

A REVIEW OF THE CODE PROVISIONS FOR SEPARATION OF ELEMENTS & BUILDINGS

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ABSTRACT

This paper provides background information on the provisions of the proposed New Zealand Standard Code of Practice for General Structural Design and Design Loadings relating to the separation of non-structural elements from the structure, and the separation of buildings from adjacent buildings. The degree of protection afforded by these provisions is indicated for various levels of earthquake attack, and designers are warned of the implications of possible non-structural damage under severe seismic loading.

1. INTRODUCTION

The author was asked by the Seismic Loads Sub-Committee of the Standards Association of New Zealand to report on the degree of protection provided by the proposed separation provisions in the draft New Zealand Standard Code of Practice for General Structural Design and Design Loadings, DZ4203 (1), and to recommend, if possible, simple modifications to provide a more comparable degree of protection against non-structural damage for structures designed to different strength levels. As a result of this report, new separation limits were set by the Committee for the final form of the code. When the code is published it will replace NZS 1900 Chapter 8 in part and MP12. The revised provisions are discussed here and designers are advised as to what they represent in terms of degree of protection, from damage under earthquake, of non-structural elements within a building and of buildings adjacent to other buildings.

The following are definitions of terms used in the text :

"code separation limit" - the minimum distance by which a non-structural element must be separated from the structure or a building separated from the property boundary, as specified by the proposed code and presented in Section 2 below.
 "degree of protection" - the code separation limit as a proportion of the maximum expected interstorey displacement or overall building displacement for a particular size of earthquake.

2. SUMMARY OF CODE SEPARATION PROVISIONS

The proposed code specifies a maximum interstorey deflection at code lateral loading of 0.0003 of the storey height where non-structural elements are not separated, and not more than 0.005 of the storey height in any case. Thus, separation of elements is now required in all buildings except those which are very rigid.

For purposes of computing the required non-structural element and building separations, a separation modification factor, v , is introduced. The value of this factor for different types of structure is listed in Table 1 below. The items referred to in this table refer to the types of structure listed in Table A.2 in the Appendix. Numerical values of the code design strength parameters: Basic Seismic Coefficient, C , and Importance Factor, I , and the derivation of the Base Shear Coefficient, C_d , are also listed in the Appendix.

Where elements must be separated from the structure as specified in the code, separation provisions shall allow for at least v times the deformation computed at code loading, but not less than 12mm ($\frac{1}{2}$ in.). The code further specifies that each building separated from its neighbour shall have a minimum clear space from the property boundary either of 1.5 v times its computed deflection or 0.004 times its height whichever is largest, and in any case not less than 12mm ($\frac{1}{2}$ in.).

In setting the separation limits the Seismic Loads Sub-Committee of SANZ has acknowledged the practical difficulties and the expense of large separations, and the required minimum separation distances are significantly smaller than the deformations that could result from the response of the building to a major earthquake.

In order to allow for such effects as the cracking of reinforced concrete frame members and joint deformations in structural steel frames, the code will provide guidance as to appropriate effective moments of inertia to be taken when computing the structural deflections under code loading. This will probably comprise a simple reduction of the gross moment of inertia of all reinforced concrete and structural steel sections by 25%. Guidance will also be given as to the contribution of floor slabs to the stiffness of the frame. Corresponding guidance has already been given for the evaluation of the elastic stiffness of reinforced concrete coupled shear walls after cracking in the members by Paulay⁽²⁾. However, the reduction multiplier for the stiffness of the coupling

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TABLE 1

TYPE OF STRUCTURE	v
Item 1 - Table A.2 (Ductile moment resisting frames in reinforced concrete or structural steel)	<u>2.0 CI</u> C_d
Item 1 - Table A.2 (Ductile moment resisting frames comprising prestressed concrete primary lateral load resisting elements)	<u>2.8 CI</u> C_d
Items 2 to 5 - Table A.2 (Ductile moment resisting frames with limited number of beam hinges, ductile coupled shear walls, and ductile cantilever walls)	<u>2.0 CI</u> C_d
Item 6 - Table A.2 (Walls dissipating energy in shear) Items 7 and 8 - Table A.2 (Diagonally braced frames) Item 9 - Table A.2 (Other buildings)	2.0

beams will be different from that quoted in that reference when these beams are diagonally reinforced. (Diagonal reinforcement of the coupling beams is necessary for a ductile coupled shear wall to qualify for a structural type factor of 0.8.)

3. DEGREE OF PROTECTION PROVIDED BY CODE PROVISIONS

3.1 Introduction

A convenient measure of the degree of protection provided by the code provisions may be obtained by comparing the non-structural element and building separation limits with the expected maximum deflections of the structure under a given size of earthquake, based on the response of an equivalent elastic system. This is illustrated in Figs. 1 to 3 which comprise a plot of the ratio of lateral force on a structure to the weight of the structure against the displacements of the structure relative to those of the equivalent elastic system. Since the mass is the same in both numerator and denominator of the ordinate term, this axis corresponds to the lateral acceleration of the structure as a fraction of the acceleration due to gravity. The abscissa is dimensionless since all displacements are relative to those of the equivalent elastic system. Fig. 1 illustrates the method for one particular building. In each of Figs. 2 and 3 the curves for a number of different buildings with the same overall structural stiffness have been superimposed. The periods of vibration in these two cases were chosen to represent, firstly, a stiff building and, secondly, a flexible building. In both cases curves have been plotted for all structural types that could conceivably have this period of vibration. Since diagonally braced frames up to three storeys in height with tension yielding braces only or squat shear walls are unlikely to have periods as long as 1.2 seconds, they have been omitted from Fig. 3.

On the basis of the assumptions listed below, the curves of Figs. 1 to 3 show the relative overall building deflections for different types of structure and can clearly be used to indicate the relationship between the building separation limit and the expected maximum building displacement in an El Centro 1940 N-S earthquake. Also, since all displacements shown in the figures are relative the curves may be used, subject to an assumption of uniform interstorey drift, to show the relationship between the non-structural element separation limit and the expected maximum interstorey displacement for a particular structure in the given earthquake. The rationale for this is amplified in Section 3.2.3.

In all cases the code separation limit marked on the figures is that derived solely from the separation modification factor as presented in Section 2. The possible influence of the lower limit constant is discussed subsequently. The sections of the load-displacement curves for each structural type on the initial elastic line, from code loading to maximum capacity, are separated only for visual clarity.

3.2 Assumptions

3.2.1 The level of seismic attack assumed

is that due to an El Centro 1940 size earthquake. The acceleration response for an elastic structure of a given period and damping ratio has been derived from the smoothed response spectra of Skinner ⁽³⁾ for a range of earthquakes scaled to have the same spectrum intensity as the El Centro 1940 N-S record. The effect of an earthquake with a lower or higher intensity may be derived from the figures by interpolation or extrapolation of the elastic response line.

3.2.2 Where the structural type and material composition comprise a stable hysteretic system with adequate energy dissipation under dynamic cyclic load, such as a Ramberg-Osgood system typifying structural steel frames or a degrading stiffness system typical of reinforced concrete structures dissipating energy in flexure, the equal displacement concept of seismic deflections has been assumed. That is, the maximum displacement of the inelastic systems responding to a given earthquake is assumed to be the same as the maximum displacement of an elastic system with the same initial stiffness and damping ratio responding to the same earthquake.

In the case of prestressed concrete frames, where the hysteresis loops of the members under cyclic loading do not exhibit such large energy dissipation under moderate rotations as those for well designed reinforced concrete frames, the displacement of the inelastic system is assumed to be 1.1 times that of the equivalent elastic system. It is to be noted that prestressed concrete frames are also considered to have a lower equivalent viscous damping ratio than reinforced concrete frames, and the cumulative effect is for a prestressed concrete frame designed according to the code strengths in the Appendix to be assumed to have a maximum seismic response displacement of 1.4 times that of a reinforced concrete frame with the same initial stiffness. The justification for this assumption may be found in the work of the author and Park ⁽⁴⁾ and Spencer ⁽⁵⁾.

The seismic performance of cross braced frames in which the braces can yield in tension only may be characterised as a "slip-type elastoplastic system". On the basis of research on the seismic response by such systems by Veletsos ⁽⁶⁾ and Bazan and Rosenbluth ⁽⁷⁾, the maximum displacement of the inelastic system is assumed to be 1.25 times that of the equivalent elastic system. A structural system which fails in shear exhibits hysteretic behaviour similar to a slip-type elastoplastic system, except that it may have an earlier reduction in load capacity, as shown by the research of Paulay ⁽²⁾ and consequently the same displacement ratio of 1.25 has been assumed for shear wall structures dissipating energy in shear.

3.2.3 It is assumed that the ratio between the interstorey deflection of a structure under code loading and the maximum interstorey deflection of the structure under a given earthquake is equivalent to the same ratio of deflections for the whole

building under these two loading conditions. This is effectively assuming uniform interstorey drifts throughout the height of the building. In fact this will be the case for a well designed structure, such as is the basis of the code provisions, with the exception of some reduction at the top, due to the effect of higher modes of vibration than the first mode, and at the base. An example of analysis results showing the interstorey drift variation for a multi-storey frame are those of Goel and Berg (8). Of course, the above assumption will not be true in the undesirable situation of plastic hinging being localised in some storeys.

On the basis of this assumed uniform interstorey drift, the load versus relative displacement curves of Figs. 1, 2 and 3 may be used to show the relationship between the minimum non-structural element separations of a particular structure and the maximum interstorey displacements under the given earthquake. Of course, for some different structural types to have the same period of vibration the number of storeys and the interstorey stiffness will be different. If the curves had been drawn in terms of absolute rather than relative displacements the initial interstorey stiffnesses for the different structural types would not have coincided. Thus, the curves do not show a comparison of absolute interstorey displacements between different structures but only relative interstorey displacements for a particular structure.

3.2.4 The assumed values of equivalent viscous damping for the various structural types are : 2% for prestressed concrete framed structures and steel cross braced frames capable of yielding in tension only, 5% for reinforced concrete and structural steel framed structures, and 10% for all concrete shear wall structures. These figures are based on the reported results of a large number of dynamic tests of buildings. Some variation will be expected according to the intensity of earthquake attack. A prestressed concrete framed structure is considered to have a lower equivalent viscous damping than a reinforced concrete framed structure, primarily because of the delayed cracking of the members in the former. Foundation compliance in soft soils is contributory to the higher damping ratio for a shear wall structure, which is generally stiffer than a frame.

3.2.5 The code design strengths for all structures have been derived from the new code provisions outlined in the Appendix. The different structural types refer to those specified in Table A.2. Those types of structure for which an S factor must be determined by a special study could not be included. The structural material factors are as given in Table A.3. The basic seismic coefficients have been determined from Fig. A.1 for buildings in Zone A, with periods up to 0.45 seconds for the curves of Fig. 1, and periods equal to or greater than 1.2 seconds for Fig. 2. In all cases the importance factor for Class III structures, comprising buildings in the private sector, and the risk factor for low risk buildings have been taken as listed in Tables A.1 and A.4,

that is $I = 1.0$ and $R = 1.0$.

3.2.6 The ratio between the strength capacity of a structure and the code design load is assumed to be : 1.5 for frames, coupled cantilever walls, and cantilever walls; and 1.25 for walls dissipating energy in shear and diagonally braced frames with braces capable of yielding in tension only. The assumed value of this ratio does not affect the degree of building or non-structural element protection derived from this approach, since for the equal displacement criterion the maximum displacement is independent of strength. In reality some reduction in response displacement would usually be expected with increasing capacity. The load-displacement curves of Figs. 1 to 3 for yielding systems such as frames and coupled shear walls are idealised since generally there will be a range of progressive yield rather than a sharp yield point, but this will not have a large effect on the maximum displacements.

3.3 Discussion of Provisions

3.3.1 Inspection of Table 1 shows that the separation modification factor, v , by which computed deflections at code loading are to be multiplied, has three separate expressions depending on the structural system considered. The three cases are : firstly, all ductile moment-resisting frames and walls except for prestressed concrete frames; secondly, prestressed concrete framed structures; and thirdly, diagonally braced frames with members capable of yield in tension only, walls dissipating energy in shear and other structures. These categories may be considered in turn as follows:

The value of the base shear coefficient, $C_d = C_{IS} \gamma R$, used in the design of ductile moment-resisting frames and walls is subject to considerable modification, for example by a Structural Material Factor, γ , of 0.8 for structural steel, a Structural Type Factor, S , of 0.8 for an adequate number of beam hinges, and a further possible factor of 0.8 where a dynamic analysis is used. Structures designed to lower C_d values will in general be subject to greater deformations in the inelastic range. Such strength reductions can be justified in cases where superior structural performance may be expected, but no such case exists for reducing separation limits. Therefore, if the separation limits were to be made a constant multiple of the deflection at code loading as in the previous code, the degree of protection would decrease as the design strength decreased. Instead, the new provisions require a uniform degree of protection for all frames and ductile walls in the same seismic zone and with the same importance factor. This is achieved by setting the separation modification factor proportional to the term CI/C_d , where C_d equals the product of $C_{IS}\gamma R$ and may include a further reduction factor for a dynamic analysis. Since C and I are present in both numerator and denominator, the factor v is inversely proportional to S , γ , R and any modification for dynamic analysis. Therefore, as the code design strength decreases or increases

with these latter four parameters, and the deflection at code loading varies accordingly for a given initial structural stiffness, the value of the factor v will increase or decrease in inverse proportion and the product of v times the deflection at code loading will remain constant. The factor, v , has the basic value of 2.0 where $S = \gamma = R = 1$, for example a low risk reinforced concrete structure with two or more ductile cantilever shear walls, and where there is no reduction for dynamic analysis. The resulting effect on structures in this category is for constant separation distances for all structures with the same C and I and equivalent initial stiffnesses, equal to the control case above as shown in Figs. 2 and 3. The degree of protection is then uniform between all frames and between all ductile walls with the same C and I , with the degree of protection for the latter structures slightly greater due to their higher damping. The reason for making the value of v independent of C and I , and the influence of the period of vibration are discussed in subsequent sections.

The separation provisions for prestressed concrete framed structures represent a 40% increase in separation distances for a prestressed concrete framed structure relative to reinforced concrete and structural steel frames with the same initial stiffness, that is the same initial period of vibration. Thus, on the basis of the assumptions listed in the second paragraph of section 3.2.2 there will be a degree of protection of non-structural elements within prestressed concrete framed structures, and of prestressed concrete frames adjacent to other buildings, equivalent to that for other frames.

The degree of protection afforded to these structures listed in Table 1 with a constant separation modification factor with the basic value of 2.0 is shown in Fig. 2. Since the value of v is now independent of the code design strength parameters, an increase of code strength with those parameters, and the corresponding increase in deflection at code loading for a given initial stiffness, results in proportional increases in separation distances. However, the degree of protection remains similar to that for ductile frames and walls. This is partially because the expected seismic response displacements of softening hysteretic systems, such as shear failure mechanisms and cross-braced frames capable of yielding in tension only, are greater than those of their equivalent elastic systems as discussed in paragraph 3 of Section 3.2.2. A further factor is that although the degree of protection of walls dissipating energy in shear may appear greater than that for ductile frames and walls on Fig. 2, this may not exist in reality because of the difficulty of accurately predicting the deflection under code design loading of a structure with shear cracking. Thus, in this case a reduction in the value of v below the basic value of 2.0 is not warranted.

3.3.2 For the first two categories of structures discussed in the preceding section, the value of the separation

modification factor is independent of either the basic seismic coefficient or importance factor for a building since the parameters C and I appear both in the numerator and inherently in the denominator. This is also obviously the case in the third category where v is constant. Thus, as the code design strength of a structure increases along with the increase in basic seismic coefficient from Zone C to Zone A or as the importance of a structure increases from Class III to Class I, and correspondingly the deflection at code loading increases for a given initial stiffness, the degree of protection in a particular size of earthquake for buildings and non-structural elements will increase in the same proportion. Thus, provision is automatically made for the likelihood of a greater level of earthquake attack in Zone A relative to Zone C, and for the extra protection of buildings required to be functional immediately after a seismic disaster. The degree of element and building protection shown in Figs. 1 to 3 would be increased by factors of 1.3 or 1.6 if the curves had been plotted for Class II or Class I buildings respectively.

3.3.3 Comparison of Figs. 2 and 3 shows that the degree of non-structural element and building protection remains similar with varying periods of vibration. This simply reflects the fact that the idealised trilinear curve of code design strength approximately follows the acceleration response spectrum of an El Centro - type earthquake, with a reduction factor of 4 representing the benefits of inelastic energy dissipation. The exception to this rule is for long period structures where the basic seismic coefficient remains constant for structures with periods of 1.2 seconds or longer, whereas the actual acceleration response spectrum continues to decrease in this range. Thus with reference to Fig. 3, for structures with periods longer than 1.2 seconds the non-structural element and building separation limit will remain constant as the seismic coefficient is constant, whereas the relative actual seismic response will reduce. This represents an extra degree of protection for long period structures, although in all practical structures with periods less than 3.0 seconds the non-structural element separation limit is still less than the expected maximum interstorey displacements in an El Centro size earthquake.

3.3.4 As discussed in section 3.2.2 the maximum seismic displacements sustained by reinforced concrete and structural steel ductile frames and shear walls have been assumed to be represented by the equal displacement criterion, that is $R = 1/\mu$, where R = Reduction Factor and μ = Displacement Ductility Factor. It is to be noted that if the equal energy criterion, that is $R = 1/\sqrt{2\mu-1}$, had been taken the maximum seismic displacements would have been greater in all cases than those shown in Figs. 1 to 3, particularly for structures with low strengths, and correspondingly the apparent degree of protection provided by the code provisions would be lower. However, the former criterion is considered to be more valid

for structural types with adequate energy dissipation capacity.

3.3.5 The lower limit of element separations of 12mm specified by the code and quoted in Section 2 represents an interstorey drift of 0.00333 times the storey height for an interstorey distance of 3.6m. For all frames the product of the interstorey deflection under code loading and the separation modification factor is likely to be greater than this, at least in seismic zones A & B, and therefore will be the critical criterion. The lower limit is more likely to modify the non-structural element separation limits of Figs. 1 to 3 in the cases of shear wall structures or cross braced frames, where it would improve the degree of protection. Likewise, the lower limits of building separations from the property boundary of 0.004 times the building height or 12mm are more likely to be the critical criterion for stiff buildings.

3.3.6 Inspection of Figs. 2 and 3 shows that all frame and cantilever wall structures designed for Zone A as Class III structures may be expected to suffer no non-structural damage when subjected to an earthquake with response spectrum accelerations of up to 1/3 of the El Centro 1940 N-S values. For example, the accelerogram recorded at Haywards, 24 kilometres north of Wellington, on 1st November, 1968 gave peak acceleration response spectrum values of the order of 0.3 g for structures with 5% equivalent viscous damping and periods of less than 0.3 seconds. This earthquake was of Richter magnitude 5.5 and occurred at a focal depth of 33 kilometres and an epicentral distance of 51 kilometres from the recording station. It may be seen from Fig. 2 that, if the peak response spectrum for a 5% damped system had been 0.3 g, the maximum seismic displacements from an equal displacement criterion would have been equal to the non-structural element separation limits for ductile frames and walls. The relatively low level of non-structural damage in Wellington as a result of that earthquake, when many buildings would not have had separated elements, may be explained by the fact that most older buildings tend to have much higher damping than the values taken for this study, which are more appropriate to structures in which elements are separated.

Clearly, an El Centro 1940 N-S magnitude earthquake, on which the figures have been based, could not be regarded as the most severe earthquake possible in this country. Some indication of what this could be is given by Jennings, Housner and Tsai (9) with their artificially generated earthquake, Al. This earthquake is intended to model the shaking in the vicinity of the fault in a Richter magnitude 8 or greater earthquake, and has a spectrum intensity half again as strong as the El Centro 1940 shock. The response spectrum of this record for a 5% damped structure has an acceleration of 1.1g up to a period of 0.4 seconds. This may be compared with the corresponding value of 0.9g for the El Centro record as shown in Fig. 2. Therefore, the maximum seismic displacements could conceivably be 20% greater than those shown in Fig. 2 for short period structures, whereas for long

period structures the difference could be as much as 50%. In compensation higher equivalent viscous damping ratios may be applicable to structures responding to the more intense earthquake.

4. DAMAGE CONTROL

Designers should be aware of the degree of protection afforded by the minimum separation limits specified in the code, as demonstrated in Figs. 2 and 3 and amplified in Section 3.3. Design according to these provisions should prevent damage under moderate earthquakes, such as the Haywards 1968 record, but would not preclude the possibility of damage to non-structural elements or "hammering" of adjacent buildings under a severe earthquake such as El Centro 1940 N-S. Accordingly, consideration should be given to minimising the effect of such damage. The experience of the large capital loss due to non-structural damage in the 16 storey reinforced concrete framed Central Bank during the Managua earthquake, together with examples of non-structural damage during moderate earthquakes in New Zealand, highlight the problem. An example of desirable design detailing to achieve non-structural element damage control is to locate the exterior panel walls of a framed building in a plane either outside or inside of the line of the columns, and this gives the panels more opportunity for movement at the corners than would be possible at junctions with columns.

5. CONCLUSIONS

The degree of protection provided by the minimum separation provisions of the proposed New Zealand Standard Code of Practice for General Structural Design and Design Loadings as they relate to non-structural elements and whole buildings has been illustrated. The basis of the method has been the assumption that the maximum displacement of a stable hysteretic system, with adequate energy dissipation, responding to a given earthquake is the same as the maximum displacement of an elastic system with the same initial stiffness and damping ratio responding to the same earthquake. Larger displacements have been assumed for structures with less energy dissipation capacity in ratios based on research evidence.

The proposed loadings code allows strength reductions in cases where superior structural performance may be expected, but no such case exists for reducing separation limits. If the minimum separation distances were to be made a constant multiple of the deflection at code design load as in the previous code, these distances may become so small in a structure with low code design strength as to lead to considerable non-structural damage in a moderate earthquake. Instead, the new code separation provisions contain an expression for a modification factor which results in a fairly uniform degree of protection against non-structural damage for all structures regardless of code design strength. This modification factor has the basic value of 2.0, as in the old code, where the product of the

code design strength parameters, S , γ and R , is unity. The degree of protection against non-structural damage in a particular size of earthquake will increase for buildings in zones considered more likely to sustain a severe earthquake, and for buildings classified as being more important.

Because the practical difficulties and expense of providing large separations could not be ignored, the Seismic Loads Sub-Committee of SANZ has set the minimum separation limits with the aim of preventing damage under a moderate earthquake but accepting that damage would be expected under a severe earthquake. Details which will minimise such damage, such as placing wall panels outside the line of the columns of a framed building, should therefore be considered by designers to limit the possibility of danger to persons in and around a building or of large capital loss due to widespread non-structural damage.

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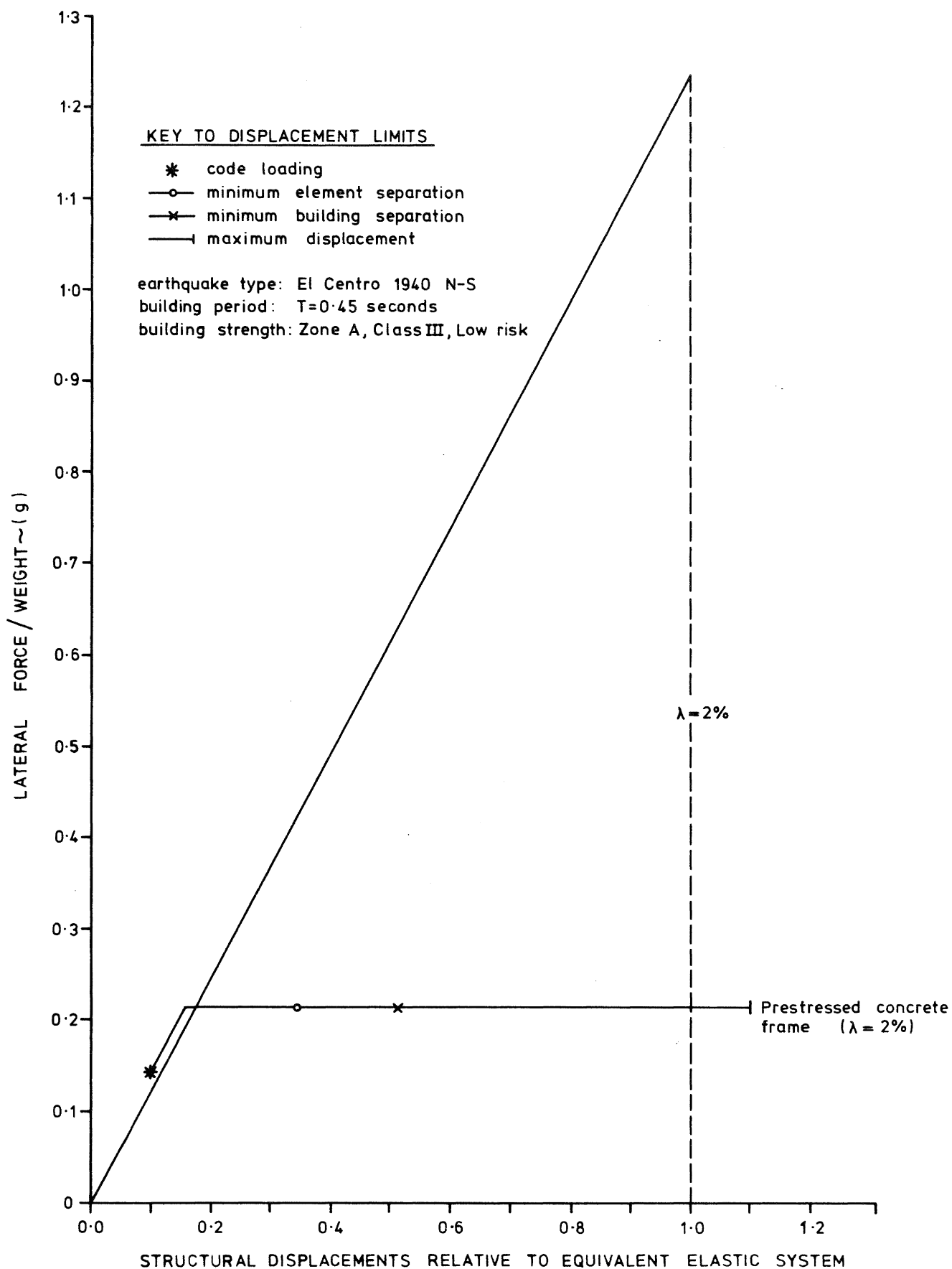


FIGURE 1: RELATIVE STRUCTURAL DISPLACEMENTS FOR A PSC FRAME

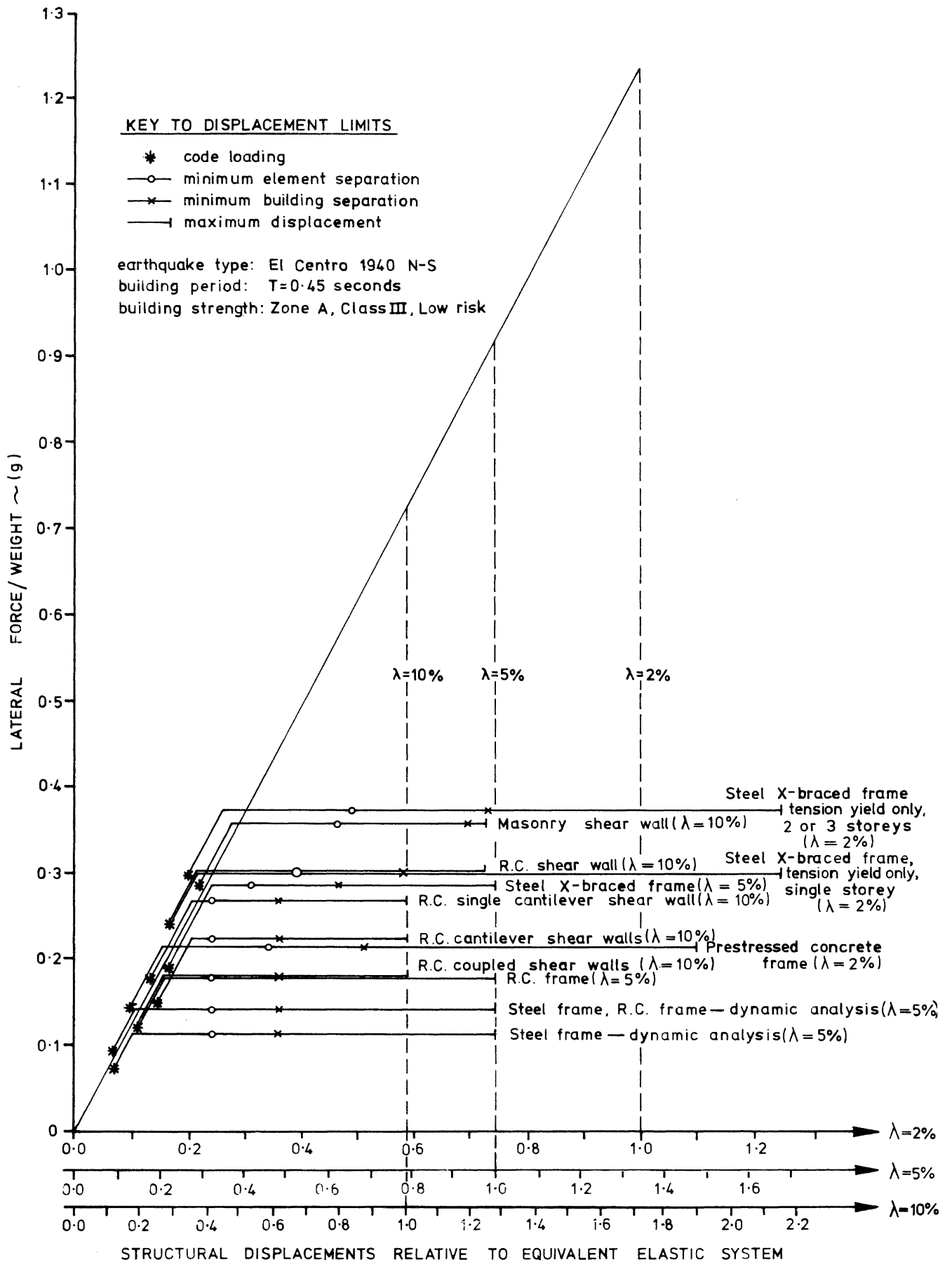


FIGURE 2: RELATIVE STRUCTURAL DISPLACEMENTS FOR SHORT PERIOD BUILDINGS

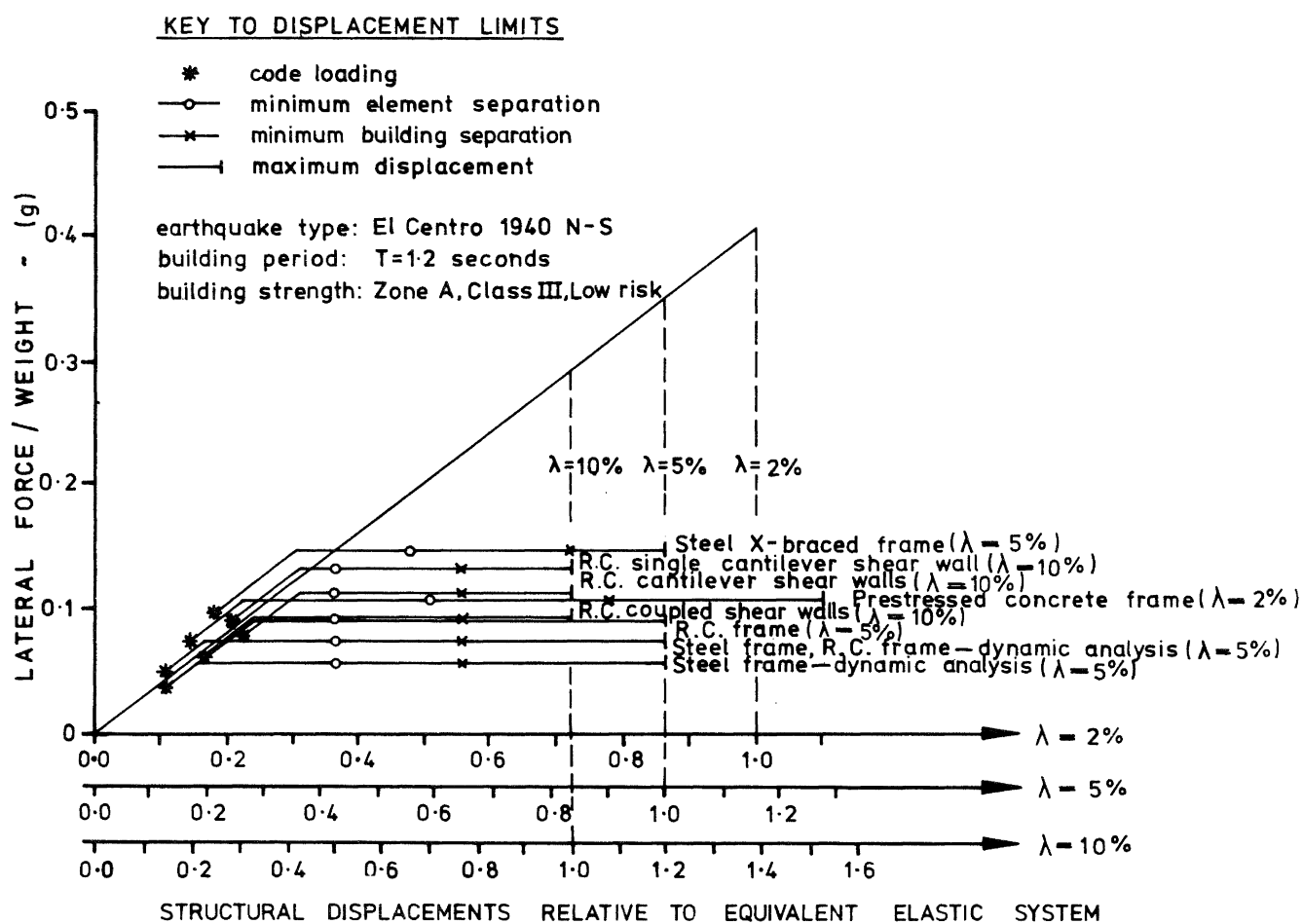


FIGURE 3: RELATIVE STRUCTURAL DISPLACEMENTS FOR LONG PERIOD BUILDINGS

APPENDIX

Since the New Zealand Code of Practice on which this paper is based, "General Structural Design and Design Loadings", is not available at the time of writing, the following clauses from the final revision of Part 4, Earthquake Provisions, are reproduced for clarification. Other details and commentary may be found in the draft code (1).

Clause 4.2 Total Horizontal Seismic Force

4.2.1 Every building shall be designed and constructed to withstand a total horizontal seismic force (V) in each direction under consideration in accordance with the following formula, except as provided in a simplified method for small buildings:

$$V = C_d W_t \quad \dots \dots \dots (1)$$

where $C_d = C I S \gamma R \quad \dots \dots \dots (2)$

and C is the basic seismic coefficient to be determined from Fig. A.1 in accordance with the seismic zone as shown in Fig. 2, Part 4 of the

draft code (1), the subsoil flexibility as defined in Clause 4.3 of the draft code (1) and the period (T) in the direction under consideration. The minimum values for C shall be those for T = 1.2 seconds. (It should be noted that the curves in Fig. 1, Part 4 of the draft code (1) were drawn for the Class II buildings, as defined below. The curves reproduced here as proposed for the final revision of this code are applicable to Class III buildings. In compensation new importance factors must be used as given below.)

I shall be as given in table A.1
S shall be as given in table A.2
 γ shall be as given in table A.3
R shall be as given in table A.4
 W_t shall be the total seismic load as specified in Part 2 of the code, provided that tanks, reservoirs and the like shall be considered to contain their full contents.

The value of C_d need not be taken greater than 3.6 CI γ but no less than 0.04.

TABLE A.1

IMPORTANCE FACTOR, I

CLASS	TYPE OF OCCUPANCY	I
I	Essential facilities required to be completely functional immediately after a seismic disaster	1.6
II	Public buildings	1.3
III	Buildings other than Class I or Class II	1.0

TABLE A.2

STRUCTURAL TYPE FACTOR, S

(To be determined separately for each direction under consideration)

ITEM	TYPE OF STRUCTURE	S
1	Ductile frames with an adequate number of beam hinges	0.8
2	Ductile frames with an inadequate number of beam hinges	1.0
3	Ductile coupled shear walls	0.8
4	Two or more approximately symmetrically arranged ductile cantilever shear walls	1.0
5	Single ductile cantilever shear wall	1.2
6	Shear walls not designed for ductile flexural yielding but having the ability to dissipate a significant amount of seismic energy	1.6
7	Buildings with diagonal bracing capable of plastic deformation in tension only:	
	(a) Single storey	2.0
	(b) Two or three storeys	2.5 or as determined by a special study
	(c) More than three storeys	As determined by a special study
8	(a) Buildings in which part of the horizontal load is resisted by item 7 bracing and part by an item 1 or item 2 frame.	As determined by a special study
	(b) Buildings with diagonal bracing capable of plastic deformation in both tension and compression	1.6 or as determined by a special study
9	All other buildings including chimneys and tanks or reservoirs on the ground.	2.0

TABLE A.3
STRUCTURAL MATERIAL FACTOR, γ

MATERIAL	γ
Structural Steel	0.8
Structural Timber: Shear-wall buildings Other buildings	0.8 1.0
Reinforced concrete	1.0
Prestressed concrete (when used in elements which resist seismic forces and movements by flexural yielding)	1.2
Reinforced masonry	1.2

TABLE A.4
RISK FACTOR, R
 (for usage and other risks)

TYPE OF BUILDING	R
Buildings other than those given below and presenting no unusual risk	1.0
Buildings accommodating more than 1,000 people, assembly buildings including theatres and cinemas	1.1
Distribution facilities for natural gas, coal gas, or petroleum products in urban areas	2.0
Structures and installations containing or directly supporting toxic liquids or gases, spirits, acids, alkalis, molten metal, or poisonous substances, including substances that could form dangerous gases if released	3.0

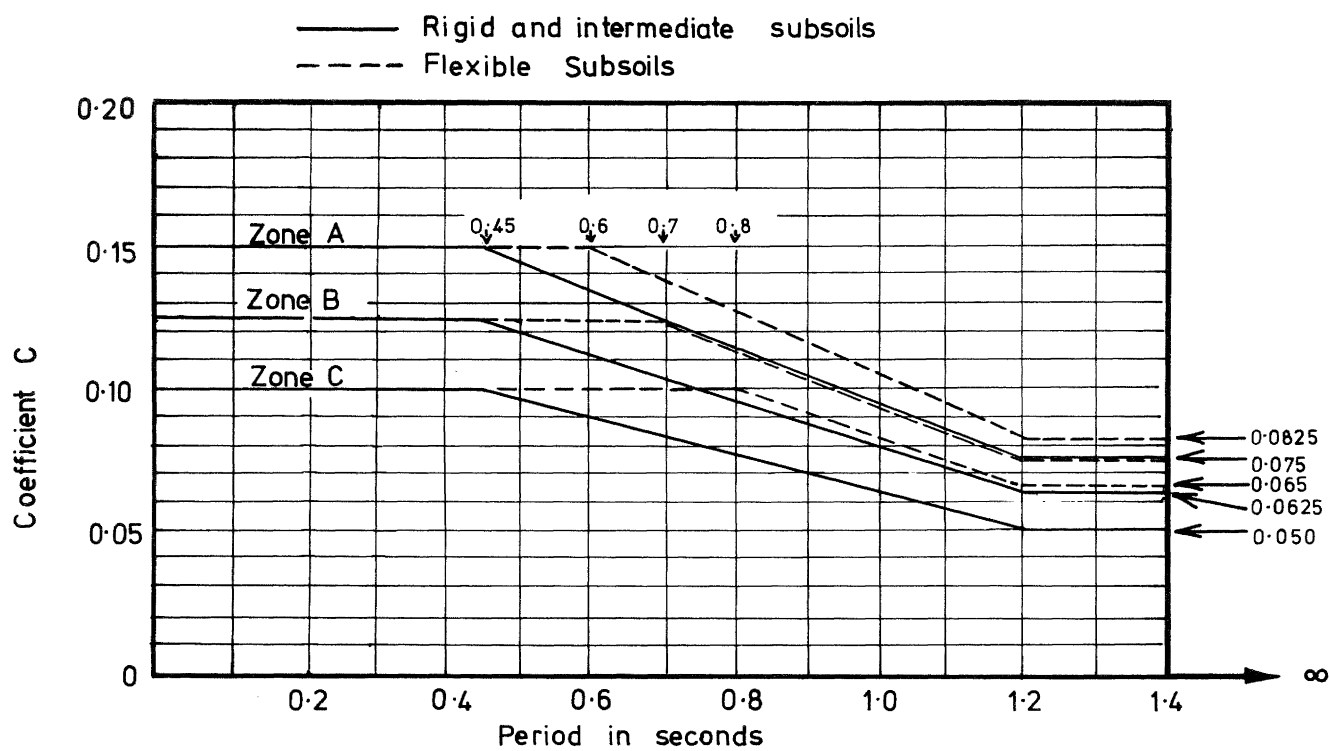


FIGURE A.1: BASIC SEISMIC COEFFICIENT 'C'