AS 1170.4 Supplement 1—1993

Minimum design loads on structures

Part 4: Earthquake loads—Commentary

(Supplement to AS 1170.4—1993)

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AS 1170.4 Supplement 1—1993®

Minimum design loads on structures (known as the SAA Loading Code)

Part 4: Earthquake loads—Commentary

(Supplement to AS 1170.4—1993)

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PREFACE

This Commentary was prepared by the Standards Australia Committee for Loading on Structures as a Supplement to AS 1170, SAA Loading Code, *Minimum design loads on structures*, Part 4: *Earthquake loads*.

The Supplement provides background material for and guidance to the requirements of the Standard.

The clause numbers of this Commentary are prefixed by the letter 'C' to distinguish them from references to the Standard clauses to which they directly relate. The Commentary clause numbers are not sequential because if no explanation of the Standard clause is necessary that clause number is not included.

Standards Australia and Standards New Zealand are moving towards harmonization of Standards in both countries and the harmonization of the building codes is taking place first. Further changes to the Standard are therefore expected but not before 1995.

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Minimum design loads on structures

Part 4: Earthquake loads—Commentary

(Supplement to AS 1170.4)

SECTION C1 SCOPE AND GENERAL

C1.1 SCOPE The Standard considers loads on structures due to earthquake ground motion. The provisions are not intended to prevent damage due to surface rupture, landslip or liquefaction. Nor do they consider other secondary earthquake effects such as fire, loss of services, seiche or tsunami. They do not consider site-structure resonance, where the natural frequency of a site is the same as that of the structure. They do not consider pounding of adjacent structures moving out of phase in an earthquake. These aspects require consideration of issues other than 'loads' and therefore are inappropriate in a loading Standard.

Special structures require special consideration of their response characteristics and environment that is beyond the scope of these provisions. Special structures such as nuclear reactors and cyrogenic tanks, which may present a considerable hazard to the community in the event of their failure, are not covered by the Standard, nor are industrial facilities of high economic importance. In either event, such structures require consideration of strong earthquake motions of very low probability, regardless of location.

Structures with unusual structural characteristics, or of novel design or construction, are not excluded by the Standard. However, the Standard may be of little assistance in the design of such structures since a structural response factor (R_f) may not have been assigned.

Bridges are not covered by the Standard but could be included with some additional requirements.

Agricultural buildings may be excluded from earthquake consideration because of the exceptionally low risk to life involved.

Alterations to existing structures are covered by these provisions (see Section 8) but the assessment, strengthening and repairs to existing structures are not covered. However, guidance and some information are included in Appendix E.

C1.2 REFERENCED DOCUMENTS The Standards listed in the Clause are subject to revision from time to time and the current issue should always be used. The currency of any Standard may be checked with Standards Australia.

C1.3 DEFINITIONS The Clause gives the definition of some essential terms used in the Standard. The meanings of the terms will become clear with the reading of the appropriate clauses. A number of the terms defined are common to other Standards such as AS 1170.1, AS 3600, AS 3700 and AS 4100.

C1.6 EARTHQUAKE LOAD COMBINATIONS Earthquake loads are the horizontal and vertical ground displacements caused by an earthquake action. These displacements induce inertial forces in a structure, related to its mass and rigidity distribution, which in turn induce action effects, i.e. axial forces, shear forces and bending moments, in its structural members.

These earthquake loads are ultimate loads even when defined in terms of unfactored loads.

SECTION C2 GENERAL REQUIREMENTS

C2.3 ACCELERATION COEFFICIENT The acceleration coefficient *(a)* can best be understood by considering it as a normalizing factor for construction of smoothed elastic response spectra for ground motions of normal duration. The nominal probability of the derived force level being exceeded is 10% in a 50-year design life, i.e. it corresponds to an average recurrence interval of approximately 500 years.

The development of the acceleration coefficient contour maps was facilitated by the work of Gaull, *et al* (Ref. 1). Several steps are involved in the preparation of such maps:

(a) Source zones in which earthquakes are known to occur are identified and brought together on a source zone map.

(b) For each source zone, the rate at which earthquakes of different magnitudes occur and the maximum credible magnitude are estimated.

(c) Attenuation relationships are used to give the intensity of motion as a function of magnitude and distance from an epicentre.

(d) A probabilistic analysis, with the above information as input, is carried out to generate values which are then used to produce contours of locations with equal probabilities of receiving specific intensities of ground motion. The calculated intensities apply to average ground conditions.

The main source of data was the Australian Geological Survey Organization (formerly Bureau of Mineral Resources) earthquake data file. This file contains locations of the Australian earthquakes from 1856.

Peak ground velocity was chosen as the intensity value thought to be the best predictor of damage. The acceleration coefficient is obtained by dividing peak ground velocity, in millimetres per second, by 750.

Critics of the earthquake risk approach argue that the historical record is far too short to justify the extrapolations inherent in the approach. Moreover, the procedure used assumes that earthquakes occur randomly in time so that the fact that a large earthquake has just occurred in an area does not make it less likely that a large earthquake will occur the following year. In the light of current understanding of earthquake occurrences, this assumption is of limited validity. However, at present there is no viable alternative approach to the construction of earthquake hazard maps that comes close to meeting the goal that the probability of exceeding the design ground motion should be roughly the same in all parts of the country. No part of Australia is free from the possibility of earthquake occurrence.

Acceleration coefficient maps Large potentially destructive earthquakes have occurred this century in eastern, central and western Australia and their spatial distribution indicates that they may occur anywhere in continental Australia. On the basis of earthquake over the last 30 to 100 years, they are less likely to occur in some areas than others, at least in the next 30 to 50 years.

The acceleration coefficient maps (see Figures 2.3(a) to (g)) depict contours of the acceleration coefficient with a 10% chance of being exceeded in 50 years. Since, by definition, return period is the inverse of the annual probability of not being exceeded, the maps depict a level of shaking with a return period of approximately 500 years. The computation of hazard was undertaken by Gaull, *et al* (Ref. 1) with peak ground velocity, acceleration and intensity as the variable. The velocity map was then smoothed and the numerical values normalized to produce an acceleration coefficient.

The smoothing was done after consideration of the gross geology and tectonics of Australia, and historical and instrumental seismicity in the period 1856 to 1990 (Refs. 2 and 3), much of which was not available to Gaull, *et al* (Ref. 1). It shows broad variations over Australia and is drawn at a scale which does not resolve individual faults nor variations from average ground conditions (firm foundations but neither rock nor soft soils).

Background Mainland Australia is not subject to the same frequency of potentially damaging earthquakes per unit area as the western USA, Japan or New Zealand. However, large earthquakes do occur. The most recent was near Tennant Creek in the Northern Territory on 22 January 1988 (Jones, *et al* (Ref. 4)) where earthquakes of magnitude 6.3, 6.4 and 6.7 occurred in a 12-hour period, buckling a buried natural-gas pipeline and causing building damage at Tennant Creek. It created a 35 km long, 2 m high fault scarp. The observed average frequency of an earthquake of magnitude 6 or more in Australia over the last 100 years is about 1 in 5 years.

Whilst these large earthquakes are potentially the most destructive, the highest contribution to earthquake hazard, as measured by the probability of exceeding a particular ground motion level, does not come from these infrequent large earthquakes, but from smaller more frequent and still potentially damaging earthquakes. This was amply demonstrated on 28 December 1989 at Newcastle where 13 people died and an estimated $1.5 billion damage was caused by a magnitude 5.5 earthquake. The apparent frequency of magnitude 5 or more earthquakes throughout Australia is about 1 in 6 months and, in the populous eastern States, 1 in 2 years.

The previous earthquake zone map of Australia in the *SAA Earthquake Code*, AS 2121—1979, included four seismic zones, Zero, A, 1 and 2, where different loading factors were to be applied. This procedure has been changed so that the new maps are presented as a set of contours and the interpolated acceleration coefficient (a) will be used by the designer to calculate the loading in the base shear equation for the following reasons:

(a) The basis for this re-evaluation of earthquake risk in Australia is the recently published study of Gaull, *et al* (Ref. 1). Their results for Modified Mercalli (MM) intensity, ground velocity and acceleration were in terms of a 10% chance of being exceeded in 50 years.

(b) Whilst the Cornell/McGuire probabilistic approach (Ref. 5) tacitly assumes that the best predictor of earthquake activity over the next 30 to 50 years is its distribution over the past 30 to 50 years, earthquakes can and do occur in places without a previous history of earthquake activity. The most recent example of this was the earthquake sequence near Tennant Creek in the Northern Territory in January 1988.

(c) The Committee subjectively smoothed, and where necessary extended, the contours of Gaull, *et al* (Ref. 1), as depicted in the accompanying map, in the light of earthquakes in the period 1984 to 1990 and recently discovered historical earthquakes not considered by them, and the broad correlation with geological, topographic and tectonic features. In recognition of the fact that no part of Australia is free from the possible occurrence of earthquakes, the contours were broadened to include the whole continent and appropriate reductions were made to the ‘bullseyes’ around past areas of high earthquake activity such as that in the Simpson Desert.

(d) The old zone boundaries were very imprecise but structures on opposite sides of the boundary were required to be designed to withstand horizontal earthquake loads which differed by a factor of two. In the Standard, interpolation between contours means that such anomalies will no longer occur.

(e) Peak ground velocity was chosen as the contour parameter, being a better predictor of damage or intensity than peak ground acceleration. The peak ground velocity (mm/s) was divided by 750 to determine the acceleration coefficient in the map and loading equation.

The only Australian territory comparable in earthquake risk to California or New Zealand is Macquarie Island, administratively part of Tasmania. It sits astride the Macquarie Ridge, a boundary between the Pacific and Australian plates. In May 1989 a great magnitude 8.3 earthquake occurred there, only 230 km north of the island. There are few structures on the island (a national park) but an acceleration coefficient of 0.3 (equivalent to the old Zone 3 status) is warranted there.

Data sources The main source of data was the Australian Geological Survey Organization (formerly the Bureau of Mineral Resources) earthquake data file which was compiled using data from all State and university seismological agencies and which now contains locations of the Australian earthquakes between 1856 and 1991. Data is reasonably complete from 1969 for magnitudes above 4, and from 1900 the data is complete only for large earthquakes (magnitude 6 and above).

Analysis Gaull, *et al* (Ref. 1) modified a computer program of Basham, *et al* (Ref. 6) which uses the Cornell/McGuire method (Ref. 5) to compute earthquake risk, and details can be found in this reference. Their results apply to average foundation conditions and do not allow for any ground motion modification caused by local foundation conditions, as are thought to have occurred at Newcastle and even more dramatically at Mexico City. Such conditions are taken into account in the site factor (S). In special cases, a microzonation study should be undertaken to determine the likely extent, frequency band and severity of the amplification.

The method does not take into consideration the proximity of active faults. If active faults could be identified, the ground motion recurrence may be significantly increased in some local areas.

The long period effects observed in the shaking of tall structures at great epicentral distances, such as in Adelaide and Perth during large earthquakes in Indonesia, and Adelaide and Cairns in the Tennant Creek sequence, have not previously caused damage and have not been taken into account in the risk analysis.

Aftershock activity Following an earthquake large enough to cause damage, aftershocks occur in the focal region of the mainshock but their pattern can show great variability as follows:

(a) A classical exponential decay of activity with time constant varying from months for a magnitude 5 earthquake to years for a magnitude 6 or greater mainshock. Examples include the 1968 Meckering and 1979 Cadoux earthquakes. The largest aftershock is often about half a magnitude unit smaller than the mainshock.

(b) Virtually no aftershocks or only a few micro-earthquakes in the weeks following the mainshock. The periods after the 1989 Newcastle and 1990 Meckering earthquakes characterized this behaviour.

(c) A few micro-earthquakes in the weeks following the mainshock but another mainshock sized earthquake 3 to 6 months after the first. This occurred in 1903 near Warrnambool, Victoria and in Central Australia at Marryat Creek in 1986.

(d) Multiple large earthquakes occurring over periods from hours to a year with thousands of smaller aftershocks during and after the sequence. The 1988 Tennant Creek and 1883/6 West Tasman Sea earthquake sequences are the type events for this behaviour. The Tasmanian sequence was apparently over when another isolated large earthquake occurred in 1892.

At this stage, there is no way to predict which particular pattern any earthquake sequence will follow but engineers and counter-disaster personnel should be aware of the risk of cumulative damage following any earthquake above magnitude 5.

The future As new earthquakes occur, these contour maps are bound to change, until a cause for intraplate earthquake occurrence is found. If sufficient strong motion data is recorded in intraplate regions over the next few years, future versions of the maps will be produced to give design response spectra rather than acceleration coefficients.

C2.4 SITE FACTOR The fact that the effects of local soil conditions on ground motion characteristics should be considered in the design of structures has long been recognized. Until the early 1980s, it was considered that the first three soil profile types listed in Table 2.4(a) and their corresponding site factor (S) provided an adequate coverage. Experience from the 1985 Mexico City and the 1989 Newcastle earthquakes, however, promoted the addition of the fourth soil profile where there is more than 20 m of soft clays or silts characterized by shear wave velocities of less than 150 m/s.

Spectral shapes representative of the different soil conditions were selected on the basis of a statistical study of the spectral shapes developed on such soils in past earthquakes (Seed, *et al* (Refs. 7 and 8)) and the appropriate site factors were derived from these.

The use of a simple site factor produces a direct approximation of the effects of local site conditions on the design requirements. This direct method eliminates the need for estimation of a predominant site period and computation of a soil factor based on the site period and the fundamental period of the structure, as was required in AS 2121.

Special dynamic analysis procedures should be adopted for structures judged to be susceptible to 'Mexico City' type damage on soft soil deposits, i.e. a resonance type response due to progressive lengthening of the structure period. NEHRP (Ref. 9) stated that this resonance-type response is most likely for structures having calculated periods greater than or equal to 0.7 s. Structures with natural periods of less than 0.5 s do not require these special analysis procedures.

Note that the 0.7 s calculated structure period is comparable to the calculated predominant soil period (T_{soil}) for soft soils considered as a single layer system using the following equation with h_n = 20 m and V_s = 120 m/s:

$$T_{soil} = \frac{4h_n}{V_s} \qquad \text{. . . C2.4}$$

where

h_n = total height of the structure above the structural base

V_s = total earthquake shear velocity

For loose sands, silts, or uncontrolled fill below the ground water table, account should be taken of the potential for liquefaction.

C2.5 IMPORTANCE FACTOR The Clause recognizes the need for special consideration to be given to certain structure classifications vital to public needs immediately following an earthquake. The amount of increase in earthquake base shear is the Committee's assessment of this increase. Higher increases may be necessary for specific conditions.

C2.6 EARTHQUAKE DESIGN CATEGORY The design requirements are constructed to reflect the relationship between the use of a structure and the level of earthquake motion to which it may be exposed. It is concerned mainly with life safety and the degree of exposure of the public to earthquake risk.

The specified requirements are the results of the Committee's collective judgment based largely on the previous *SAA Earthquake Code* (AS 2121) and on technical work from USA which was modified to suit Australian conditions.

C2.7 REQUIREMENTS FOR GENERAL STRUCTURES

C.2.7.1 General The terms used in the Section are defined in the subsequent Clause 2.7. It will be necessary, for each category, to refer to specific material Standards for particular structural requirements. Structures should also be designed to resist horizontal wind loads. For Australian conditions, the wind strength requirements may be more severe than the earthquake strength requirements. However, even for these cases, additional requirements for earthquake such as detailing and restriction on the use of certain materials still have to be considered.

C2.7.2 General structures of earthquake Design Category A Because of the relatively low hazard associated with earthquake Design Category A structures, it is considered appropriate to require only good quality construction materials and detailing. A large portion of Australian building stock will be of earthquake Design Categories A and B.

C2.7.3 General structures of earthquake Design Category B Because of the relatively low hazard associated with earthquake Design Category B structures, it is considered appropriate for regular ductile structures to require good quality construction materials and detailin. The detailing requirements for earthquake resistance relate to load paths, ties and wall anchorages. Non-ductile and irregular ductile structures need to be analyzed either by static analysis described in Section 6 or dynamic analysis described in Section 7.

Limits on the number of storeys are imposed if loadbearing masonry components are used. These limits depend on whether the masonry components are reinforced or unreinforced and whether the structure is regular or irregular. As specified in Clause 2.7.3(c), it is necessary to ensure that the deflections imposed on the unreinforced masonry will not precipitate failure of the unreinforced components.

No limitation is place on the use of non-structural masonry but the implications of earthquake induced deflections on its strength and stability should be checked.

C2.7.4 General structures of earthquake Design Category C Appreciable increases in earthquake resistance requirements are specified but are still quite simple compared with those structures of high earthquake hazard.

Limits on the number of storeys are imposed if loadbearing masonry components are used. These limits depend on whether the masonry components are reinforced or unreinforced. As specified in Clause 2.7.4(b), it is necessary to ensure that the deflections imposed on the unreinforced masonry will not precipitate failure of the unreinforced components.

No limitation is place on the used of non-structural masonry but the implications of earthquake induced deflections on its strength and stability should be checked.

C2.7.5 General structures of earthquake Design Category D In addition to the requirements of earthquake Design Category C, there are further structural detailing requirements and restriction on material requirements. For masonry structures, further strengthening of detailing, restrictions on height of unreinforced loadbearing masonry and minimum levels of reinforcement are required.

Limits on the number of storeys are imposed if loadbearing masonry components are used. These limits depend on whether the masonry components are reinforced or unreinforced. As specified in Clause 2.7.5(b), it is necessary to ensure that the deflections imposed on the unreinforced masonry will not precipitate failure of the unreinforced components.

No limitation is place on the use of non-structural masonry but the implications of earthquake induced deflections on its strength and stability should be checked.

For reinforced concrete structures, details that help to sustain integrity of the frames when subjected to deformation reversals into the non-linear range of response are required, i.e. frames should be at least semiductile. For braced frames, the connections should have a minimum amount of ductility and should have equivalent strengths in opposing directions to protect against progressive inelastic deformation accumulating in one direction. For timber structures, the use of certain materials and practices (e.g. screws, fibreboard diaphragms and eccentric timber joints) that have performed poorly in earthquakes should be limited.

C2.7.6 General structures of earthquake Design Category E Requirements for earthquake Design Category E correspond roughly to those for ordinary structures in highly earthquake-prone areas of USA. All masonry and concrete structures should be reinforced. All moment resisting frames of concrete or steel should meet the ductility requirements of special moment resisting frames over the specified heights.

C2.8 STRUCTURAL SYSTEMS OF BUILDINGS The Clause defines the general types of structural systems that may be used to resist earthquake forces. These systems are further classified in Table 6.2.6(a), and maximum values of R_f are specified for each classification. The R_f values have been assigned according to the relative capabilities of building systems to provide energy dissipation in the inelastic range.

In any structural system, it is preferable that the individual horizontal force resisting elements be located on separate vertical planes for each direction of loading. It is important that adequate resistance to torsion be provided both by the strength and arrangement of these elements in the plan of the structure.

The particular requirement necessary to qualify for a given type of structural system and its corresponding R_f value are as follows:

(a) *Bearing wall system* This type of structural system is characterized by shear wall or braced frame horizontal force resisting elements that carry substantial vertical loads and which, if they fail, eliminate the vertical load capacity of a substantial portion of the structure and lead to vertical instability. The presence of minor loadbearing walls in a structure that would normally be classified as a building frame system does not necessarily mean that the structure should be categorized as a bearing wall system. One reason, for example, is that the resistance of a multistorey structure is not significantly influenced by the presence of a minor portion of bearing walls around a stairwell. Also, in a tall structure with setbacks, the completeness of the frame for the tower, when carried through to the foundation, is not adversely affected by bearing walls in the base structure below the tower.

(b) *Building frame system* This consists of a vertical-load carrying space frame with horizontal load resistance provided by braced frames or non-loadbearing shear walls. This definition does not require that absolutely all the vertical load be carried by the vertical-load carrying frame (see Item (a)).

While there is no requirement to provide horizontal resistance in the vertical-load framing, it is strongly recommended that nominal moment resistance be incorporated in the vertical-load frame design. In steel structures, this might be in the form of nominal moment resisting beam flange or web connections to the columns. In reinforced concrete structures, the nominal moment resistance inherent in ‘cast-in-place’ concrete may be considered sufficient to qualify for this system, while most types of ‘precast’ concrete systems would not. In reinforced concrete structures, continuity and full development and anchorage of longitudinal steel, along with stirrups over the full length of beams, would be good design practice. The vertical-load frame provides a nominal secondary line of defence, even though all required horizontal forces are resisted by other earthquake resisting structural systems. Consideration should be given to the deformation compatibility between individual members. The presence of a frame can provide vertical stability to the structure and prevent collapse after damage to the shear walls or braced frames. The frame also acts to tie the structure together and redistribute the horizontal force to undamaged elements of the horizontal force resisting system.

Columns of the frame system may also function as the boundary elements of the shear walls. As such, these columns should be designed to resist the vertical forces resulting from overturning moment in the shear wall along with the load effects associated with the frame system.

(c) *Moment resisting frame system* As in the case of the building frame system, it is not necessary that all the vertical load be carried by the vertical-load carrying frame. The entire horizontal force stipulated should be capable of being resisted by moment resisting frames.

(d) *Dual system* The vertical-load carrying space frame should be substantially complete. Special details are required according to the R_f value.

The shear walls or braced frames and moment resisting frames of the horizontal force resisting system should conform to the following criteria:

(i) The frame and shear walls or braced frames should resist the total required horizontal force in accordance with relative rigidities, considering the interaction of the walls and frames as a single system. This analysis should be made in accordance with the principles of structural mechanics considering the relative rigidities of the elements and torsional effects in the system. Deformations imposed upon members of the frame by the interaction with the shear wall or braced frame should be considered in this analysis. In the previous Standard, it was required to design the shear walls or braced frames for 100% of the required horizontal force. This was a simple rule that permitted design without the necessity of evaluating the interaction effects with the moment resisting frame. With the availability of computer programs for the analysis of the complete structure model, the walls or braced frames can be designed for the more realistic interaction forces, and the 100% rule is no longer needed. While this may result in a somewhat reduced loading when compared to the previous requirement, there are extra detailing and design provisions to ensure the attainment of the performance objectives for the structure.

(ii) The special moment resisting frame (SMRF) acting independently should be designed to resist not less than 25% of the total required horizontal force including torsional effects. Columns of the frame system may also function as the boundary elements of shear walls. As such, these columns should be designed to resist the vertical forces resulting from overturning moment in the shear wall along with the load effects associated with the frame system.

C2.9 CONFIGURATION Irregularities in load paths and in structural configuration are major contributors to structural damage and failure due to strong earthquake ground motion.

SEAOC (Ref.10) and ATC-3-06 (Ref.11) have described undesirable configuration characteristics that are to be avoided wherever possible.

Extensive engineering experience and judgment are required to quantify irregularities and provide guidance for special analysis. A complete prescription of all the considerations of special analysis for irregularities is not yet available. However, recommendations have been provided both in plan and vertical dimensions for irregularity. These additional provisions are especially beneficial considering that structures are becoming increasingly complex, and their design requires additional care to achieve the required earthquake performance. Also, engineering knowledge and understanding of structure response and the effect of irregularities have progressed, particularly with the increasing use of computer analysis.

In reviewing the requirements for dealing with irregularities, it was recognized that the use of the elastic model in dynamic analysis can detect and correct the response effects of only certain types of irregularities. The vertical irregularities specified in Clause 2.9.3 are cases in point. Other irregularity effects are mitigated through design details and other analytical considerations such as limitations on storey drift ratio differences and torsional effects.

Regular structures Regular structures, designed to the prescribed earthquake design load levels, are expected to meet the requirements of the Standard. Two important concepts, or assumptions, apply for regular structures. Firstly, the linearly, varying horizontal force distribution given by Equations 6.3(1) and 6.3(2) is a reasonable and conservative representation of the actual dynamic response inertia force distribution due to earthquake ground motions. Given a common base shear value, the equivalent static force distribution provides a reasonable envelope of the forces and deformations due to the actual dynamic response. Secondly, when the design of the elements in the horizontal force resisting system is governed by the specified load combinations, then the cyclic inelastic deformation demands during a major level of earthquake ground motion will be reasonably uniform in all elements, without large concentrations in any particular part of the system. It is assumed that the inelastic deformation demands are proportional to, and reasonably approximated by, the deformations of a fully elastic (non-yielding) model of the structure when subject to major levels of ground motion. The acceptable level of inelastic deformation demand for a given material and system is, therefore, represented by the R_f value for the system, and the earthquake design level at a permissible stress basis is given by the fully elastic response divided by the R_f value as given by the base shear Equation 6.2.2. When a structure has irregularities, then these concepts, assumptions and approximations may not be reasonable or valid, and corrective design factors and procedures are necessary to meet the design objectives.

Irregular structures Vertical and plan irregularities can result in loads and deformations significantly different from those assumed by the equivalent static procedures. Those distributions of mass, stiffness, or strength which result in earthquake forces and deformations over the height of the structure, which are significantly different from linearly varying distributions, are designated as vertical irregularities. Those plan or diaphragm characteristics which result in significant amounts of torsional response, diaphragm deformations, or diaphragm stress concentrations, are designated as plan irregularities. The lack of a direct path for force transfer is referred to as an irregularity in force transfer, and an example would be where a horizontal force resisting element is offset or discontinuous. This condition can cause a concentration of inelastic demand, and can occur even when no irregularities in plan or elevation exist.

It is most important for designers to understand that irregularities create great uncertainties in the ability of the structure to meet the design objectives of the Standard. Therefore, the presence of recommendations to accommodate irregularities should not be construed as an endorsement of irregular structures, a 'catalogue' of imaginative structural forms, nor a manual for their design. The requirements and provisions are intended only to make designers aware of the existence and potential detrimental effects of irregularities, and to provide minimum requirements for their accommodation. If an irregularity condition cannot be avoided or eliminated by design changes, then the designer should not only apply the requirements and provisions given in the Standard, but also consider the ability of the structural system to provide the objective of life safety protection at the major level of earthquake ground motion. This is particularly necessary for cases of force transfer path irregularity. While incorporating provisions for irregularity runs the risk of their misuse as a 'design manual' for these undesirable structural features, they do serve to correct a far more serious danger. Designers should be aware of the existence and effects of irregularities. They should not employ conventional methods, as if the structure were regular, without accommodating dynamic response or force transfer deficiencies induced by these irregularities.

The various ratios and related criteria stated to define irregularity in Clauses 2.9.2 and 2.9.3 have been assigned by judgment based on the interpretation of past earthquake damage effects and design experience overseas. In many cases there is no way, short of an inelastic dynamic analysis of the complete structural and non-structural system, to assess or quantify the effects of these irregularities. It is also essential that sound professional judgment be used to formulate these analyses and to interpret their results. However, since such detailed types of analysis are generally not economically justifiable, it is the intent of these guidelines to assist the designer in detecting particular problem areas and correcting their detrimental effects. For this reason, the Commentary given for the assessment of irregularities is qualitative in nature and should be interpreted with sound engineering judgment. Further, it is of foremost importance in the design process to eliminate, or at least minimize, significant irregularities.

C2.10 DEFLECTION AND DRIFT LIMITS Two kinds of horizontal deflection need to be considered:

(a) The absolute horizontal displacement of any point in the structure relative to the base. No limit is set for this but the separation should be at least equal to the total maximum horizontal deflection of the two units, assuming deflection towards each other.

(b) The interstorey drift is the displacement of one floor relative to the floor below. This drift needs to be controlled to restrict damage to partitions, shaft and stair enclosures, and glazing. The drift limits also indirectly provide upper bounds for the *P*-delta effects. It is also a way of controlling inelastic strains in the structural members.

SECTION C3 DOMESTIC STRUCTURES

C3.1 GENERAL The previous *SAA Earthquake Code*, AS 2121—1979, excluded domestic structures but these are now covered by Section 3.

C3.2 REQUIREMENTS FOR EARTHQUAKE DESIGN CATEGORIES Domestic structures are separated into three earthquake Design Categories, H1, H2 and H3, in order of ascending earthquake loads, as given in Table 2.6.

The great majority of domestic structures in Australia will fall into earthquake Design Category H1 as they will have a value for (aS) of not more than 0.1. The product (aS) is the value of the acceleration coefficient (a) from Table 2.3 and the appropriate Figures 2.3(a) to (f) multiplied by the site factor (S) taken from Table 2.4(b).

Domestic structures of earthquake Design Category H1 do not require to be checked or structurally detailed for earthquake loads. Domestic structures on soft soil sites or in areas with higher acceleration coefficients are likely to fall into earthquake Design Category H2 or H3, and need to be designed and detailed as set out in the Standard.

Most domestic structures are not required to be designed for earthquake because the system already in place for wind resistance is usually adequate for earthquake resistance. However, if in doubt, such as for two storey structures or structures on soft soil, the following bracing guide can be used to check its adequacy.

Bracing requirements in each of the two axes at right angles to each other in any storey of steel or timber frame domestic structures can be expressed approximately in terms of floor area as follows:

(a) Light roof (having a mass less than 20 kg/m^2).

Single storey or top storey: 0.5(aS) kN/m^2

Lower storey of two storeys: 1.0(aS) kN/m^2

(b) Heavy roof (having a mass more than 20 kg/m^2 but less than 70 kg/m^2).

Single storey or top storey: 0.75(aS) kN/m^2

Lower storey or two storeys: 1.30(aS) kN/m^2

These requirements are generally expected to be less than that required for wind resistance except for special circumstances of earthquake Design Category H3.

C3.3 STRUCTURAL DETAILING REQUIREMENTS FOR DOMESTIC STRUCTURES Trusses, beams and the like are required to be horizontally restrained at their support. In most cases of timber framing, a ‘triple grip’ or similar will provide the required level of restraint. ‘Deemed to comply’ publication is being prepared by Standards Australia.

C3.4 STATIC ANALYSIS FOR NON-DUCTILE DOMESTIC STRUCTURES OF EARTHQUAKE DESIGN CATEGORY H3 This Design Category applies to domestic structures in only those few areas where the value of the acceleration coefficient (a) is high (e.g. Meckering) or where the value of the site factor (S) is high. In these instances, specific design and detailing for non-ductile domestic structures is required.

C3.5 NON-STRUCTURAL COMPONENTS The Clause applies to all design categories of domestic structures.

In many cases, the minimum earthquake forces are less than the wind forces for external elements.

Experience, both in Australia and overseas has shown that uninforced masonry, such as large or high walls, gable ends and chimneys, which are not properly restrained against horizontal earthquake load are most at risk.

Again, ‘deemed to comply’ publication is being prepared by Standards Australia for these components.

SECTION C4 STRUCTURAL DETAILING REQUIREMENTS FOR GENERAL STRUCTURES

C4.1 GENERAL These requirements have been based on those given in the ATC provisions (Ref. 11).

They are general structural detailing requirements for the structural system and are additional to the specific detailing requirements given in the various material Standards and the requirements for non-structural components given in Section 5. These requirements do not apply to domestic structures, which are covered by Section 3.

C4.2 STRUCTURAL DETAILING REQUIREMENTS FOR STRUCTURES OF EARTHQUAKE DESIGN CATEGORY A Ductility is defined in Clause 1.3.10 and the implication is that a structure which possesses this property is ductile and one that does not is non-ductile. In general, structures of steel, reinforced concrete, reinforced masonry, steel/concrete composite construction and timber will be ductile. Those of unreinforced (plain) masonry and unreinforced (plain) concrete will be non-ductile. Structures of prestressed concrete and precast concrete probably will be ductile but should be evaluated. In these cases, the joint connection details will have a major influence on the behaviour of the structure and thus whether or not it is ductile.

C4.3 STRUCTURAL DETAILING REQUIREMENTS FOR STRUCTURES OF EARTHQUAKE DESIGN CATEGORY B

C4.3.2 Load paths, ties and continuity The connection should be designed to carry the specified forces without reliance being placed on friction, except in the case of loadbearing masonry where the transfer of horizontal loads by friction can be calculated in accordance with AS 3700. This is because under the action of an earthquake, vertical movements may occur which may inhibit the development of horizontal friction. The connection will also help in holding the two pieces together, for example, tending to prevent a bearing jumping off its seating or roofs jumping off walls. Connection may have to allow for movements due to other design considerations.

For loadbearing masonry structures, slabs should be supported on a series of walls at right angles to each other to avoid the possibility of the slab being dislodged from its supporting wall.

C4.3.3 Wall anchorage The value of the force in the ATC document (Ref. 11) is given as 1000*a* lb/lin.ft. This would roughly correspond to 15*a* kN per metre. However, to take account of the lower forces for a 500-year return period used for these requirements compared to a 1000-year return period in the ATC document, this figure has been divided by 1.5, giving 10*a* kN per metre.

Where this requirement is called up for earthquake Design Category A, i.e. for non-ductile structures, the design force is reduced by 50% to account for the lower forces likely to be generated.

C4.4 STRUCTURAL DETAILING REQUIREMENTS FOR STRUCTURES OF EARTHQUAKE DESIGN CATEGORIES C, D AND E

C4.4.2 Ties and continuity The requirements of the Clause are similar to those in the ATC document (Ref. 11). The variation being accounted for is by the difference in '*a*'. The minimum percentage load has been retained.

C4.4.6 Footing ties The wording of the Clause has been changed from that in AS 2121. Restraint is now required for piles, caissons and spread footing only when these are founded in or on low-strength soils.

'Other means' of tying footings include floors or a combination of floors and retaining walls.

SECTION C5 REQUIREMENTS FOR NON-STRUCTURAL COMPONENTS

C5.1 GENERAL REQUIREMENTS Design provisions for the mitigation of damage to the non-structural components of a building in AS 2121 gave only limited guidance in an area of almost unlimited variety. Experience, not only overseas but also locally, suggests that non-structural components can have a significant effect on life, safety and economic loss, and therefore this aspect is of considerable importance in the earthquake resistant design of a facility. The rules of the Section are extensively based on the NEHRP provision (Ref. 9). However, a number of simplifications have been implemented where the original has been considered too onerous.

C5.1.1 General The Standard requires that, except for domestic structures and general structures of earthquake Design Category A in areas where the product of the acceleration coefficient and site factor (*aS*) is less than 0.1, all architectural, mechanical and electrical components should be designed to resist the nominated earthquake forces. It is recognized that reliance on the *aS* value in this context is not ideal, but it was considered desirable to use this simple assessment in order to simplify the development of industry standards. It is not the intention, however, that individual design be required for each component within a structure. Rather it is expected that design details will be developed to meet these requirements by industry groups.

Although the Standard refers to equipment mounted on structures and portions thereof, the Standard should also be applied to equipment mounted on the ground such as high-voltage circuit breakers, switchgear, transformers and large horizontally mounted tanks.

C5.1.2 Forces on components The earthquake forces called up in the Section correspond to the limit state of strength. The Standard assumes that the earthquake forces induced can be applied at the centre of gravity (mass) of the component and requires that it be applied in the direction most critical for the element.

The Clause also suggests that the vertical earthquake forces on mechanical and electrical components may be taken as 50% of the horizontal earthquake forces. Whilst not physically correct, the Clause does not require the simultaneous application of both horizontal and vertical earthquake forces. It is considered that in view of the other uncertainties, the need to consider the simultaneous application of these loads is unwarranted.

Whilst there is no explicit requirement for architectural components to be designed for vertical earthquake forces, the designer should be aware that the consideration of such effects may be warranted in some cases. For example, differential vertical movement between cantilevered floor slabs in adjacent storeys has occurred in past earthquakes and should be considered for higher occupancy structures.

C5.1.3 Interrelationship of components The design of components that are in contact with or in close proximity to other structural or non-structural systems should recognize the potential for one component to adversely affect another. For example, if a ceiling supports a wall for loads perpendicular to the plane of the wall, the intersection should be detailed to accommodate the in-plane differential movements caused by drift of the frame. Where an important element of a system, such as the backup generator for a hospital, is located adjacent to a non-loadbearing partition then the wall should be designed to ensure that it does not jeopardize the generator.

C5.1.4 Connections and attachments Connections for non-structural components are to be designed to resist the earthquake forces calculated in accordance with the Section. In designing such connections, it should be recognized that friction cannot be relied upon to keep a component in place. It is, for example, commonly observed that equipment and fixtures tend to 'walk' due to the rocking induced by the ground motion.

Whilst structures are generally designed to accept significant post-yield deformation, for many structure service installations it will be more appropriate to secure the equipment without any post-yield deformation, particularly where the operation of the equipment might be impaired by the misalignments caused by yielding. Nevertheless, in the design of such connections, the desirability of achieving adequate ductility in the connection is emphasized.

C5.2 REQUIREMENTS FOR ARCHITECTURAL COMPONENTS

C5.2.1 Forces Although the Clause permits the use of any rational analysis to determine the design forces on components, it is expected that in general the empirical approach set out in the Standard will be used.

The procedures adopted in Clauses C5.2 and C5.3 remove some of the complexity of the NEHRP approach (Ref. 9) by avoiding the use of performance levels. It was considered that the degree of complexity introduced by the use of performance levels was not warranted for the level of earthquake risk experienced in Australia.

The horizontal earthquake force (F_p) applied to a component is assumed to be dependent on the following parameters:

(a) Acceleration coefficient (a) for the locality.

(b) Attachment amplification factor (a_c).

(c) Height amplification factor at level x (a_x).

(d) Earthquake coefficient (C_{c1}) for the architectural component.

NOTE: Tables, such as Table 5.1.5(a), do not cover all conceivable components. The Clause requires that, for unlisted elements, designers select appropriate values by comparison with the values given in the Table.

(e) Importance factor (I).

(f) Weight of a component of a structure or equipment (G_c).

Although amplification effects due to the height of a component do not appear in the NEHRP document (Ref. 9), they have been included in the Clause for consistency with the equation for mechanical and electrical components (see Clause 5.3).

C5.2.2 Exterior wall panel attachment Inadequate deformation capacity in a connection may result in brittle-type failure. To avoid the risk of catastrophic failure that may be induced under such circumstances, the exterior wall panel connections are required to have adequate ductility and rotational capacity to accommodate the design storey drift.

C5.2.3 Component deformation Walls, partitions and glazing at each storey of a structure should be capable of accommodating the storey drift induced during an earthquake without causing a life safety hazard. The larger storey drift experienced by frame structures may cause substantial damage to these items unless such deformations are allowed for in the design. The following components should be considered:

(a) Direct deformation in the component itself.

(b) Direct deformation in the joints or connections of the component systems.

(c) Deformation in the component produced by structural frame or structural wall movements.

(d) Deformation in the joints or connections of the component produced by the structural frame or structural wall movements.

C5.2.4 Out-of-plane bending Out-of-plane forces are induced in a wall when a structure is subjected to an earthquake. These forces do not figure in the overall frame design and therefore need to be separately considered in the design of the wall panel. This is particularly important in the design of brittle or low-flexural-strength materials.

C5.3 REQUIREMENTS FOR MECHANICAL AND ELECTRICAL COMPONENTS

C5.3.1 Forces Although the Clause permits the use of any rational analysis to determine the design forces on components, it is expected that in general the empirical approach set out in the Standard will be used.

The horizontal earthquake force (F_p) applied to a component is assumed to be dependent on the following parameters:

(a) Acceleration coefficient (a) for the locality.

(b) Attachment amplification factor (a_c).

(c) Height amplification factor at level x (a_x).

(d) Earthquake coefficient (C_{c2}) for the mechanical or electrical component.

NOTE: Tables, such as Table 5.1.5(b), do not cover all conceivable components. The Clause requires that, for unlisted elements, designers select appropriate values by comparison with the values given in the Table.

(e) Importance factor (I).

(f) Weight of a component of a structure or equipment (G_c).

C5.3.2 Ducts and piping distribution systems Specific design for earthquake effects for ducts and piping distribution systems of earthquake Design Categories A and B is not required. The risks associated with such elements do not warrant special consideration.

Whilst earthquake restraints for ducts and piping distribution systems of earthquake Design Categories C, D and E is required, the Clause lists a series of elements which may be excluded from such consideration.

C5.4 AMPLIFICATION FACTOR

C5.4.1 Attachment amplification factor Where equipment is mounted on flexible supports, such as springs, a potential for enhanced response occurs when the mounting frequency of the component coincides with the natural frequency of the structure. Some references have suggested that the amplification factor can be determined assuming sinusoidal motion of the floor. Such an assumption results in very large amplifications which for the following reasons are considered unwarranted:

(a) The floor motions are unlikely to be truly sinusoidal.

(b) The duration of strong shaking is likely to be limited.

(c) The structure period (T) is unlikely to remain constant under strong shaking.

The Committee thus decided to limit the attachment amplification factor (a_c) to a maximum of 2.

Where the ratio of periods (T_c/T) is less than 0.6 or greater than 2.0, it is considered that significant amplification is unlikely and so a factor equal to unity may be used.

For a single degree of freedom (SDOF) system, the period of vibration (T_c) of the component and its attachment is given by the following equation:

$$T_c = 2\pi\sqrt{\left(\frac{M}{K}\right)} \qquad \text{. . . C5.4.1(1)}$$

where

M = mass of the single degree of freedom system

K = stiffness of the single degree of freedom system

As

$$M = G_c/g \qquad \text{. . . C5.4.1(2)}$$

where

G_c = weight of component

g = gravitational constant (9.81 m/s^2)

Then

$$T_c = 2\pi\sqrt{\left(\frac{G_c}{gK}\right)} \qquad \text{. . . C5.4.1(3)}$$

Note that the period of vibration (T_c) of the component should be determined for the direction of motion of interest.

For non-linear springs, e.g. rubber isolators, the slope of the load deflection curve at the level of load applied should be used as the stiffness.

C5.4.2 Height amplification factor The attachment forces developed as a result of the motion of a floor are dependent on the absolute acceleration of the floor to which they are connected.

The absolute acceleration experienced at any floor of a structure is the algebraic sum of the ground acceleration and the floor acceleration relative to the base of the structure at any instant. It can be shown that the absolute acceleration increases with height to a maximum at the top of the structure.

As an approximation, it has been assumed that the maximum acceleration at the top of the structure is twice that at the ground level and that, for any other floor, linear interpolation may be used.

Clearly, the height amplification factor (a_x) enhances the desirability of locating heavy mechanical equipment on the lower levels of the structure.

SECTION C6 STATIC ANALYSIS

C6.1 GENERAL The Section specifies equivalent quasi-static design earthquake forces which designers may use in lieu of dynamic earthquake forces (which may be determined by dynamic analysis).

In the static analysis, the time varying inertial forces are replaced by equivalent (quasi) static forces applied at each storey level. The relative values of these storey forces are based on the simplifying assumptions that storey drifts and masses are reasonably uniform over the height of the structure and the modes of vibration as a result of the earthquake are primarily translational.

The term significant yield is not the point where first yield occurs in any member but is defined as that level causing complete plasticity of at least the most critical region of the structure, e.g. formation of the first plastic hinge in the structure.

An earthquake resisting system with redundant characteristics is assumed wherein overstrength above the level of significant yield is obtained by plasticity at other points in the structure prior to the formation of a complete mechanism. This continued inelastic action provides the reserve strength necessary for the structure to resist the extreme motions of the actual horizontal earthquake forces that may be generated by the specified ground motion.

C6.2 HORIZONTAL FORCES

C6.2.1 General Earthquake forces are completely random and should be expected from any horizontal direction. The Clause, however, allows designers to consider only two directions, namely the directions of the main axes of the structure.

It also acknowledges that earthquake effects in the two main directions are unlikely to reach their maximum simultaneously. The combinations of effects of gravity loads and horizontal earthquake forces in the two orthogonal directions (x- and y-direction) can be expressed as follows:

(a) Gravity 100% of x-direction, 30% of y-direction.

(b) Gravity 30% of x-direction, 100% of y-direction.

These effects are slight on beams, slabs and other horizontal elements but they may be significant in columns and other vertical members that participate in resisting horizontal earthquake forces in both main directions of the structure.

It should be noted that components of an earthquake resisting system utilized in only one of the two orthogonal direction need not be designed for the combined effects.

C6.2.2 Earthquake base shear Equation 6.2.2 is the heart of the static analysis. V is the equivalent static horizontal force for which the structure may be designed instead of the actual inertial force generated in the structure by the movement of the ground. The value of the base shear in the Standard is higher than that given in AS 2121 and the Uniform Building Code (UBC) of California (Ref. 12) for regions of the same earthquake level because of the following:

(a) The value given is for the strength limit state and not for working stress design.

(b) The acceleration coefficient (a) given is for a recurrence interval of 500 years.

(c) The values reflect the general trend of increased base shear from UBC-1985 to UBC-1991 (Ref. 12), particularly for non-ductile systems and for low-level structures.

C6.2.3 Earthquake design coefficient The earthquake design coefficient (C) as given in Equation 6.2.3 is based on the nominated response spectra of Figure 7.2 used in dynamic analysis (see Section 7).

C6.2.4 Structure period The fundamental structure period (T) of a structure is given by Equation 6.2.4(1). It corresponds to the motion of the structure along its most flexible axis. Where the same framing system and structure dimensions are used in both directions, this period may also be used for the period in the orthogonal direction.

Where a different framing system is used in the orthogonal direction, however, the period of vibration of the structure for the orthogonal direction should be determined using Equation 6.2.4(2). Note that Equation 6.2.4(2) should be associated with the stiffer of the two framing directions.

Other methods of determining the fundamental periods of the structure may be used providing they are based on established principles of structural mechanics.

C6.2.5 Gravity load The gravity load (G_g) should include the total weight of the structure and that part of the live load that might reasonably be expected to be on the structure at the time of an earthquake. This is consistent with AS 1170.1 but is higher than many overseas codes. The final static force has been adjusted to correlate with overseas codes allowing for the higher gravity loads.

C6.2.6 Structural response factor The structural response factor (R_f) in the denominator of Equation 6.2.2 is an empirical response reduction factor intended to account for both damping and the ductility inherent in the structural system at displacements great enough to surpass initial yield and approach the ultimate load displacements of the structural system. Thus, for a lightly-damped building structure of brittle material that would be unable to tolerate any appreciable deformation beyond the elastic range, the response factor (R_f) would be close to 1, i.e. no reduction from the linear elastic response would be allowed. At the other extreme, a heavily-damped building structure with a very ductile structural system would be able to withstand deformations considerably in excess of initial yield and would, therefore, justify the assignment of a larger response factor (R_f). Table 6.2.6(a) stipulates response factors for different types of structural systems using several different structural materials. The response factor (R_f) ranges in value from a minimum of 1.5 for an unreinforced masonry shear wall system to a maximum of 8.0 for a special moment resisting frame system.

In Table 6.2.6(a), the response factor (R_f) and the K_d value for deflection amplification have been established in view of the fact that structures generally have additional capacity above that where the design loads cause significant yield. The response factor essentially represents the ratio of the forces that would develop under the specified ground motion if the structure behaved in an entirely linear elastic response to the prescribed design forces. The structure is to be designed so that the level of significant yield exceeds the prescribed design force. The response factor (R_f) is always greater than 1.0; thus, all structures are designed for R_f forces smaller than the design ground motion would produce in a completely linear-elastic-responding structure. This reduction is possible because of the capacity that the whole structure possesses due to its capability to deform inelastically. In establishing the R_f value, consideration also has been given to the performance of the different materials and systems in past earthquakes.

Note that the value of R_f increases with higher ductility and damping whereas the design earthquake force decreases.

The value of R_f should be chosen and used with careful judgment. For example, lower values would be used for structures possessing a low degree of redundancy wherein all the plastic hinges required for the formation of a mechanism may be formed essentially simultaneously and at a force level close to the specified design strength. This situation can result in considerably more detrimental P-delta effects.

C6.3 VERTICAL DISTRIBUTION OF HORIZONTAL EARTHQUAKE FORCES The distribution of horizontal earthquake forces over the height of a structure is generally quite complex because these forces are the result of superposition of a number of natural modes of vibration. The relative contributions of these vibration modes to the total horizontal earthquake forces depends on a number of factors including the shape of the earthquake response spectrum, the natural periods of vibration of the structure, and the shapes of vibration modes that, in turn, depend on the mass and stiffness over the height. The basis of this method is discussed below.

The horizontal earthquake force (F_x) at level x, (see Figure 6.3) due to response in the first (fundamental) natural mode of vibration is calculated as follows:

$$F_x = V_1 \frac{G_{gx}\phi_x}{\sum_{i=1}^{n} G_{gi}\phi_i} \qquad \text{. . . C6.3}$$

where

V_1 = contribution of this mode to the base shear

G_{gx} = portion of gravity load (G_g) located or applied at level x

ϕ_x = amplitude of the first mode at level x

G_{gi} = portion of gravity (G_g) located or applied at level i

ϕ_i = amplitude of the first mode at level i

It is well known that the influence of modes of vibration higher than the fundamental mode is small in the earthquake response of short-period structures and that, in regular structures, the fundamental vibration mode departs little from a straight line. This, along with the matters discussed above, provides the basis for Equation 6.3(2).

It has been demonstrated that, although the earthquake response of long-period structures is primarily due to the fundamental natural mode of vibration, the influence of higher modes of vibration can be significant and, in regular structures, the fundamental vibration mode lies approximately between a straight line and a parabola with the vertex at the base.

However in view of the level of earthquake in Australia, the Committee did not consider it warranted to consider other modes of vibration because of the so-called 'whiplash' effect and the additional horizontal earthquake force added at the top to simulate the effects of different modes of vibration.

C6.4 HORIZONTAL SHEAR DISTRIBUTION The horizontal earthquake shear force at any level is the sum of all the horizontal forces acting at and above that level. Level x is the level immediately below level (x + 1) (see Figure 6.3). Reasonable and consistent assumptions regarding the stiffness of concrete and masonry elements may be used for analysis in distributing the shear force to such elements connected by a horizontal diaphragm. Similarly, the stiffness of moment of braced frames will establish the distribution of the horizontal earthquake shear force to the vertical resisting elements at that level.

C6.5 TORSIONAL EFFECTS An earthquake induces horizontal inertia forces associated with the mass distribution in a structure. At any particular floor level, the resultant may be considered to act through the centre of mass at that floor level. These inertia forces are resisted by the vertical components of the earthquake resisting system (resisting elements), and the resultant acts through the shear centre of the storey under consideration. In many structures, for any particular storey, these opposing forces are not coincident. This gives rise to eccentricities and consequent torsional moments (Refs. 13, 14 and 15).

These eccentricities (termed static eccentricities) may be quantified, as they result from irregularities such as those caused by functional architectural requirements, an irregular plan configuration or an unbalanced distribution of vertical resisting elements. However, in any structure, allowance needs to be made for uncertainty about the magnitude of eccentricities during an earthquake, as accidental eccentricities which cannot be quantitatively predetermined are present. Accidental eccentricities are due to the actual (as opposed to the assumed) mass and stiffness distributions, the presence and participation in the structural response of stairs, partitions and masonry infill walls and such other non-quantifiable characteristics of any particular structure.

The dynamic earthquake response of any particular element in an idealized structure with no eccentricities consists of a horizontal earthquake shear force, the magnitude of which corresponds to the particular element's contribution to the overall horizontal stiffness of the storey (see Clause 6.4).

However, in any real structure, torsional effects arise due to both accidental and (if present) static eccentricities. These effects change the shear forces in any particular element. The magnitude of these changes, which may be positive or negative, depends on both the torsional moment acting at any instant during the dynamic response, and the plan location of the element under consideration. For a design earthquake applied in any single direction, elements situated in plan close to either edge of the structure are most vulnerable to adverse torsional effects.

From the above definition of static eccentricity, the static torsional moment applied at any storey level can be defined as the product of the static eccentricity and the total horizontal earthquake shear force (storey shear), as determined from Clause 6.4. However, in the dynamic response of real structures, an amplification of this torsional effect occurs (Refs. 11 to 13). The magnitude of this amplification depends on the relationship between the natural vibration frequencies of the structure, the magnitude of the static eccentricity and the degree of inelastic response, if any. The amplification effect is larger for structures responding elastically, compared with those excited beyond the elastic limit.

Further, since the resisting element shear forces depend on the combination of the horizontal and torsional components of the vibration, the dynamic phasing of these components at any instant results in the peak responses of edge elements on opposite sides of the structure occurring at different times. Consequently, the Clause accounts for the changes in the element shear forces due to torsional effects by specifying two equivalent static torsional moments for design purposes. These torsional moments are used to determine the more unfavourable change in shear force, for each resisting element.

These design torsional moments are calculated using primary and secondary design eccentricities (Equations 6.5.3(1) and 6.5.3(2) respectively, and Figure 6.5.3.1). Appropriate dynamic eccentricity factors are included in the first term of each equation, to account for the dynamic amplification or reduction of static eccentricity. For the primary case, the dynamic eccentricity factor (A_1) is greater than or equal to 1.4, and is dependent on the magnitude of the normalized static eccentricity (e_s/b), as shown in Figure 6.5.3.2. For the secondary case, a dynamic eccentricity factor (A_2) of 0.5 is applied. Further, the second term of each equation represents the allowance for accidental eccentricity, namely, 5% of the maximum dimension (b) of the structure at the level under consideration, transverse to the loading direction. This is added to the appropriate factored static eccentricity for the primary case, and subtracted for the secondary case, to ensure that the more unfavourable case for resisting elements on each side of the structure is included.

C6.6 STABILITY EFFECTS In the design of footings using static analysis, the calculated overturning moment (M) may be reduced by 25%. The reasons for reducing the calculated overturning moments are as follows:

(a) The distribution of the horizontal earthquake shear forces over height calculated from the horizontal earthquake forces (F_x) specified in Clause 6.3 is intended to provide an envelope since the shear forces in all storeys do not attain their maximum simultaneously. Thus, the overturning moments calculated from the envelope of the horizontal earthquake shear forces will be overestimated.

(b) It is intended that the design shear envelope, which is based on the simple distribution of horizontal earthquake forces specified in Clause 6.3, be conservative. If the shear forces in a specific storey is close to the exact value, the shear forces in almost all other storeys are almost necessarily overestimated. Hence, the overturning moments which are statically consistent with the horizontal shear forces will be overestimated.

(c) Under the action of overturning moments, one edge of the footing may lift from the ground for short durations of time. Such behaviour leads to substantial reduction in the earthquake forces and, consequently, in the overturning moments.

C6.7 DRIFT DETERMINATION AND *P*-DELTA EFFECTS The *P*-delta effects at a storey are due to the eccentricity of the gravity load (G_g) above that storey. If the storey drift is δ, then the bending moment at the storey would be increased by an amount equal to δ times the gravity load above it. The ratio of the *P*-delta moment to the horizontal shear force moment ($V_x h_{sx}$) is designated as the stability coefficient (m).

The Standard allows the static stability of the structure to be checked, based upon its initial elastic stiffness, hence δ equals Δ/K_d. This might not be conservative. Designs that produce a value of m greater than 0.25 should be examined carefully. Structural analysis computer programs, which take *P*-delta effects into account, are available commercially.

C6.8 VERTICAL COMPONENTS OF GROUND MOTION Effects of vertical components of ground motion are significant for cantilever type structures and prestressed concrete components, and components which cannot withstand reversal of stress.

SECTION C7 DYNAMIC ANALYSIS

C7.1 GENERAL The Section provides minimum requirements, limitations, and guidelines for performing dynamic analysis. It is not intended to be a complete coverage and should not be used as a 'how-to' manual for dynamic analysis. The designer should consult appropriate references on dynamic analysis for detailed procedures—for example, see Refs. 16 to 20. The analysis procedures outlined in the Section incorporate dynamic aspects of earthquake response in the design process. A dynamic analysis, *per se*, will not necessarily provide response estimates consistent with actual earthquake performance nor will it give all the answers or solutions to an earthquake design problem. The accuracy of the results depends upon many things, among which are—

(a) the simplifying assumptions made to produce the computational procedures;

(b) appropriate assignment of material property values, structure dimensions, damping coefficients, and other characterizations of the structure;

(c) how close the model reflects the real structure and its foundation conditions;

(d) the representation of the maximum expected earthquake input for the site; and

(e) the correct interpretation of the analysis.

Dynamic analysis, however, can greatly aid the earthquake design process by clarifying certain important aspects of the structure's dynamic response characteristics that may not be apparent from the static analysis procedure of Section 6. For example—

(i) the effects of the structure's dynamic characteristics on the distribution of horizontal force along its height, which could differ from the distribution considered in the quasi-static approach;

(ii) the existence of normal modes with significant components of torsional motion that can lead to increased dynamic loads in the structure's horizontal force resisting system; and

(iii) the effects of the structure's higher modes of response that could contribute substantially to the individual storey shears and deformations.

The dynamic analysis procedures presented were developed to address the fact that the distribution of forces on some structures is often considerably different from those given in Section 6. As an example, such differences occur in structures with severe set-backs, structures of unusual configuration, and structures with significant variation in drift from one storey to another. Implicit in these provisions on dynamic analysis is the assumption that the analysis will normally be performed using a computer program. Thus the provisions are formulated in a manner suitable for such implementation.

Even though the earthquake loads specified in the Section are for ultimate strength conditions, an elastic dynamic analysis is deemed to be acceptable on the grounds that a structure designed to resist elastically the level of earthquake force which has been reduced by the appropriate structural response factor will have the inelastic capacity to withstand the maximum expected earthquake forces. The response to the maximum expected earthquake will depend on inelastic behaviour for energy absorption and the resulting displacements will be well beyond those corresponding to the design loads. The details and concepts used by designers are the critical elements in earthquake resistant designs and should be so recognized.

The Section provides the designer with a tool for dealing with structures that may violate the assumptions inherent in the use of the static approach of Section 6. Regular and symmetric structures generally exhibit much more favourable and predictable earthquake response characteristics than irregular structures. Therefore, the use of regular structures is encouraged and irregular structures should be avoided wherever possible.

The implementation of dynamic horizontal force procedures involves—

(A) identification of appropriate ground motion representations for use as input to the analysis;

(B) development of an elastic mathematical model of the structure that represents the important geometric, stiffness, inertial and damping characteristics;

(C) computation of the model's dynamic response to the earthquake input motions, using response spectrum or time history methods and established structural analysis computer programs; and

(D) careful interpretation and application of the analysis results.

C7.2 EARTHQUAKE ACTIONS The Clause describes the type of representation of earthquake ground motion, principally by a response spectrum, that can be used in a dynamic analysis. The results of a dynamic analysis are normally sensitive to the estimated intensity and frequency content of the earthquake ground motion. The spectrum can be considered to have two characteristics—shape and magnitude. For the design of structures using dynamic analysis, the shape of the response spectra is more important than the magnitude because the structural response is scaled as specified in Clause 7.4.2.4. The duration of the ground motion, which also is an important factor in structure response, is not directly represented by the response spectrum procedures. However, duration representative of major earthquake ground motion is recognized implicitly by the special provisions for systems and elements. These provisions are intended to provide stable levels of resistance under multiple reverse cycles of inelastic deformations.

C7.2(a) The spectrum shapes given in Figure 7.2 are a smoothed average of normalized 5% damped response spectra. They were obtained from actual ground motion records that were grouped by subsurface soil conditions at the location of the recording instrument (Ref. 16) and are applicable for earthquakes characteristic of those that occur in California. Where a site's specific ground motion may significantly differ from those used to determine the standard spectra, site specific ground motions should be used. In order to perform a response spectrum analysis using the spectra shown in Figure 7.2, the normalized spectra should be multiplied by the appropriate acceleration coefficient (a) and the ratio I/R_f in order to account for structural importance and ductility.

C7.2 (b) and (c) The use of site-specific response spectra and time histories is mentioned to allow designers to make use of these tools when it is desirable to go beyond the minimum requirements of the Standard or where a specific spectrum is required for a soft-soil site.

The site-specific ground motion time histories used should not have spectra that have significant valleys at periods important to the structure's response. For this reason, the provisions of Clause 7.2(c) require that the spectra from the time histories, either individually or in combination, should approximate site-specific spectra developed in accordance with Clause 7.2(b). Also, the time variation of recorded earthquake motions typically exhibit an initial rise, a central segment of strong-shaking, and a decaying tail. The duration of each of these segments is dependent on the nature of the faulting and surrounding geological conditions. Designers should recognize this fact and should therefore develop site-specific time histories whose time variation is reasonably consistent with the time variation of actual ground motions that have been recorded under comparable geological conditions.

When time history analyses are performed, good practice requires that independent time histories be used in each of the orthogonal directions of input and that several time histories be used in each direction, to assure that the responses are characteristic of those which may occur and to provide a sufficient number of response estimates to ensure that the possible response of the structure is fully represented. The multiple responses provide a basis for estimating expected values of the dynamic forces in the structure's critical elements.

Again, when complying with the Standard, the content of the input is important over the full frequency range of the structural response and not so much the magnitude because the final forces should be scaled up, and may be scaled down to reflect the design levels of the static analysis procedure of Section 6.

When site-specific spectra for time histories are used, it is recommended that these equal or exceed the level of ground motion having a 10% probability of being exceeded in 50 years at the site. It is further recommended that the base shear force determined by these procedures be reduced to a design base shear by dividing by a factor not greater than the appropriate *R* factor for the structure but subject to the minimum design value required by Clause 7.4.2.4. The structural response distribution due to the particular site-specific ground motion characteristics is better represented by the dynamic analysis procedure. The scaling procedure of Clause 7.4.2.4 is intended for use with the spectra of Figure 7.2 which are general averages and not necessarily representative of site-specific conditions.

C7.3 MATHEMATICAL MODEL A mathematical model is an idealized, simplified representation of a complex building structure to facilitate computation of the structure's response for design purposes. Modelling is an art in which the most important characteristics of the structure are synthesized into a mathematical representation for computation of the dynamic response. Regular structures can usually be adequately represented with a one- or two-dimensional model where accidental torsion is included with additional manual calculation. Very complex, irregularly-framed structures, or those with large eccentricities between centres of mass and resistance, will require a three-dimensional analysis. Only an outline of some of the elements of modelling will be discussed (see Refs. 17, 18, 19 and 21).

There is a series of key assumptions that are common to most analysis models—

(a) the structure is assumed to be linearly elastic;

(b) small deformation theory applies;

(c) structural mass is commonly applied at a few selected joints or nodes; and

(d) energy dissipation (damping) is assumed to be viscous or velocity proportional.

General stiffness modelling considerations Great care should be exercized in developing a model for the structure because a structure is necessarily represented by an idealization containing fewer, conceptually simpler elements than actually exist in the structure. The designer should ascertain that the model adequately represents the structure such that the desired behaviour and response will be obtained, and that no extraneous or artificial responses occur from use of the simplified model. These artificial responses can result from the arithmetical errors introduced by truncation, often caused by large differences in stiffness from—

(a) the introduction of rigid elements (stiff ends);

(b) improper stiffness assignments, particularly where different types of elements intersect at joints; and

(c) not recognizing the fundamental behaviour pattern of the structure in preparing the model.

Three-dimensional models are required when the use of a two-dimensional model would obscure some significant information. It should be noted that by themselves two-dimensional models are unable to reflect orthogonal effects and torsion in the structure.

Finite joint size In modelling beams and/or columns of frame structures, beam elements are usually considered as concentrated about their principal axes and then geometrically represented as lines. Intersections of such lines at model node points are the physical connection points in the real structure. In mathematical models, these physical connection points (called nodes or joints) are normally treated as single point locations which have no size dimension. For relatively slender beam and column elements, the assumption of dimensionless or negligible size of joints is satisfactory. However, where beams and columns have significant depth at intersections, the overall structure is stiffer than as represented by the assumption of zero joint size, and this should be considered in the model formulation.

Diaphragm flexibility Diaphragms are generally classified into the following three groups:

(a) Rigid.

(b) Semirigid.

(c) Flexible.

These terms are relative and depend on the stiffness of the horizontal resisting elements. For example, a rigid diaphragm in a frame structure may be semirigid in a shear wall structure. The response of a structure with a large plan eccentricity could be very sensitive to the diaphragm flexibility and appropriate evaluations of stiffness should be made to model this element correctly.

Section properties The selection of section properties for the structural model is complex and requires considerable judgment to properly understand and represent the structure's expected performance. Reinforced concrete and masonry are particularly difficult because they are non-homogeneous materials with properties that can vary significantly at different levels of response. For concrete and masonry, a first approximation would be to use gross uncracked section properties and the modulus of elasticity. Assuming cracked sections and transformed steel area throughout, or using the moments of inertia for deflection calculations may be an excessive refinement, because the model response results may have to be scaled and the period is limited. All significant resisting elements should be included in the stiffness model, such as steel members embedded in concrete or cast-in-place concrete beams and columns that may not be part of the horizontal force resisting system. Stiff dummy members should not be used without checking the impact this may have on the results because of potential numerical difficulties.

Boundary conditions Boundary conditions are the constraints in a model designed to match physical conditions at the edges of a structure or component. Boundaries for models required for earthquake response are usually those which move with the ground at the location of the specified base motions. Boundary elements or nodes may be utilized in response analyses to model several conditions or effects. Base elements or nodes may be fully or partially constrained to model the support stiffness of a structure, including its foundations. Most commonly, nodes at 'ground' are fixed in rotation or translation (or both) in order to model boundary restraint. It is assumed in these provisions that most structures will be analyzed without explicit consideration of soil stiffness. Where the fixed base assumption produces unconservative results, soil flexibility should be considered in the analysis.

Mass representation Distribution mass commonly will be either lumped at the nodes in some fashion, or placed as a single mass lumped at each floor level. Where lumped mass is used, as is the case in most computer programs, care should be taken to ensure that any mass eccentricity of the real structure is properly retained and that the application of the resulting forces on the structural model does not cause erroneous beam shortening. The mass of mechanical equipment, curtain walls and other non-structural elements should be considered. Storey masses should be lumped only when the diaphragm is rigid, otherwise they should be distributed.

There are computer programs that permit an actual mass distribution within individual finite elements. These programs use the consistent mass formulation. Care should be taken to understand the method used in the computer program selected for the analysis.

Damping Energy dissipation in the form of a damping ratio is normally idealized in linear elastic dynamic analyses as viscous or velocity-proportional for convenience of solution. The response spectrum curves (see Figure 7.2) have 5% damping, and other spectra that may be used should use this 5% damping value. Damping in time history analysis should also be 5% unless a higher value can be substantiated by data. It is recommended that 7% damping not be exceeded.

C7.4 ANALYSIS PROCEDURES While two methods of analysis are represented, it is expected that a majority of designers will use the response spectrum analysis procedure. The time history analysis procedure is included for use where more detail is required or where the characteristics of the model could yield incorrect or misleading results when response spectrum analysis is applied.

C7.4.2 Response spectrum analysis The response spectrum analysis is the preferred method for most structures. It is strongly recommended that designers first perform an equivalent static analysis to help verify the model and to provide a basis for comparison of the response spectrum results. It is considered good practice to plot the significant mode shapes as a means of both checking the model and gaining insight into how the structure performs. Clauses C7.4.2.2 to C7.4.2.6 are intended to provide the minimum requirements for its application.

C7.4.2.2 *Number of modes* For an analytical approach to determining the required number of modes, see Refs. 17, 18 or 19. Demonstration that 90% of the participating mass of the structure is included in the calculation of response for each principal horizontal direction is an acceptable as well as a straightforward procedure.

C7.4.2.3 *Combining modes* Horizontal forces, storey drifts, base shear forces, and base reactions, among other measures of structural response, are calculated by combining the respective values for each mode. The same method of modal combination should be used to combine these terms as is used for member forces and displacements. It should be noted that in modal analysis when the modal member forces are combined, they are no longer in equilibrium, nor are the combined modal displacements consistent with the member forces. This result occurs because the signs of individual modal contributions are lost, and the individual modal maxima do not all occur at the same time. Equilibrium and displacement can be checked only on an individual mode basis (see Refs. 22 to 26).

The methods, such as ‘square root of the sum of the squares’ (SRSS) and ‘complete quadratic combination’ (CQC), used to combine the effects of several modes are approximate and account for the fact that all modal maxima do not occur at the same time. There is no assurance that either method will produce conservative results; however, under the proper conditions, each method will produce results of acceptable accuracy.

When response spectrum analyses were first performed, the modal terms were combined by taking the square root of the summation of the squares of the individual modal terms. This procedure may be acceptable when the natural frequency of each mode is at least 20% greater that the frequency of the next lower mode. However, this condition may not be satisfied for some structures, particularly when a three-dimensional model is required. There are modified forms of the SRSS that allow for the combination of closely-spaced modes (Ref. 25). The adequacy of the SRSS method also depends on damping ratios. As damping increases, model coupling effects become more important for a given modal frequency ratio. The CQC method accounts for this, whereas the SRSS method does not. The CQC method is a general, theoretically well-based approach to modal combinations. It is described in more detail in Refs. 22, 23 and 26. The CQC method reduces to the SRSS method when the modes have widely separated frequencies. Thus, use of the CQC method will always be acceptable.

Modal combinations present several important problems in the interpretation of results. Firstly, all computed terms are positive. Secondly, the value associated with each term may correspond to a different point in time. Thus member and joint equilibrium cannot be checked; moments, shear forces, and deformations at points between the nodes in the model cannot be directly calculated. Designers needs to consider these conditions when using the terms, and should assign signs to the individual terms to ensure that the results are conservative. An examination of individual modes may be useful in those assessments.

Some other design codes require that designers know if a member is in single or double curvature, and the predominant mode of response could be used to determine this condition.

C7.4.2.4 *Scaling of results* The earthquake force levels of Section 6 for static analysis reflect the influence of structural period (T), ductility and damping (energy absorption) for various structural systems. Because it is difficult to reflect these influences on a consistent basis for dynamic analysis, a procedure for scaling the dynamic response has been included. This scaling is used to make dynamic analysis results consistent with the results of the static design approach which has gained acceptance since regular structures designed by this approach have, in general, performed acceptably in earthquakes. Refs. 16, 17, 22, 26 and 27 provide a discussion of the relationship between traditional design load levels, structural configuration and details, and structure performance. It has been determined that this scaling procedure is appropriate for design, but it is an interim procedure subject to future replacement. Great care should be exercised in interpreting scaled results to assure that they are physically reasonable and appropriate, since many quantities may not actually scale linearly with base shear force.

C7.4.2.4(a) The Clause requires the base shear force resulting from a dynamic analysis be increased to that required by Section 6. All corresponding forces should also comply with this increase. For the purpose of evaluating this base shear force increase factor, the period of the first mode may be used in calculating the earthquake design coefficient (C) by Clause 6.2.3. The value of C should be consistent with the calculated period but should not be less than 80% of the value obtained by use of the period calculated from Equation 6.2.3.

Dynamic analysis procedures are considered to provide a somewhat improved or more accurate force distribution. Therefore a base shear force of 90% of that determined by static analysis procedure of Section 6 is permitted for a regular structure when a dynamic analysis is performed. Also, when dynamic analysis is used, the fundamental period is provided and qualifies for the advantage given in Clause 6.2.4, where the resulting earthquake design coefficient (C) can be as low as 80% of the value determined using T calculated from Equations 6.2.4(1) and 6.2.4(2). Thus, it is possible to qualify for both directions. However, in order to prevent excessively low base shear force resulting from combining both reductions, the lower limit of 80% of the value obtained using T calculated from Equations 6.2.4(1) and 6.2.4(2) was set.

It is recommended that building structures classified as being irregular be designed for 100% of the base shear force value determined by static analysis of Section 6.

C7.4.2.4(b) This is a permissive section in that designers are permitted to decrease the dynamic base shear force to that required by Section 6. However, there may be cases where the designer or owner may wish to use force levels greater than the minimum in order to provide more damage control or continuity of operation.

C7.4.2.5 *Directional effects* It is understood that the direction of earthquake ground motion is random. It bears no relation to the axes of the structure and, in fact, the response is a composite response of concurrent motion in relation to the several axes about which the structure can vibrate. Normally, this will include fundamental motion about the two principal orthogonal axes in plan, plus torsional motions.

Nevertheless, for structures, it is implied that the independent design about each of the principal axes will generally provide adequate resistance for forces applied in any direction. One cautionary note is worth emphasizing: exterior and re-entrant corners of structures are especially vulnerable to the effects of concurrent motions about both principal axes. Designers should pay particular attention to the effect of combined forces on members common to the systems along both axes.

For structures other than buildings, the requirement that the horizontal earthquake forces be considered to come from any direction should be respected more rigorously. For structures circular in plan, like tanks, towers, and stacks, the design should be equally resistant in all directions.

Statistical analysis of ground motion records have shown that it is generally reasonable for design purposes to use a vertical component of ground motion whose peak amplitude is two-thirds of the peak amplitude of the horizontal motion. However, for unusual site locations or conditions, a site-specific evaluation should be performed to specify vertical ground motions for earthquake design. The resulting forces may not be less that those required by the static analysis procedure (see Clause 6.8).

C7.4.2.6 *Torsion* The occurrence of significant torsional motions can lead to increased loads in the structure's horizontal force resisting system and can be strongly coupled with the structure's horizontal motions if large eccentricities exist between the centres of storey resistance (often called centre of stiffness) and the centres of floor mass, and if the natural frequencies of the structure's normal modes are closely spaced (Refs. 21 and 24). For such conditions, a three-dimensional model should be used to conduct the dynamic analysis and the CQC approach is recommended for combining the maximum modal responses.

Accidental torsional effects prescribed in Clause 6.5 are also required for the dynamic analysis procedures. Torsional motions can occur in a structure even if its centres of mass and resistance are coincident and the natural frequencies of its predominant modes of vibration are well spaced. Such torsional motions, termed accidental torsion, can arise from several factors not typically considered in the dynamic analysis of structures, such as—

(a) spatial variations of horizontal input motions;

(b) rotational components of the ground motions;

(c) the effects of non-structural elements (e.g. partitions and stairs) on the structure's stiffness and inertial characteristics;

(d) the actual distribution of dead and live loads; and

(e) uncertainties in defining the structure material properties for the dynamic analysis.

To account for such effects, torsional moments due to accidental torsion can be computed using the static analysis procedures prescribed in Clause 6.5 and then distributed to the various members of the structure's horizontal force resisting system to obtain the corresponding member forces. This distribution can be accomplished by either an equivalent static analysis method or a dynamic analysis method. For the static analysis method, the accidental torsional moments at each level may be calculated as the product of the equivalent static force calculated in accordance with Clause 6.5 times the 5% eccentricity specified in Clause 6.5. The resulting moments are applied as pure couple loadings, all of the same sense, at their corresponding levels. The effects of these couple loads are then added, as an increase, to the results of the dynamic analysis. For the dynamic analysis method and non-flexible diaphragms, it is required to displace the mass in the dynamic model to alternate sides of the calculated centre of mass and use a three-dimensional analysis to calculate the effects directly. For highly-eccentric structures, care should be taken to ensure the critical location of this mass displacement to obtain the highest load effect.

C7.4.3 Time history analysis The time history procedure described in the Section uses a time-dependent representation of the earthquake input motions. The procedure is applicable to either a linear elastic or a non-linear model of the structure. It computes the time-dependent dynamic response of the model through numerical integration of the model's equations of motion. Time history analysis procedures are typically more complex to implement than are response spectrum analysis procedures. However, such procedures do have certain advantages over response spectrum methods, particularly in representing non-linear behaviour and incorporating time-dependent effects.

It is the intent that the resulting structure be in general compliance with the Standard. When the designer, for whatever reason, uses the time history analysis procedure, complete back-up information should be provided. This is to facilitate checking to determine that the proposed structure is in general compliance with the force levels permitted in the Standard.

Time history analyses are performed to gain insight into element and subsystem performance. They are not normally used as the basis for design since maximums occur at different times in different elements and conversion of these results to design levels can be complex. Thus time history analysis generally should be used as a supplemental method to verify the design, rather than as a replacement.

It is strongly recommended that both a static analysis and a response spectrum analysis be performed before any time history analysis is conducted. This provides a means of validating the analysis and gives the designer something with which to compare the time history analysis results. If this is not done, results may be questionable and difficult to interpret. A further reason for not emphasizing time history analysis is because its computational effort can be prohibitive and often may not be warranted in view of uncertainties in modelling, identifying earthquake input motions, and so on.

APPENDIX CE

STRUCTURAL ALTERATIONS

CE1 INTRODUCTION The Section provides measures intended to reduce the risk which may result from the effects of earthquakes on existing structures. Many existing structures are typically of masonry loadbearing wall construction, and the ability of such structures to withstand significant shaking caused by earthquakes is limited, as has been demonstrated in earthquakes in Australia and overseas.

It is recommended that structures should achieve, wherever possible, a load carrying capacity equivalent to those specified in the Standard. It should also be recognized that it may not be possible to show that an existing structure can withstand the horizontal forces acting as a whole and it is not intended that these measures should result in disproportionate expenditure to achieve strengthening.

Experience in Australia and overseas has shown that unreinforced masonry which is not horizontally supported is one of the most likely areas of major damage in an earthquake.

CE2 DESIGN Because of quality of manufacture and age factors including weathering, deterioration with time and fatigue, the strength of existing materials is extremely variable and should be considered in any upgrading. The Items listed in Paragraph E2 of Appendix E are areas which have been shown from previous experience to require special attention by designers.

It is recommended that, where renovations or alterations are to be carried out which affect the strength of the structure, the assessment and strengthening procedures are carried out by qualified and experienced persons such as architects, builders and engineers, and this information should be submitted to the regulatory authorities when details of upgrading are submitted.

Overseas experience indicates that the problems of alterations to existing structures and the need to upgrade structures to the current earthquake loads have been tackled only in recent times in earthquake areas, such as USA and New Zealand, which are at a much higher risk than Australia. It was not until the early 1970s that surveys of existing structures were carried out in California. These have now resulted in statutory requirements for upgrading structures and a considerable amount of public attention and research work has been applied to this problem.

Similarly in New Zealand, despite the history of earthquake activity, it was only in 1974 that local councils were empowered to deal with existing structures likely to be dangerous in moderate earthquakes.

The following basic courses of action have been adopted to date:

(a) Do nothing in the hope that time and natural attrition will remove many of the hazards before the next major earthquake.

(b) Examine and assess structures considered to be potentially hazardous in an earthquake and require the owners to upgrade these structures within a given time scale. In some cases, this work is required even if the structure is not being upgraded or renovated.

The Standard has accepted that strengthening work should be carried out in conjunction with alterations. For this case, assessments should be carried out at two levels as follows:

(i) Quantitatively, based on sound engineering judgment and upgrading using accepted details.

(ii) Analysis as for new structures where this is feasible.

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