initially deflect in an elastic manner which configuration will be intermediate between that of a shear beam and a flexural beam.
3. Systems carrying a substantial part of the vertical load by bearing walls and resisting the lateral force through shear walls or braced frames which may or may not be parts of the vertical load carrying system. NSCP, 1986 terms this system a "Box System".

4. Substantially complete vertical load carrying systems with braced frames or non-bearing shear walls but which do not qualify under items 1. and 2. of the foregoing.

Comments: NSCP, 1986 provides considerable latitude to the structural engineer to designate the parts which may be considered as the lateral force resisting ductile moment resisting space frame. By operation of the sentence "The value of K shall be not less than set forth in Table 2. 1-A." (Table A-1 of this paper), the engineer, by implication, can combine different systems in the different directions of the lateral force or at different levels.

However, it is this latitude that can cause problems.

Assuming certain elements as "non-structural" could lead to neglecting the influences of elements which are rigid enough to alter significantly the vibration characteristics of the structure. Also, since ductility is relied upon to reduce the lateral earthquake force magnitudes below those indicated by purely elastic behavior, the corresponding displacements must be capable of being accommodated by those vertical load carrying elements which are not considered to be part of the lateral force resisting system in order to prevent any reduction in their vertical load carrying capacity. A final, potential source of problem is a provision, easily overlooked because it is tucked away as a part of the ductility requirements, which limits buildings which do not qualify for classification under 1. or 2. of the foregoing to a maximum height of 50 meters.

NZS 4203:1984 NZS 4203-1984 classifies structural systems into seven categories (Table B-2). This code also further ties the behavior of a particular system to the material of which it is made, hence, the detailed provisions governing the product SM of the structural type and structural material factors.

Yugoslav Code, 1981 The code does not have any classification of structural systems.

IS:1893-1975 does not have any classification system comparable to that of NSCP, 1986. A limited classification is made in Ref. 9.

ATC 3-06 provides a classification system of structural framing systems very similar to that of NSCP, 1986. Table E-4 details the classification.

Ductility Requirements and Designing in Different Materials

Since ductility is the primary basis of design for the vast majority of building codes, the requirements to achieve these need some discussion.

NSCP, 1986 requirements not yet discussed are the means of attaining ductility. NSCP, 1986 points to the different chapters governing the design for different materials to cover the ductility requirements as set forth in the chapters on steel, concrete, etc.

Comments: It is interesting to note that, notwithstanding the constant use of the word "ductility", no definition or description of what constitutes ductile behavior is given anywhere in the code.

With respect to the concrete design portion, NSCP, 1986 commendably makes use of the provisions for design and detailing of concrete structures of ACI-318-83 which are based on the most up-to-date research results.

It is curious, though, that in NSCP, 1986, the means of designing and detailing of concrete structures for resistance to earthquake lateral forces is relegated to an appendix unlike in the immediately previous edition where these requirements were integrated into the main body of the chapter on concrete. Also, perhaps due to the printer's collation error, this appendix is attached to the chapter on excavations and foundations rather than the chapter on concrete. This comment is made since the different appendices to the chapter on steel are correctly found immediately after the chapter on steel.

A further curious item in Section 5.1-General Requirements of Chapter 5-Concrete is the requirement that provisions of Appendix A should be adhered to in regions of "moderate or high seismic risk" but that in regions of "low seismic risk", these provisions would not apply. Yet, as already discussed relative to the Z factor, the Philippines has been classified as a single seismic zone – one of moderate to high seismic risk.

NSCP, 1986, under a subsection on "Deformation Compatibility" requires that framing elements which are not part of the lateral force-resisting system must be capable of resisting the vertical load and induced moments corresponding to 3/K times the distortion produced by code-required lateral forces.

NZS 4203:1984 Except for small buildings, NZS 4203:1984 requires that seismic forces and movements will be resisted by systems which possess ductility. Where ductile flexural yielding is the primary means of dissipating seismic energy transmitted to a building, adequate ductility must be provided. The Commentary to this code defines a building as having adequate ductility if it is "capable of *deflecting laterally through at least eight load reversals* so that *the total horizontal deflection at the top of the main portion of the building* under earthquake load combinations and calculated under the assumption of appropriate plastic hinges *is at least four times that at first yield, without the horizontal carrying capacity* of the building *being reduced by more than 20 per cent.*

Designing for adequate ductility is described fully in other New Zealand Standards dealing with structural design in different materials. However, the requirements that must be met for frames to be considered as ductile frames are meticulously detailed in NZS 4203:1984. Space limitations prevent their being covered fully in this paper. Nevertheless, among the more important requirements are:

1. Plastic hinges should preferably form in beams.
2. If plastic hinges do form in a column then there must be at least three other columns at the same level of the main lateral load resisting system which remain elastic.
3. Shear walls are designed for ductility.
4. Capacity design is used. This method of design is based on making sure that the energy-dissipating mechanisms are properly detailed and all other structural elements provided with the necessary reserve strength that the mechanisms are maintained during earthquake deformations.

Yugoslav Code, 1981 The code has, scattered among different sections, some general provisions on attaining ductility. The amount of detail does not approach that available in the design for different materials found in NSCP, 1986.

IS:1893-1975 does not provide any specific requirements in this regard. However, in Ref. 9 detailed provisions are given.

ATC 3-06 provides a comprehensive set of specifications to achieve ductility for structural design in different structural materials.

ADDITIONAL CONSIDERATIONS

Unique among the codes, ATC 3-06 contains detailed provisions on the P-Δ effect, on abatement of seismic hazards in existing buildings, guidelines for repair and strengthening of existing buildings and guidelines on emergency post-earthquake inspection and evaluation of earthquake damage in buildings.

CONCLUSIONS AND RECOMMENDATIONS

From the foregoing review of five different codes, it becomes obvious that the provisions in any one code is an amalgam of theory, approximations, simplifications, judgment and experience.

While many provisions are similar in form, values of many factors are different. Also, some codal requirements take for granted that engineers using a given code are familiar with terminology and techniques presented.

However, in the Philippines, technical information is hard to come by and if ever available, they are in books or journals which are prohibitively expensive. Thus it is an imperative need that NSCP, 1986 provide a detailed commentary discussing, in particular, the background of the dif ferent provisions and the constraints in their use.

As far as specific items of major importance are concerned, the equivalent static lateral force method should be explicitly limited to regular structures with the term "regular" defined clearly.

The use of spectral modal superposition should be encouraged.

Many of the items which are described in comments as either missing from, or glossed over in NSCP, 1986 should be provided in detailed form.

Finally, because of lack of information interchange of local seismic and other pertinent data, code writing authorities are constrained to adopt foreign codes almost *in toto*. It is necessary then that interested government agencies as well as the allied professions pool their data and human resources together and cooperate with one another in adapting foreign codes to local conditions so that the resulting code will provide the desired degree of protection to the public.

REFERENCES

1. *National Structural Code of the Philippines,* Vol. 1, Buildings, Towers, and other Vertical Structures, 3rd Edition, June 1986, Association of Structural Engineers of the Philippines.
2. *Tentative Provisions for the Development of Seismic Regulations for Buildings,* Applied Technology Council, ATC Publication ATC 3-06, California.
3. *Code of Practice for General Structural Design and Design Loadings for Buildings,* 2nd Edition Incorporating Amendments to 1976 Edition, Standards Association of New Zealand, NZS 4203:1984.
4. *Code of Technical Requirements for Construction of Buildings in Seismic Regions,* No. 50-3547/1, 1981, Belgrade, Yugoslavia.
5. *Criteria for Earthquake Resistant Design of Structures,* 3rd Revision, Indian Standards Institution, IS: 1893-1975.
6. Seismology Committee, *Recommended Lateral Force Requirements and Commentary,* Structural Engineers Association of California, 1975, San Francisco.
7. SEED, H. BOLTON & IDRISS, I. M., *Ground Motions & Soil Liquifaction During Earthquakes,* Earthquake Engineering Research Institute, Berkeley, California, 1982.
8. DE PINERES, OSCAR, "A Safer Earthquake Design Approach", *Civil Engineering,* May 1987, V. 57, No. 5, pp. 52-54.
9. *Code of Practice for Earthquake Resistant Design and Construction of Buildings,* First Revision, IS:4326-1976.

APPENDIX A: NSCP, Vol. 1 HIGHLIGHTS

I. Equivalent Lateral Force Analysis

A. Base Shear

$V = ZIKCSW$ (A-1)

where

$C = \frac{1}{15\sqrt{(T)}}$ (A-2)

< 0.12

where,

$T = 2\pi\sqrt{\{(\Sigma w_i \delta_i^2)/(g[\Sigma f_i \delta_i + (f_n + f_t)\delta_n]\}}$ sec (A-3)

or,

$T = (0.09\, h_n)/(\sqrt{D})$ sec (A-4)

or,

$T = 0.1N$ sec (A-5)

$S = 1.0 + (T/T_s) - 0.5\,(T/T_s)^2$ for $T/T_s \leq 1$ (A-6)

or,

$S = 1.2 + 0.6\,(T/T_s) - 0.3\,(T/T_s)^2$ for $T/T_s > 1$ (A-7)

where,

$0.5 \leq T_s \leq 2.5$ sec and $T \geq 0.3$ sec

but,

$S = 1.5$ when T_s has not been established properly

When T has been established accurately and

$T > 2.5$ sec, $T_s = 2.5$ sec

The product $CS \leq 0.14$

$Z = 1$

K values are given in Table A-1

I values are given in Table A-2

II. Distribution of Base Shear among the different Floors

$V = F_t + \Sigma F_i$ (A-8)

where,

$F_t = 0.07V$ (A-9)

$\leq .25V$

$= 0$ when $T \leq 0.7$ sec

$F_x = [(V - F_t)\, w_x h_x]/[\Sigma w_i h_i]$ (A-10)

III. Lateral Forces on Structural and Non-Structural Elements and Components (A-11)

$F_p = ZIC_pW_p$

TABLE A-1

HORIZONTAL FORCE FACTOR "K" FOR BUILDINGS OR OTHER STRUCTURES[1]

TYPE OR ARRANGEMENT OF RESISTING ELEMENTS	VALUE[2] OF K
All building framing systems except as hereinafter classified.	1.00
Buildings with a box system as defined in Sec. 2.1 (b) **Exception:** Buildings not more than three stories in ht. w/ stud wall framing and using plywood horizontal diaphragms and plywood vertical shear panels for the lateral force system may use K = 1.0	1.33
Buildings with a dual bracing system consisting of a ductile moment resisting space frame and shear walls and braced frames using the following design criteria: 1. The frames and shear walls or braced frames shall resist the total lateral force in accordance with their relative rigidities considering the interaction of the shear walls and frames. 2. The shear walls or braced frames acting independently of the ductile moment-resisting portions of the space frame shall resist total required lateral forces. 3. The ductile moment-resisting space frame shall have the capacity to resist not less than 25 per cent of the required lateral force.	0.80
Buildings with a ductile moment-resisting space frame designed in accordance with the following criteria: The ductile moment-resisting space frames shall have the capacity to resist the total required lateral force.	0.67
Elevated tanks plus full contents, on four or more cross-braced legs and not supported by a building.[3, 4]	2.5
Structures other than buildings and other than those set forth in table 2.1-B.	2.00

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

[2]The coefficient may be modified by the Building Official upon advice of seismological and structural engineers specializing in aseismic design.

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

[4]The tower shall be designed for an accidental torsion of five percent as specified in Sec. 2.1 (e) 5. Elevated tanks which are supported by buildings or do not conform to type or arrangement of supporting elements as described above shall be designed in accordance with Sec. 2.1 (g) using Cp = 0.3.

TABLE A-2

VALUES FOR OCCUPANCY IMPORTANCE FACTOR I

TYPE OF OCCUPANCY	I
Essential Facilities	1.5
Any building where the primary occupancy is for assembly use for more than 300 persons (in one room)	1.25
All others	1.0

TABLE A-3

HORIZONTAL FORCE FACTOR "C_p" FOR ELEMENTS OF STRUCTURES STRUCTURES AND NONSTRUCTURAL COMPONENTS

PART OR PORTION OF BUILDINGS	DIRECTION OF FORCE	VALUE OF C_p[2]
1. Exterior bearing and nonbearing walls, interior bearing walls and partitions, interior nonbearing walls and partitions over 3 meters in height – see also Section 2.1 (j) 3C. Masonry or concrete fences over 2 meters in height.	Normal to flat surface	0.30[1]
2. Cantilever elements a) Parapets b) Chimneys or stocks	 Normal to flat surface Any direction	0.80
3. Exterior and interior ornamentations and appendages	Any direction	0.80
4. When connected to, part of, or housed within a building: a) Penthouses, anchorage & supports for chimneys, stacks and tanks, including contents b) Storage racks with upper storage level at more than 2.50 meters in height plus contents c) All equipment or machinery	Any direction	0.30[3,4]
5. Suspended ceiling framing systems	Any direction	0.3[5,7]
6. Connections for prefabricated structural elements other than walls, with force applied at center of gravity of assembly	Any direction	0.30[6]

[1] A minimum of 240 Pa shall be applied perpendicular to the walls. The deflection of such walls under a load of 240 Pa shall not exceed 1/240 of the span for walls with brittle finishes and 1/120 of the span for walls with flexible finishes.

[2] When located in the upper portion of any building where the h_n/D ratio is five-to-one or greater the value shall be increased by 50 percent. Cp for elements laterally self-supported only at the ground level may be two thirds of the value shown.

[3] W_p for storage racks shall be the weight of the racks plus contents. The value of C_p for racks over two storage support levels in height shall be 0.24 for the levels below the top two levels. In lieu of the tabulated values steel storage racks may be designed in accordance with U.B.C. Standard No. 27-11.

Where a number of storage rack units are interconnected so that there are a minimum of four vertical elements in each direction on each column line designed to resist horizontal forces, the design coefficients may be as for a building with K values from Table 2.1-A. CS = 0.20 for use in the formula V = ZIKCSW and W equal to the total deal load plus 50 percent of the rack rated capacity. Where the design and rack configurations are in accordance with this paragraph the design provisions in U.B.C. Standard No. 27-11 do not apply.

[4] For flexible and flexibly mounted equipment and machinery, the appropriate values of C_p shall be determined with consideration given to both the dynamic properties of the equipment and machinery and to the building or structure in which it is placed but shall not be less than the listed values. The design of the equipment and machinery and their anchorage is an integral part of the design and specification of such equipment and machinery.

For Essential Facilities and life safety systems, the design and detailing of equipment which must remain in place and be functional following a major earthquake shall consider drifts in accordance with Sec. 2.1 (k).

[5] Ceiling weight shall include all light fixtures and other equipment which are laterally supported by the ceiling. For purposes of determining the lateral force, a ceiling weight of not less than 200 Pa shall be used.

[6] The force shall be resisted by positive anchorage and not by friction.

[7] Does not apply to ceilings constructed of lath and plaster or gypsum board screw or nail attached to suspended members that support a ceiling at one level to w_x extending from wall to wall.

APPENDIX B: NZS 4203:1984 HIGHLIGHTS

I. Base Shear

A. Formulas

$$V = C_d W_t \tag{B-1}$$

where

$$Cd = CRSM \tag{B-2}$$

$$T = \pi\sqrt{\{\Sigma (W_x d_x^2)/[g\Sigma (F_x d_x)]\}} \tag{B-3}$$

or, approximately,

$$T = 0.063\sqrt{\Delta} \tag{B-4}$$

B. Definitions of Flexible Subsoils

A subsoil is considered flexible if there are uncemented soils exceeding one of the following depths below the lowest continuous horizontal subsystem:

6 m of cohesive soils with average undrained shear strength not exceeding 50 kPa

8.5 m of cohesive soils with average undrained shear strength not exceeding 100 kPa

12 m of cohesive soils with average undrained shear strength not exceeding 200 kPa

15 m of cohesionless sands or gravels

II. Distribution of Lateral Forces Among Floors

$$F_x = (VW_x h_x)/[\Sigma (W_x h_x)] \tag{B-5}$$

III. Lateral Forces on Structural and Non-Structural Elements or Components

$$F_p = C_p W_p \qquad \text{(B-6)}$$

where R_p is available from Table B-5

$$C_p = 1.5\, K_x S_p M_p R_p C_d \qquad \text{(B-7)}$$

subject to $\alpha K_x Z R C_{p\ min} \leq C_p \leq \alpha K_x Z R C_{p\ max}$

or where no R_p value is available

$$C_p = \alpha K_x Z R C_{p\ max} \qquad \text{(B-8)}$$

where

K_x = 1 for single-storey buildings; & h_x/h_{cg} for multi-storey buildings with h_x and h_{cg} being the heights to the x level and the center of gravity of W_t.

α = 1 for single storey buildings; & h_x/h_{cg} for multi-storey buildings

S_p is from Table B-4

M_p is from Table B-3

$C_{p\ max}$, $C_{p\ min}$, & R_p are from Table B-9

R is from Table B-1

Z is 1 for seismic zone A; 5/6 for zone B & 2/3 for seismic zone C

C_d = CRSM for the supporting building.

TABLE B-1

RISK FACTOR *R*

Category	*Description*	*R*
1	Structures containing highly hazardous contents.	2.0
2a	Buildings which are intended to remain functional in the Emergency Period for major earthquakes.	1.6
2b	Buildings whose failure could cause high loss of life in the surrounding area.	1.6
3a	Buildings which should be functioning in the Restoration Period for major earthquakes.	1.3
3b	Buildings whose contents have a high value to the community.	1.3
4	Buildings with normal occupancy or usage.	1.0

TABLE B-2

STRUCTURAL TYPE FACTOR S

Item	*Description*	S
1	Ductile frames	0.8
2	Ductile coupled shear walls: (a) $A \geqslant 0.67$ (b) $A \leqslant 0.33$ (c) $0.33 \leqslant A \leqslant 0.67$	 $0.8Z \leqslant 1.6$ $1.0Z \leqslant 2.0$ By linear interpolation between 2(a) and 2(b)
3	Ductile cantilever shear walls. Single-storey ductile columns (a) Two or more elements linked together (b) Single element	 $1.0Z \leqslant 2.0$ $1.2Z \leqslant 2.0$
4	Frames of limited ductility of maximum height four storeys or 18 m or, with top storey, roof and wall mass less than 150 kg/m^2, five-storeys or 22.5 m. Cantilevered shear walls of limited ductility.	2.0
5	Buildings with diagonal bracing: (i) Capable of plastic deformation in tension only: (a) Single storey (b) Two or more storeys (c) More than three storeys (ii) Capable of plastic deformation in both tension and compression.	 2.0 2.5 or by special study By special study 1.6 or by special study
6	Single-storey cantilevered buildings supported by face loaded walls constructed of reinforced masonry or concrete.	2.0
7	Elastically responding structures: (a) Reinforced concrete (b) Reinforced masonry (c) Prestressed concrete (d) Steel	 5.0 4.0 5.0 6.0

Where A is the proportion of total overturning moment resisted by all beams (moments referred to the centroidal axes of all walls) and where:

Z $= 3.0 - h_w / l_w$ subject to $1 \leqslant Z \leqslant 2$

h_w is the height from base of the wall to top of uppermost principal storey.

l_w is the horizontal length of the wall in the direction of the applied load.

TABLE B-3

STRUCTURAL MATERIAL FACTOR M or M_p

Item	*Material*	*M or M_p*
1	Structural steel	0.8
2	Structural timber: As given by table 5B	
3	Reinforced non-prestressed concrete	0.8
4	Prestressed concrete	1.0
5	Reinforced masonry	1.0

TABLE B-3a

SM or $S_p M_p$ FACTORS FOR TIMBER

Item	*Description*	*SM or $S_p M_p$*
B1	Shear walls or diaphragms:	
	(a) Ductile	1.0
	(b) Ductile and stiffened with elastomeric adhesive	1.0
	(c) Limited ductility fixed with elastomeric adhesive	1.2
B2	Moment resisting frames:	
	(a) Ductile with an adequate number of possible plastic beam hinges	1.2
	(b) As for item B2 (a) but with connections of limited ductility	1.5
B3	Diagonally braced with timber members capable of acting as struts or ties:	
	(a) With ductile end connections	1.7
	(b) With end connections having limited ductility	2.0
B4	Elastically responding structures	2.4

TABLE B-4

STRUCTURAL TYPE FACTOR S_p FOR A PART OR PORTION OF A BUILDING

Item		*Description*	S_p
1	(a)	Adequately reinforced walls and partition walls subjected to face loads	
	(b)	Floor and roof diaphragms (see clause 3.4.6.3)	1.0
	(c)	Other principal members distributing seismic forces such as reinforced concrete or reinforced masonry wall bands detailed for ductility	
	(d)	Other reasonably ductile parts	
2		Diaphragms with diagonal bracing capable of deformation in tension only and detailed in accordance with clause 3.3.2.2	1.2
3	(a)	Principal members distributing seismic forces such as reinforced concrete or reinforced masonry wall bands not detailed for ductility	
	(b)	Tied veneers subject to face loadings	2.0
4		Unreinforced or partially reinforced walls and partitions subject to any loading	3.0

TABLE B-5

SEISMIC FORCE FACTORS FOR PARTS AND PORTIONS OF BUILDINGS (see clause 3.4.9)

Item	*Part or portion*	*Direction of force*	C_p *max.*	R_p	C_p *min.*
1	Walls, partition walls, infilling panels, exterior prefabricated panels:	Normal to face			
	(a) Adjacent to an exitway, street, or public place, or required to have a fire resistance rating of 2 h or more.				
	(i) Single-storey buildings:				
	S_p less than or equal to 1		0.3		
	S_p greater than 1		0.9		
	(ii) Multi-storey buildings:				
	S_p less than or equal to 1		0.5	1.1/3	0.3
	S_p greater than 1		1.8	1.1/3	0.6
	(b) Others:				
	(i) Single-storey buildings:				
	S_p less than or equal to 1		0.2		
	S_p greater than 1		0.6		
	(ii) Multi-storey buildings:				
	S_p less than or equal to 1		0.4	1.0	0.3
	S_p greater than 1		1.3	1.0	0.6
2	Vertical cantilevers and elements fastened to them.	Normal to face of wall			
	(a) Horizontal restraint from ductile cantilevered columns or walls on				
	(i) One-storey building, and cantilevered top storey in a two-storey building		0.3		
	(ii) Multi-storey buildings and other cantilevered top storeys		0.6	1.5	0.3
	(b) Horizontal restraint of lesser ductility				
	(i) One storey building, and cantilevered top storey in a two-storey building		0.6	2.0	0.4
	(ii) Multi-storey buildings and other cantilevered top storeys		1.3	2.0	0.9
3	Ornamentations, tied veneers, appendages:	Any horizontal			
	(i) Single-storey buildings		1.0		1.0
	(ii) Multi-storey buildings		2.0		2.0
4	Connections for exterior pre-fabricated panels (item 1) and all items 3:				
	(i) Single- storey buildings		1.0		
	(ii) Multi-storey buildings		2.0		
5	Horizontally cantilevered floors, beams etc. (see clause 3.6.3):	Vertically downward or upward	0.9		
6	Floors and roofs acting as diaphragms and other primary elements distributing seismic forces, see clause 3.4.6.3:	Any horizontal			
	(a) Single-storey buildings				
	S_p equal to 1.0		0.2		
	S_p equal to 1.2		0.25		
	S_p greater than 1.2		0.4		
	(b) Multi-storey buildings				
	S_p equal to 1.0		0.3		
	S_p equal to 1.2		0.4		
	S_p greater than 1.2		0.6		
7*†	Towers not exceeding 10% of the mass of the building. Tanks and full contents, not included in item 8 or item 9; chimneys and smoke stacks and penthouses connected to or part of the building except where acting as vertical cantilevers:	Any horizontal			
	(a) Single-storey buildings where the height to depth ratio of the horizontal force resisting system is:				
	(i) Less than or equal to 3		0.2		
	(ii) Greater than 3		0.3		

Item	Part or portion	Direction of force	C_p max.	R_p	C_p min.
	(b) Multi-storey buildings where the height to depth ratio of the horizontal force resisting system is:				
	(i) Less than or equal to 3		0.3		
	(ii) Greater than 3		0.5		
8*†	Containers and full contents and their supporting structures; pipelines, and valves:	Any horizontal			
	(a) For toxic liquids and gases, spirits, acids, alkalis, molten metal, or poisonous substances, liquid and gaseous fuels including containers for materials that could form dangerous gases if released:				
	(i) Single-storey buildings		0.6	2.0	0.5
	(ii) Multi-storey buildings		1.3	2.0	0.9
	(b) Fixed fire fighting equipment including fire sprinklers, wet and dry riser installations and hose reels:				
	(i) Single-storey buildings		0.5	1.5	0.3
	(ii) Multi-storey buildings		1.0	1.5	0.6
	(c) Others:				
	(i) Single-storey buildings		0.3	1.0	0.2
	(ii) Multi-storey buildings		0.7	1.0	0.4
9*†*	Furnaces, steam boilers, and other combustion devices, steam or other pressure vessels, hot liquid containers; transformers and switchgear; shelving for batteries and dangerous goods:	Any horizontal			
	(i) Single-storey buildings		0.6	2.0	0.5
	(ii) Multi-storey buildings		1.3	2.0	0.9
10*†	Machinery; shelving not included in item 9; trestling, bins, hoppers, electrical equipment not specifically included in other items 8, 9 or 11; other fixtures:	Any horizontal			
	(i) Single-storey buildings		0.3	1.0	0.2
	(ii) Multi-storey buildings		0.7	1.0	0.3
11†	Lift machinery, guides etc; emergency standby equipment.	Any horizontal	0.6		
12†	Connections for items 8 to 11 inclusive shall be designed for the specified forces provided that the gravity effects of dead and live loads shall not be taken to reduce these forces.				
13†	Suspended ceilings including attached equipment, lighting and attached partitions, see clause 3.6.5.	Any horizontal	0.6		
14	Communications, detection or alarm equipment for use in fire or other emergency:				
	(i) Single-storey buildings,	Any horizontal	0.5	1.5	0.3
	(ii) Multi-storey buildings,		1.0	1.5	0.6

*See clause 3.6.4.
†See clause 3.6.6.

APPENDIX C: YUGOSLAV CODE, 1981 HIGHLIGHTS

I. Equivalent Lateral Force Analysis

A. Base Shear

$$S = KG \tag{C-1}$$

where

$$K = K_o K_s K_d K_p \tag{C-2}$$

≥ 0.02

K_o from Table C-1

K_s from Table C-2

K_d from Table C-3

K_p from Table C-5

G = dead load, probably live load, snow load, and weight of permanently attached equipment

II. Distribution of Base Shear

A. Structures up to 5 stories high

$$S_i = (G_i H_i)/[\Sigma\,(G_i H_i)] \tag{C-3}$$

B. Structures more than 5 stories high 15% of S applied on n-th floor with the remainder distributed according to Eqn. (C-3)

III. Torsional Moment

$$M_{t,i} = Q_i e_i K_t \tag{C-4}$$

Q_i = larger of horizontal seismic forces in two directions at i-th floor

e_i = distance between center of mass and center of rigidity at i-th floor

K_t = coefficient of eccentricity increase due to coupling of translational and torsional modes and due to unequal displacements of footings. If not analytical determination is made, to be taken = 1.5

IV. Lateral Forces on Structural & Non-Structural Elements

$$S = K_s K_e G_e \tag{C-5}$$

where K_s is from Table C-2

K_e is from Table C-6

G_e is the weight of structural element for which lateral force is being calculated.

V. Vertical Seismic Force

$$S = K_v G \tag{C-6}$$

where $K_v = 0.7K$

K from Eqn. (C-2)

G = as defined in the foregoing

TABLE C-1

VALUES OF K_o

Category of the structure	Kind of the Structure	Category coefficient of the structure K_o
Out of Category	Building structures within the technological solution of nuclear power plants; structures for transportation and storage of inflammable gas; warehouses for toxic materials; energetic structures with installed power over 10 MW; industrial chimneys; more important structures for connections and telecommunications, high buildings over 25 storeys, and other building structures on which accuracy depends the functioning of other technical-technological systems, in which the disorder can cause catastrophic consequences, i.e. to cause enormous material losses to the wider economic community.	
I Category	Buildings with halls planned for gathering of many people (cinema halls, theatres, sport, exhibition and other halls); faculties, schools; health facilities; fire stations, structures for connections not included in the previous category (PTT, Radio and Television Studios and other); industrial buildings with valuable equipment; all energetic structures with installed power up to 10 MW; buildings with things of rear cultural and art value and other buildings in which activities of special interest for the economic-political community are organised.	1.5
II Category	Residential buildings; restaurants; public buildings not classified in Category I; industrial buildings not classified in Category I.	1.0
III Category	Auxiliary-production buildings; agro-technical structures	0.75
IV Category	Temporary structures whose destruction does not endanger human lives.	

TABLE C-2

VALUES OF K_s

MCS Degree*	K_s
VII	0.025
VIII	0.050
IX	0.100

*Seismicity based on Mercalli, Cancani, Sieberg Scale

TABLE C-3

VALUES OF K_d

Soil Category*	Coefficient	Limit values of the Coefficient K_d
I	$K_d = \frac{0,50}{T}$	$1,0 > K_d < 0,33$
II	$K_d = \frac{0,70}{T}$	$1,0 > K_d < 0,47$
III	$K_d = \frac{0,90}{T}$	$1,0 > K_d < 0,60$

*Refer to Table C-4

TABLE C-4

SOIL CATEGORIES

Soil Category	Characteristic soil profile
I	Rocky and semi-rocky soils (christal rocks, shales, Carbonate rocks, lime, marl-stone, well cemented conglomerates and other). Very dense and hard soils with thickness smaller than 60 m., of stable gravel layers, sand and hard clay over a solid geological formation.
II	Stiff and semi-hard soils, as well as very dense and hard soils with thickness bigger than 60 m of stable gravel layers, sand and hard clay over a solid geological formation.
III	Slightly dense and soft soils of over 10 m thickness, of loose gravel, medium dense sand and soft-to-medium stiff clay, with or without layers of sand and other cohesionless soil materials.

The construction site of high-rise structures of the categories I and II the soil conditions of which are not well known can be classified in the Soil Category II.

TABLE C-5

VALUES OF K_p

1. For all contemporary structures of R. C., for all steel structures, except for the structures named in item 2 of this article, and for all contemporary timber structures, except for the structures named in item 3 of this article, and amounts 1;
2. For structures with reinforced concrete walls and braced steel structures, it amounts 1,3;
3. For masonry structures strengthened with vertical columns of reinforced concrete; for very high and slim structures with small damping, as high as industrial chimneys, antennae, water towers and other structures with natural period of oscillation $T \geq 2,0$ sec., amounts 1,6;
4. For structures with flexible ground floor or first floor, i.e. with abrupt change of the rigidity, as well as for structures with plane walls, it amounts 2,0.

TABLE C-6

K_e VALUES

Structural elements	K_e	Effect direction
– in-fill walls, nonbearing walls	2,5	orthogonally on the surface
– balconies	6,0	orthogonally on the surface
– chimneys and reservoirs on the structure	6,0	in any direction
– masonry parapets, fences	10,0	orthogonally on the surface
– ornaments	10,0	in any direction

APPENDIX D: IS:1893-1975 HIGHLIGHTS

I. Equivalent Lateral Force Analysis

A. Base Shear (D-1)

$V_B = C\alpha_h W$

where

C is obtained from Figure 4

T is obtained from experimental observations on similar buildings or rational analysis

or,

T = $(0.09\, h_n)/(\sqrt{D})$ sec (D-2)

or,

T = 0.1N sec (D-3)

W = Total dead load + appropriate amount of live load

1. α_h by Seismic Coefficient Method

$\alpha_h = \beta I \alpha_o$ (D-4)

where β is obtained from Table D-1

α_o is obtained from Table D-2

I is obtained from Table D-3

2. α_h by Response Spectrum Method (D-5)

$\alpha_h = \beta I F_o (S_A/g)$

where β and I are obtained from Tables D-1 and D-2 as previously

F_o is taken from Table D-2

S_A/g is taken from Figure 5

II. Distribution of Base Shear among the different Floors

A. By equivalent static load analysis

$$Q_i = V_B [W_i h_i^2] / \Sigma [W_i h_i^2] \quad \text{(D-6)}$$

where Q_i = i-th Floor Load

V_B = Base Shear

W_i = Weight of i-th floor

h_i = height of i-th floor above base

B. By Spectral Modal Analysis

$$Q_i = W_i \phi_i^{(r)} C_r \alpha_h \quad \text{(D-7)}$$

where $\phi_i^{(r)}$ = free vibration mode shape coefficient of i-th floor

C_r = mode participation factor

$= [\Sigma(W_i \phi_i^{(r)})] / [\Sigma\{W_i (\phi_i^{(r)})^2\}]$

other factors as defined previously

$$V_i = (1 - \tau) \Sigma V_i^{(r)} + \tau \sqrt{\{V_i^{(r)}\}^2} \quad \text{(D-8)}$$

where $V_i^{(r)}$ = shear force at i-th floor

τ as given below:

Height (m)	τ
up to 20	0.40
40	0.60
60	0.80
90	1.00

τ for intermediate values of height may be obtained by linear interpolation.

TABLE D-1

VALUES OF β FOR DIFFERENT SOIL-FOUNDATION SYSTEMS

(Clause 3.4.3)

SL No.	TYPE OF SOIL MAINLY CONSTITUTING THE FOUNDATION	VALUES OF β FOR					
		Piles Passing Through Any soil, But Resting on Soil Type 1	Piles Not Covered Under Col 3	Raft Foundations	Combined or Isolated RCC Footings with Tie Beams	Isolated RCC Footings Without Tie Beams or Unreinforced Strip Foundations	Well Foundations
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	Type I Rock or Hard Soils	1.0	—	1.0	1.0	1.0	1.0
ii)	Type II Medium Soils	1.0	1.0	1.0	1.0	1.2	1.2
iii)	Type III Soft Soils	1.0	1.2	1.0	1.2	1.5	1.5

NOTE — The value of β for dams shall be taken as 1.0.

TABLE D-2

VALUES OF BASIC SEISMIC COEFFICIENT AND SEISMIC ZONE FACTORS IN DIFFERENT ZONES

(Clauses 3.4.2.1., 3.4.2.3 and 3.4.5)

SL No.	ZONE No.	METHOD: Seismic Coefficient Method — Basic horizontal seismic Coefficient, α_0	METHOD: Response Spectrum Method (see Appendix F) — Seismic zone factor for average acceleration spectra to be used with Figure 2, F_0
(1)	(2)	(3)	(4)
i)	V	0.08	0.40
ii)	IV	0.05	0.25
iii)	III	0.04	0.20
iv)	II	0.02	0.10
v)	I	0.01	0.05

NOTE — For underground structures and foundations at 30 m depth or below, the basic seismic coefficient may be taken as $0.5\,\alpha_0$; for structures placed between ground level and 30 m depth, the basic seismic coefficient may be linearly interpolated between α_0 and $0.5\,\alpha_0$.

The seismic coefficients according to 3.4.2.1 for some important towns and cities are given in Appendix E.

TABLE D-3

TABLE 4 VALUES OF IMPORTANT FACTOR, I

(Clause 3.4.4)

SL No.	STRUCTURE	VALUE OF IMPORTANT FACTOR, I (see Note)
(1)	(2)	(3)
i)	Containment structures of atomic power reactors for preliminary design	6.0
ii)	Dams (all types)	2.0
iii)	Containers of inflammable or poisonous gases or liquids	2.0
iv)	Important service and community structures, such as hospitals; water towers and tanks; schools; important bridges; important power houses; monumental structures; emergency buildings like telephone exchange and fire brigade; large assembly structures like cinemas, assembly halls and subway stations	1.5
v)	All others	1.0

NOTE — The values of importance factor, I, given in this table are for guidance. A designer may choose suitable values depending on the importance based on economy strategy and other considerations.

APPENDIX E: ATC 3-06 HIGHLIGHTS

I. Equivalent Lateral Force Analysis

A. Base Shear

$$V = C_c W \tag{E-1}$$

where W = total gravity load of structure consisting of total weight of structure including partitions and permanent equipment, 25% of floor live load in the case of storage and warehouse structures, applicable effective snow load

$$C_s = (1.2A_v S) / (RT^{2/3}) \tag{E-2}$$

where A_v = velocity-related acceleration coefficient
R = response modification factor, Table E-4
S = soil profile characteristic coefficient, Table E-5
T = natural period of building as described in the following.

or when period of building is not calculated:

$$C_s = 2.5A_A/R \tag{E-3}$$

or, when $A_A \geq 0.3$:

$$C_s = 2A_A/R \tag{E-4}$$

where A_A = effective peak acceleration coefficient

B. Natural Period Determination

1. Rayleigh Formula
2. Approximate Methods

a. **Moment-resisting structures not enclosed by more rigid components preventing the frames from deflecting under seismic forces:**

$$T_A = C_T h_n^{3/4} \tag{E-5}$$

where C_T = 0.035 for steel frames
= 0.025 for concrete frames
h_n = height in ft to highest level

b. All other buildings:

$$T_A = (0.05h_n) / \sqrt{L} \tag{E-6}$$

where L = length in ft of building in direction under consideration

II. Vertical Distribution of Base Shear

$$F_x = C_{vx} V \tag{E-7}$$

where $C_{vx} = (W_x h_x^k) / (\Sigma W_x h_x^h)$ (E-8)

III. Modal Analysis

A. Modal Base Shear

$$V_m = C_{sm} W_m \tag{E-9}$$

$$\text{where } C_{sm} = (1.2A_v S) / (RT_m^{2/3}) \tag{E-10}$$

$$\leq 2.3A_A/R$$

$$W_m = [\Sigma (W_i \phi_{im})]^2 / \Sigma (W_i \phi^2) \tag{E-11}$$

B. Modal Forces, Deflections and Drifts

$$F_{wm} = C_{vxm} V_m \tag{E-12}$$

$$\text{where } C'_{vxm} = (W_x \phi_m) / \Sigma (W_i \phi_{im}) \tag{E-13}$$

$$\delta_{xm} = C_d \delta_{xEm} \tag{E-14}$$

$$\text{where } \delta_{xEm} = (g/4\pi^2) [(T_m^2 F_{xm}) / W_x] \tag{E-15}$$

TABLE E-1

COEFFICIENTS A_a AND A_v AND SEISMICITY INDEX

Coeff. A_a Figure 1	Map Area Number	Coeff. A_v Figure 2	Seismicity Index
0.40	7	0.40	4
0.30	6	0.30	4
0.20	5	0.20	4
0.15	4	0.15	3
0.10	3	0.10	2
0.05	2	0.05	2
0.05	1	0.05	1

TABLE E-2

Group III: Buildings housing critical facilities which are necessary to post-disaster recovery and require continuous operation during and after an earthquake. The term critical facilities and emergency is defined as meaning designated by the governmental entity having jurisdiction.

Examples:

Fire facilities
Police facilities
Hospital facilities with emergency treatment facilities
Emergency preparedness centers
Emergency communications center
Power stations and other utilities required as emergency facilities

Group II: Buildings housing dense occupancies having a high transient population and/or sleeping conditions, or critical facilities requiring operation in the immediate post-disaster period. Restricted movement facilities.

Examples:

Public assembly for 100 or more persons
Open air stands for 2,000 or more persons
Day care
Schools
Colleges
Retail stores $\geq$ 5,000 square feet floor area per floor or more than 35 feet in height

Shopping centers with covered malls over 20,000 square feet gross area excluding parking
Offices over four stories in height or more than 10,000 square feet per floor
Hotels over four stories in height
Apartments over four stories in height
Emergency vehicle garages
Detention facilities
Ambulatory health facilities
Hospital facilities other than those in Group III
Wholesale stores over four stories in height
Factories over four stories in height
Printing Plants over four stories in height
Hazardous occupancies consisting of flammable or toxic gases, flammable or toxic liquids including storage facilities for same

Group I: Low density occupancies and generally low transient population.

Examples:
Aircraft hangars
Woodworking facilities
Factories four stories or less
Repair garages
Service stations
Storage garages
Wholesale stores four stories or less
Printing plants four stories or less
Ice plants
Dwellings, single and two family
Townhouses
Retail stores less than 5,000 square feet per floor and 35 feet or less in height
Public Assembly for less than 100 persons
Offices four stories or less in height or less than 10,000 square feet per floor
Hotels four stories or less in height
Apartment houses four stories or less in height

Multiple-Occupancy Structures

At some time in the future, judging from recent architectural trends, mega-structure type buildings with multiple-occupancy groups will be designed or constructed. Due to economic pressures on the cost of construction, cost of travel and high values of land, shopping, living, entertainment, medical, and working facilities may be combined and designed into a single structure. Any "preconceived boxes", or occupancy classifications within which buildings are classified must be designed to take into consideration the possibility of multiple-occupancy type structures. Some of the new convention centers or regional shopping center malls, are in this category and represent a high-occupancy risk situation.

In this case it was concluded that the architectural systems and components are even more critical than in conventional type buildings. Egress and accessibility to these structures are most important.

TABLE E-3

SEISMIC PERFORMANCE CATEGORY

Seismicity Index	Seismic Hazard Exposure Group III	II	I
4	D	C	C
3	C	C	B
2	B	B	B
1	A	A	A

TABLE E-4

RESPONSE MODIFICATION COEFFICIENTS[1]

Type of Structural System	Vertical Seismic Resisting System	Coefficients R[7]	C_d[8]
BEARING WALL SYSTEM: A structural system with bearing walls providing support for all, or major portions of, the vertical loads. Seismic force resistance is provided by shear walls or braced frames.	Light framed walls with shear panels	6½	4
	Shear walls		
	Reinforced concrete	4½	4
	Reinforced masonry	3½	3
	Braced frames	4	3½
	Unreinforced and partially reinforced masonry shear walls[6]	1¼	1¼
BUILDING FRAME SYSTEM: A structural system with an essentially complete Space Frame providing support for vertical loads. Seismic force resistance is provided by shear walls or braced frames.	Light framed walls with Shear panels	7	4½
	Shear walls		
	Reinforced concrete	5½	5
	Reinforced masonry	4½	4
	Braced frames	5	4½
	Unreinforced and partially reinforced masonry shear walls[6]	1½	1½
MOMENT RESISTING FRAME SYSTEM: A structural system with an essentially complete Space Frame providing support for vertical loads. Seismic force resistance is provided by Ordinary or Special Moment Frames capable of resisting the total prescribed forces.	Special moment frames		
	Steel[3]	8	5½
	Reinforced concrete[4]	7	6
	Ordinary moment frames		
	Steel[2]	4½	4
	Reinforced concrete[5]	2	2
DUAL SYSTEM: A structural system with an essentially complete Space Frame providing support for vertical loads. A Special Moment Frame shall be provided which shall be capable of resisting at least 25 percent of the prescribed seismic forces. The total seismic force resistance is provided by the combination of the Special Moment Frame and shear walls or braced frames in proportion to their relative rigidities.	Shear walls		
	Reinforced concrete	8	6½
	Reinforce masonry	6½	5½
	Wood sheathed shear panels	8	5
	Braced frames	6	5
INVERTED PENDULUM STRUCTURES. Structures where the framing resisting the total prescribed seismic forces acts essentially as isolated cantilevers and provides support for vertical load.	Special Moment Frames		
	Structural steel[3]	2½	2½
	Reinforced concrete[4]	2½	2½
	Ordinary Moment Frames		
	Structural steel[2]	1¼	1¼

[1] These values are based on best judgement and data available at time of writing and need to be reviewed periodically.
[2] As defined in Sec. 10.4.1.
[3] As defined in Sec. 10.6.
[4] As defined in Sec. 11.7.
[5] As defined in Sec. 11.4.1.
[6] Unreinforced masonry is not permitted for portions of buildings assigned to Category B. Unreinforced or partially reinforced masonry is not permitted for buildings assigned to Categories C and D; see Chapter 12.
[7] Coefficient for use in Formula 4-2, 4-3, and 5-3.
[8] Coefficient for use in Formula 4-9.

TABLE E-5

SOIL PROFILE COEFFICIENT

	Soil Profile Type & Description	Coefficient S
S_1 –	Rock of any characteristic with shear wave velocity $>$ 2500 ft/sec Stiff soil with soil depth $<$ 200 ft and soil overlying rock are stable deposits of sands, gravels or stiff clays	1.0
S_2 –	deep cohesionless soil or stiff clay with soil depths in excess of 200 ft and soils overlying rock are stable deposits of sands, gravels or stiff clay	1.2
S_3 –	soft to medium-stiff clays and sands 30 ft or more in thickness with or without intervening layers of sands or cohesionless soils	1.5

TABLE E-6

ALLOWABLE STORY DRIFT

	Seismic Hazard Exposure Group		
	III	II	I[1]
Δ_A	$0.01h_{sx}$	$0.015h_{sx}$	$0.015h_{sx}$

[1]Where there are no brittle-type finishes in buildings three stories or less in height these limits may be increased by one-third.