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The development of a rational procedure to determine structural performance factors

B J Davidson, R C Fenwick, Department of Civil & Environmental Engineering, University of Auckland, D K Bell, Compusoft Engineering Ltd

The School of Engineering

The University of Auckland

Department of Civil and Resource Engineering

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B. J. DAVIDSON

D. K. BELL
COMPUSOFT ENGINEERING LTD.

R. C. FENWICK

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Department of Civil and Environmental Engineering
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ABSTRACT

The New Zealand Loadings Standard, NZS4203:1992 incorporates a Structural Performance Factor, S_p, in the expressions for the lateral force coefficient "C". The reason given for its introduction was to allow for effects that were not addressed directly by other design spectra factors. The S_p factor was assigned a value of 0.67 at its introduction. This was considered to be "a reasonably average value", which it effectively maintained a similar average level of seismic demand in the transition from a 150 year design return period of the previous code to the 450 year return period of the new code.

This report examines the reasoning behind the introduction of the S_p factor. It concludes that many of the reasons that were quoted as justifying its introduction are already allowed for in the design process. However, there are two factors that can justify the use of the S_p factor. These are the "duration effect" and the influence of "structural redundancy". The first of these arises from the different ways the term "ductility" is used in the definition of the "C" factors, and the methods of assessing ductility levels in structural testing. Structural redundancy in statically indeterminate structures increases the robustness and reliability of the structure allowing it to perform better than the single degree of freedom structure, for which the "C" coefficients are developed.

A method of developing rational S_p factors is proposed by assessing damage levels using time history analyses and damage indices. It is concluded that a single factor for use with all limit states and design ductility levels cannot be justified. The value that is used should be specified in the materials code taking into account the ductility level together with the reliability and robustness of the structure.

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1.0 OVERVIEW

The New Zealand Loadings Standard, NZS4203:1992 [1] incorporates a Structural Performance Factor, S_p, in the expressions for the lateral force coefficient "C" and the seismic design spectrum, "C(T)". The reason given for its introduction was to allow for effects that were not addressed directly by other design spectra factors. In particular, the S_p factor was to take consideration of the fact that "a single peak response of short duration will not necessarily lead to damage [2]", and to account for observations that "buildings, on average, perform better than can be predicted by calculation using simplified analyses".

The S_p factor was assigned a value of 0.67 at its introduction. This was considered to be "a reasonably average value" which "should be appropriate for the majority of structures and material types in use in New Zealand". The 0.67 value effectively maintained a similar average level of seismic demand in the transition from a 150 year design return period of the previous code [3] to the 450 year return period of the current code. However, for short period structures it resulted in an increase in required strength, while for long period structures there was a considerable reduction in required strength.

This project investigates the validity of the S_p factor, and develops a procedure by which code writers and designers may assign rational values to the factor for specific structural forms.

1.1 OUTLINE OF STUDY

The study reviews the background to the introduction of the S_p factor which includes a discussion of the base shear equations from the two codes NZS4203 :84 [3,2] and :92 [1]. The S_p factor was introduced to account for a number of effects that influence the strength demands of actual structures as compared to the demands on idealized single degree of freedom oscillators and effects that have not been accounted for in previous codes. These effects are itemized, discussed and their worthiness for inclusion as a valid influence on the S_p factor decided.

The study then describes a proposed procedure for obtaining a consistent set of values for S_p factors that would be suitable over the range of structural forms and materials.

2.0 BACKGROUND TO INTRODUCTION OF STRUCTURAL PERFORMANCE FACTOR

In 1992 a limit state 'Loadings Standard', NZS 4203:1992 [1], was introduced in place of the 'Loadings Code, NZS 4203:1984 [3]. The Loadings Standard incorporates a Structural Performance Factor, S_p, in the expression for the lateral force coefficient "C" used to determine seismic design loads. It was stated in the commentary of the Standard that the S_p factor was introduced to allow for effects that were not addressed directly by other design spectra factors. This section of the report reviews and discusses the background of the introduction of the S_p factor.

2.1 SEISMIC DESIGN

The S_p factor must be assessed in the context of the New Zealand seismic design procedures. The following is a brief review of relevant seismic loading provisions.

2.1.1 1984 Loadings Code

The 1984 Loadings Code was based upon the previous 1976 [4] Code and required every building to be designed to resist a total horizontal seismic force, V, the value of which was determined by

$$V = C\,I\,S\,M\,R\,W_t. \qquad \text{Eq.2.1(a)}$$

or

$$V = C_d\,W_t \qquad \text{Eq.2.1(b)}$$

The Importance and Risk factors were typically "1" for most structures and "SM", the

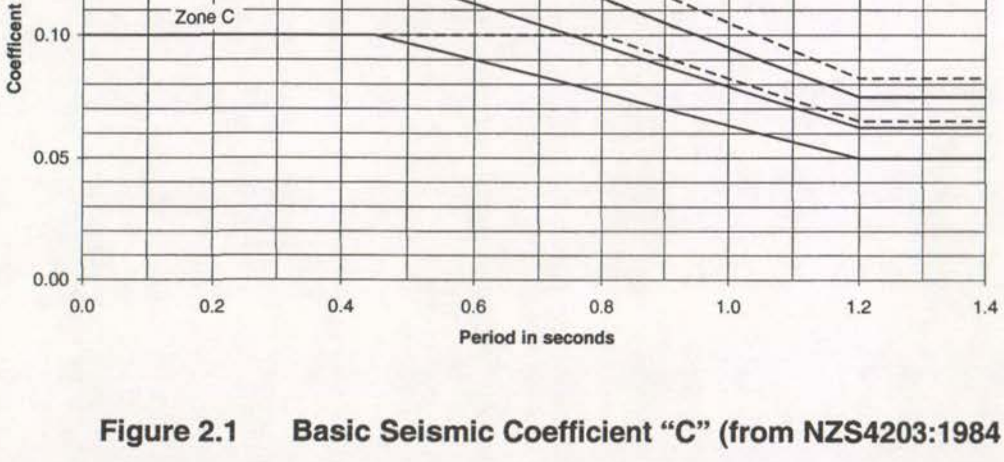

Figure 2.1 Basic Seismic Coefficient "C" (from NZS4203:1984 [3])

product of the Structural type and Material factors, varied from 0.64 to 5.0. W_t was the seismic weight of the structure and C, the Basic Seismic Coefficient. The form of this coefficient is reproduced in Fig. 2.1. As seen in the figure the basic seismic coefficient was provided for three seismic zones and two subsoil types. Its development was based on a 5% elastic spectrum with an approximate return period of 150 years [5]. The C coefficient given in the code was approximately one quarter of the appropriate elastic spectra and with the equal-displacement philosophy adopted by the code. It corresponded to a displacement ductility requirement of 4 [6,7].

The structural type and material factors effectively controlled the design level of ductility and made some allowances for structural damping and redundancy. For example, "ductile frames" constructed from steel or reinforced concrete were designed with a strength of "Cx0.64" or, using the assumption that the "C" factor was approximately one quarter the elastic spectrum, to $1/6.25^{th}$ the strength required of an elastically responding single degree of freedom oscillator. In contrast "elastically responding structures" constructed from reinforced concrete and masonry had a SM product equal to 4. Using the same assumption for the "C" coefficient they were effectively designed to the elastic spectral value. Elastically responding structural steel and prestress concrete structures were assigned SM products of 4.8 and 5 respectively, which corresponded to design strengths of 1.2 and 1.25 times the required elastic strengths. Though not explained in the code it is assumed that these amplifications from the elastic spectral values were made to account for anticipated lower levels of damping occurring in these material types compared to that in reinforced concrete. If the Kawashima [8] relationship

$$Sa(T,\xi)\,/Sa(T,.05) = (\,1.5\,/\,(\,1 + 40 * \xi\,) + 0.5\,)$$

is utilized then the SM products for steel and prestress concrete implies damping of 2.5 and 3% in these structures.

The 1984 Code differed from the 1976 code in the definition of a "ductile frame". The 1976 Code required that a ductile frame should have an "adequate number of hinges", which was an attempt to reward a design that incorporated an increase of reliability (robustness) into the structural system. This was not included in the 1984 upgrade of the Code.

2.1.2 1992 Loadings Standard [1]

In the 1992 Code the 'basic seismic coefficient' was replaced by the 'basic seismic hazard acceleration coefficient, C_h'. The values of this coefficient correspond to 5% Uniform Hazard Spectrum (UHS) with a return period of 450 years [2]. The increase in the return period from 150 to 450 years was made to ensure the New Zealand code was consistent with other international codes. The spectra are based on elastic UHS derived by the SANZ Seismic Risk Sub-Committee using modified Katayama attenuation relationships [9]. The basic seismic hazard acceleration coefficients are provided for three sites subsoil categories, "intermediate soils", "rock or very stiff soils", and "flexible or deep soil". For the "Equivalent Static Method" the basic design strength of the structure is determined by the base shear

$$V = C_h(T,\mu)\, S_p\, R\, Z\, L\, W_t\,. \qquad \text{Eq. 2.2(a)}$$

or

$$V = C\, W_t \qquad \text{Eq. 2.2(b)}$$

The S and M factors of the earlier code were effectively replaced by the structural ductility factor, μ. Inelastic spectra corresponding to selected ductility levels were introduced. These inelastic spectra were derived from the elastic spectra using the equal displacement concept for periods greater than 0.7 seconds and a relationship providing a transition from equal displacement to equal energy for periods below 0.7 seconds [10]. The validity of the spectra was checked by evaluating the response of elasto-plastic single degree of freedom models with varying levels of ductility using numerical integration time history analyses [11]. In the context of this report it is important to note that this validation was carried out using 6 ground motions and the displacement ductility calculated was determined as the average of the maximum values that occurred **once** only throughout the duration of each of the earthquakes.

In the 1992 Loadings Standard three new factors were introduced to define the lateral force coefficient, a zone factor, Z, a limit state factor, L, set at $\frac{1}{6}$ for serviceability and 1.0 for ultimate, and the structural performance factor, S_p, nominally set at 0.67. A typical basic seismic hazard acceleration coefficient plot, for "intermediate soil sites", is shown in Fig 2.2.

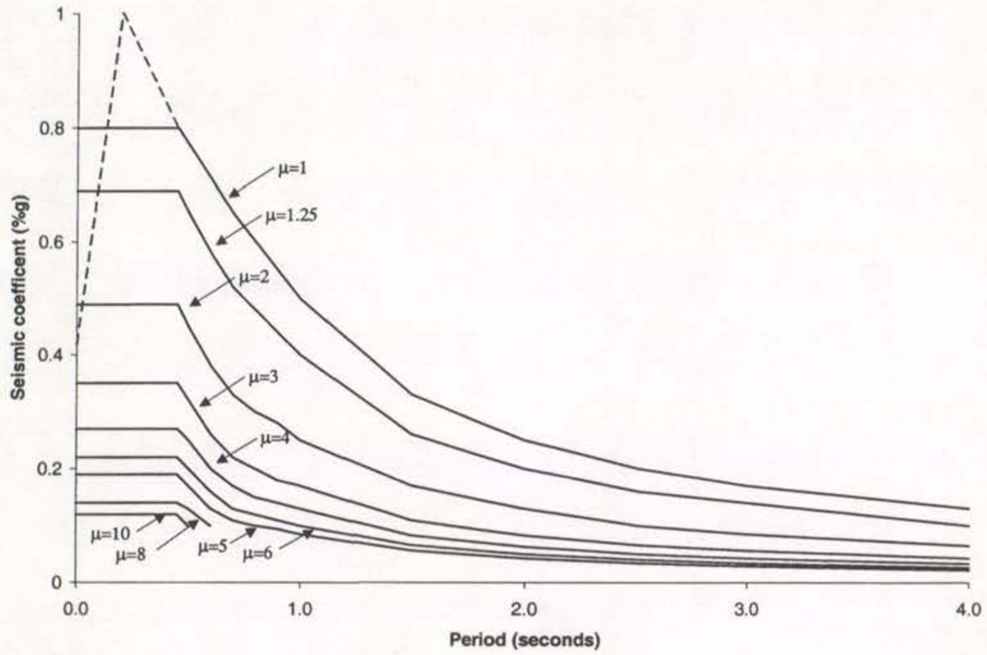

Figure 2.2 Basic Seismic Hazard Coefficient (from NZS4203:92)

2.1.3 Structural Ductility

The Loadings Standard assesses seismic demand on the basis of the ductility capacity of a structure. 'Ductility' is defined in the Loadings Standard as 'the ability of a structure to sustain its load carrying capacity and dissipate energy when subject to cyclic inelastic displacements during an earthquake'.

The 'Structural Ductility Factor' is used as the standard measure of ductility displacement in New Zealand limit State Codes. This factor is defined as 'a numerical assessment of the ability of the building to survive cyclic displacements'. The Loadings Standard guidelines in Appendix C4.A [2] specifies that the structure should be able to undergo four complete loading cycles to positive and negative displacements representing the nominated design displacement ductility and have a residual strength of not less that 80% of the maximum measured value.

Whereas surviving a test by 'four cycles to the displacement corresponding to the design ductility' is the Loading Standard requirement, to illustrate the ductility capacity of a structure or structural assemblage, the test method commonly used in New Zealand is that developed at the University of Canterbury [12]. The test was developed so that the ductility capacity of a test specimen need not be anticipated prior to starting the test, and so that it would provide approximately the same energy demands on the test specimen, as would the Code ductility requirement. The significant features of the test method are the means of standardizing the definition of the yield displacement of structures, and the progressive displacement control used. The yield displacement is defined as the yield displacement of the equivalent elasto-plastic system with reduced stiffness found as the secant stiffness taken to either 75 percent of the nominal strength, or the first yield strength, whichever is less. This definition is sketched in Fig. 2.3.

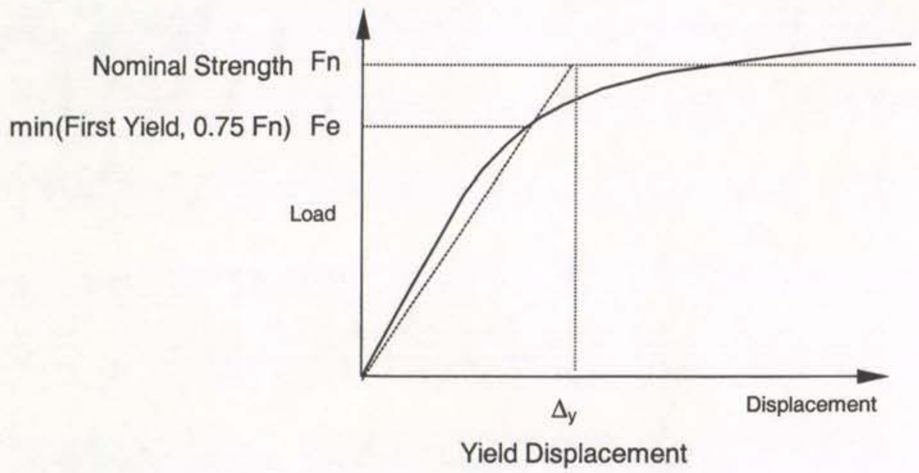

Figure 2.3 Definition of Yield Displacement (from Park [12])

Following the calculation of the yield displacement, the Canterbury test proceeds to cycle the displacement of the structure incrementally to a displacement ductility of +2, then -2, +2, -2, and two cycles each to ductilities of 4, 6, & 8 are applied. Using the results of this test, the ductility of the structure or structural assemblage tested is defined as:

$$\mu_a = \frac{\sum \mu}{8} \quad \leq 6 \qquad \text{Eq 2.3}$$

where $\sum \mu$ is the cumulative ductility at which the lateral load sustained has reduced to 80%. The displacement sequence for the test is illustrated in fig. 2.4.

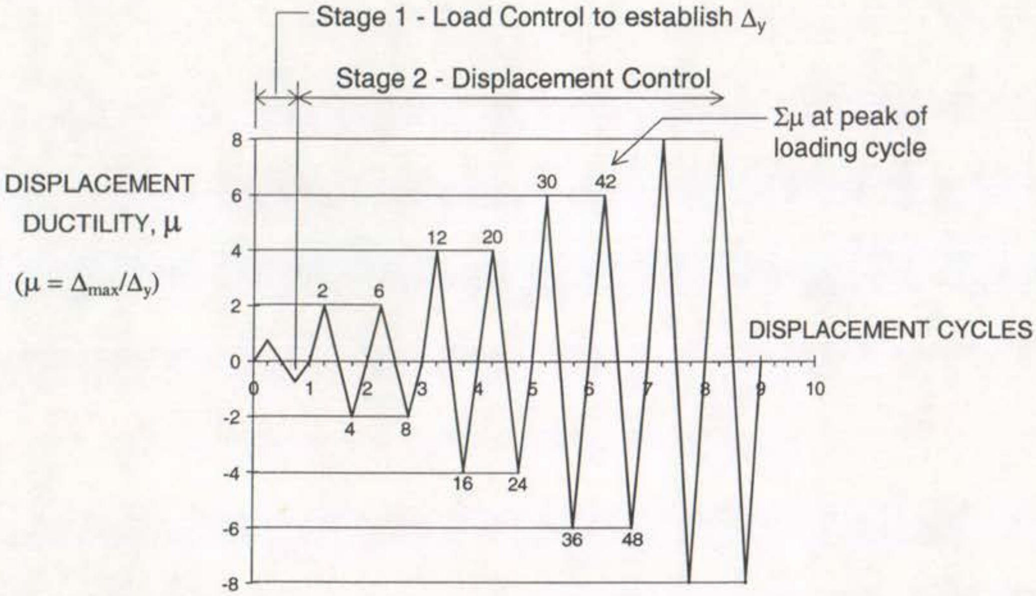

Figure 2.4 Test Sequence to establish the Structural Ductility Factor

In the context of this report it is worth noting that this test is described as suitable for 'structures and structural assemblages'. From a practical point of view almost all tests are performed on structural assemblages. As a consequence, typical test results do not demonstrate the increase in reliability of strength that may occur in a structure made up from a number of assemblages. For example, the force deflection response of a pinned portal frame is going to represent the moment rotation relationship of the beam hinges. We would expect the response of a multi-bay frame structure made up from a number of similar portals to perform better, as the weaker beams would be supported by the others through the redistribution of the moments. This concept is discussed further in section 5.

2.2 Structural Performance Factor

The S_p factor was introduced to account for effects that were not addressed directly by other design spectra factors. The S_p factor was assigned a value of 0.67 at its introduction. This value was considered to be "a reasonably average value" which "should be appropriate for the majority of structures and material types in use in New Zealand" [2]. The 0.67 value was assigned uniformly to all serviceability and ultimate limit state designs, with one exception. For the determination of inelastic effects in an ultimate limit state design using time-history analysis, an S_p value of 1.0 was specified. It was felt that with time-history analyses, duration of strong ground motion was considered explicitly.

In Figs. 2.5 to 2.7 the lateral force coefficient "C" (for the 1992 Code and calculated using a S_p = 0.67) and the seismic design coefficient "C_d" (for the 1984 Code) are compared. In Fig. 2.5 the elastic, μ=1, and SM=4 values are compared for the highest seismic zone in the 1984 Code (Zone A) and the two highest in the 1992 Code (Z=1.2 and 1.0). It can be seen that the two 1992 values bound the 1984 Code values for periods less than 1.5 seconds. This supports the theoretical calculation that states that the ratio of the elastic spectral values for earthquakes with return periods of 150 and 450 years is 0.67. It should be noted however that both the 1992 plots fall below the 1984 plot for periods greater than 1.5 seconds. The reason for this is that the 1984 C coefficients were kept at a conservatively high value for periods greater than 1.2 seconds. At the time of writing that Code it was felt that only high-rise structures would have periods in this range and some additional caution in their design was justified.

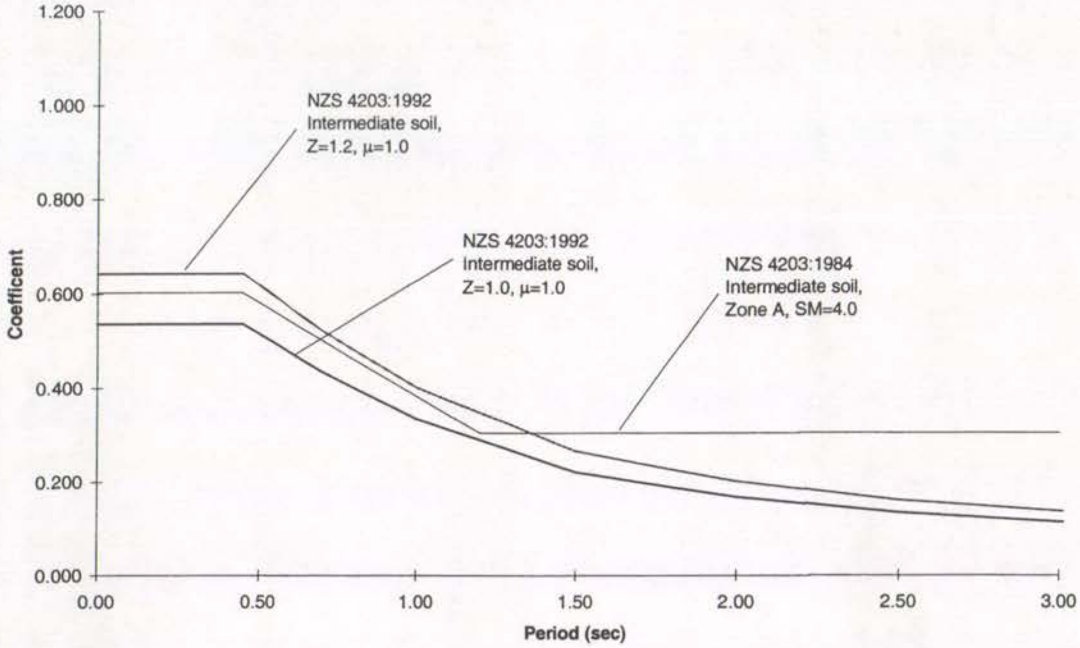

Figure 2.5 Comparison of Lateral Force Coefficients for Elastic Design

In Figs. 2.6 and 2.7 the ductility 4 and 6 curves are compared. Both Codes produce similar values for the lateral force coefficients in the period range 0.5 to 1.5 seconds. However in the short period range of less than 0.5 seconds, the 1992 results in values that are approximately 50% higher than the 1984 Code and at longer periods the lateral force coefficient values are approximately 50% lower than those from the 1984 Code. These figures confirm that for an elastic design the introduction of the S_p factor effectively maintained the design forces in the 1992 code at approximately the same average level of the 1984 Code over the period range. For designs based upon ductility 4 and 6 the same situation holds.

As explained above, the differences between the two codes at the longer period range is explained by the conservative approach of the 1984 Code. The difference at the short period range is the result of an incorrect scaling used in the 1984 Code. The 1984 Code implicitly used the same scale factor over the entire period range to obtain a ductility μ set of design forces and made no allowance for an "equal energy regime" that is postulated to act at the low period range. The 1992 Code makes allowance for this phenomenon.

2.3 THE EQUVALENT OF THE S_P FACTOR IN OTHER CODES

The term "structural performance factor" was introduced into the New Zealand loading Standard, NZS4203, in the 1992 edition [1]. This terminology is not used in other codes of practice. However, a number of recognised codes do apply factors that have a similar effect to the structural performance factor. These factors modify either, the design spectrum, or

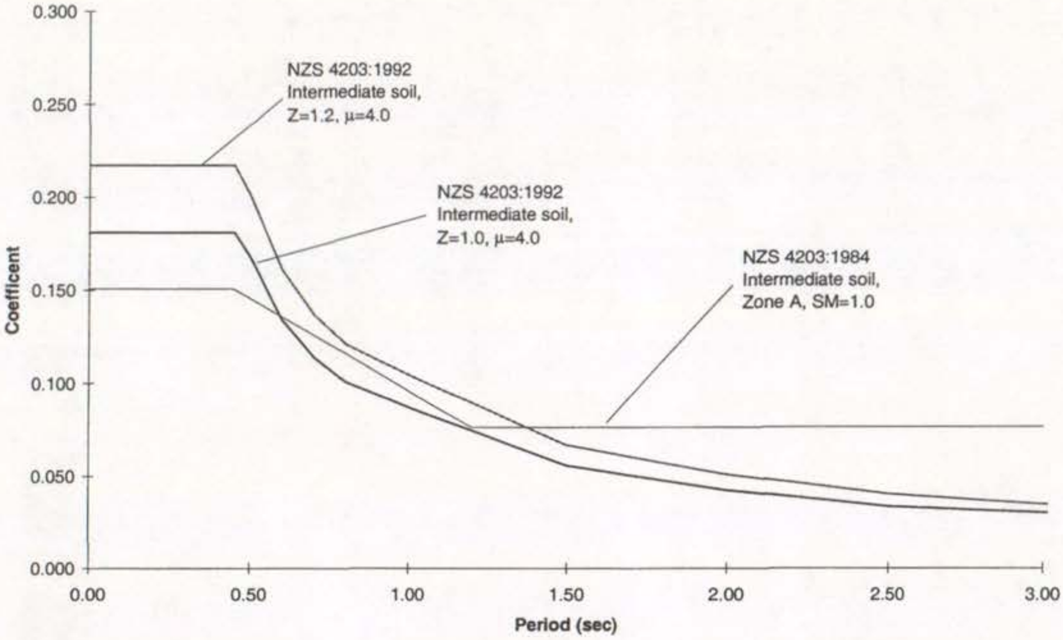

Figure 2.6 Comparison of Lateral Force Coefficients for Ductility 4 Design

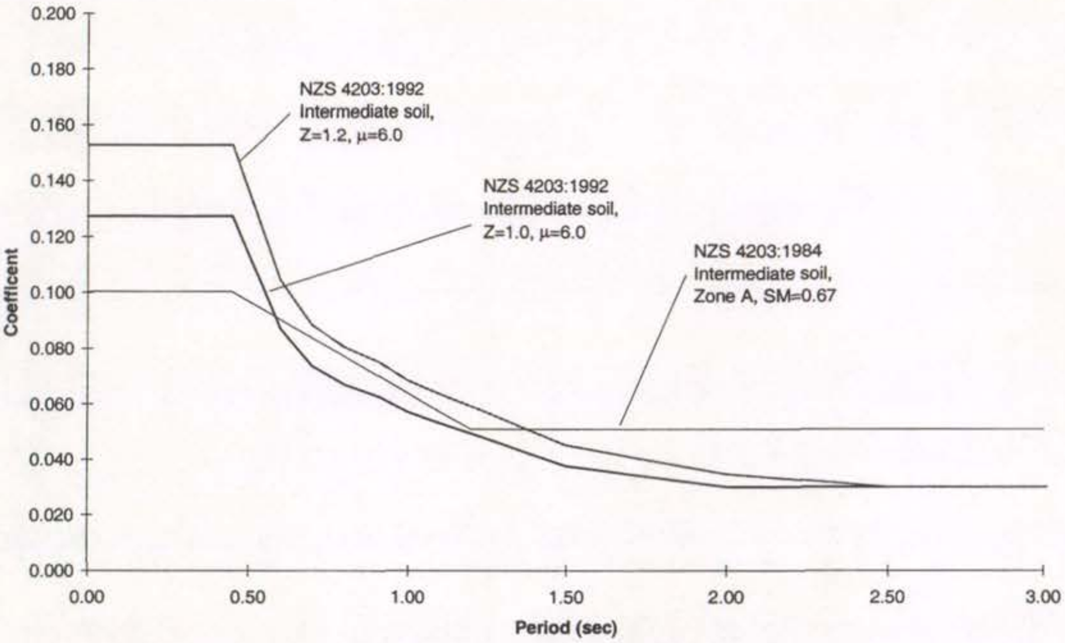

Figure 2.7 Comparison of Lateral Force Coefficients for Ductility 4 Design

the values obtained from an analysis, to allow for "duration effects", that is repeated inelastic deformation, and specific characteristics of behaviour that are not accounted for by the structural ductility factor, or its equivalent such as the response modification factor.

2.3.1 IBC 2000 Code of Practice [13]

This code of practice has a number of specific factors, which have an effect similar to that of the structural performance factor. There are two sources of these. The first is used to modify the response spectrum, with the extent of the modification depending on the structural form and structural materials. The second varies the required strength values found from an analysis, with the modification varying with the redundancy of the structure.

The design spectral response values are divided by a response modification factor, "R", and multiplied by the importance factor, I. The value of R ranges from 1.25 to 8 with different structural forms and materials. This factor is equivalent in New Zealand code terminology [1] to the structural ductility factor divided by the structural performance factor (μ / S_p). The analysis using this spectrum gives the minimum required strengths. As in the New Zealand Standards some strength values are increased to ensure that in the event of a major earthquake a ductile failure mechanism develops in preference to non-ductile failure modes (capacity design). The corresponding design displacements are found by multiplying the displacements found in the analysis by a deflection amplification factor, C_d, which allows for the expected increase in displacement due to in elastic deformation. The ratio of C_d / R is equivalent to the S_p factor in the New Zealand Loadings Standard. In the IBC code values of C_d and R are given for a wide range of materials and structural forms in Table 1617.6, which occupies $3^1/_2$ pages. A few typical values are reproduced in the Table 2.3 below. From this Table it can be seen that the equivalent S_p value ranges for 0.5 to 1.0.

A redundancy factor, which modifies the required strength found using the design spectrum, depends on the level of redundancy of the system. The importance of this characteristic in a structure on seismic performance has been illustrated in a number of structural collapses, such as the Cyprus Viaduct, where the failure of hinge elements led, or made a major contribution, to the failure of a very significant portion of the structure. This particular viaduct was supported by portal frames, which had been made statically determinate to simplify their analysis. The result was that the failure of a hinge element on the portal destroyed the lateral resistance of the structure and allowed collapse to occur. Caltrans, in assessing the potential seismic performance of their bridges, identified the level of redundancy as a major factor in contributing to seismic performance.

The IBC 200 code requires a redundancy factor, ρ, to be determined for structures, which are in zones of moderate and high seismic activity. The factor, ρ, has a minimum value of 1 and a maximum value of 1.5, and values between these two limits are given by the expression-

$$\rho = 2.0 - 6.1/(r_{max} \sqrt{A})$$

where A is the floor area in square metres, and r_{max} is the ratio of greatest shear resisted by a single element to the storey shear in the direction being considered. Rules are given for calculating the r_{max} values in different structural elements. For example, for moment resisting frames the value of shear is calculated for each bay of the frame, with the bay shear consisting of the sum of the shears resisted by the columns if there is only one bay in the frame. However, if one of the columns contributes to more than one bay only 70% of its shear may be counted with the bay being considered. For walls critical ratio of r_{max} is taken as the product of the shear resisted by the wall times the factor of $3.3/L_w$, all divided by the storey shear. The value of L_w is the length of the wall in metres.

Table 2.3: Typical response modification and deflection amplification factors from the IBC 2000 code and the equivalent S_p factors

Seismic Force resisting System	Response Modification coefficient "R"	Deflection Amplification factor C_d	Equivalent factor S_p
Building frame system			
Structural steel eccentrically braced frames	8	4	0.500
Moment frame systems			
Special structural steel moment resisting frames	8	5.5	0.688
Ordinary structural Steel moment resisting frames	4	3.5	0.875
Special reinforced concrete moment frames	8	5.5	0.688
Bearing wall systems			
Special reinforced concrete shear walls	5.5	5	0.901
Ordinary reinforced masonry shear walls	2.5	1.75	0.700
Light frame walls with shear panels-wood structural panels/sheet steel panels	6	4	0.667
Inverted pendulum Systems			
Cantilever column systems	2.5	2.5	1.000

2.3.2 UBC 1997 Code of Practice [14]

The approach in this code of practice was similar to that adopted by the IBC code. However, instead of specifying a range of deflection amplification factors just one value was given, namely 0.7 R, where R was the response modification factor. In effect this corresponds to an S_p value of 0.7.

The UBC code has a redundancy factor, ρ, which was very similar to that used in the IBC document.

2.3.3 NZS 4203, 1976 and 1984 [4,3]

In both these codes the importance of redundancy was recognised. In particular the structural type factor was varied depending on the level of redundancy. Ductile frames were given a 25% increase in required strength where these were considered to have an inadequate number of potential plastic hinge zones. Likewise singe ductile walls on one line were designed for a strength that was 20 greater than that required where two or more walls were used in line. It is surprising his recognition of the importance of redundancy on seismic performance was not carried forward into later editions of the code.

3.0 A REVIEW OF SOURCES CONTRIBUTING TO THE STRUCTURAL PERFORMANCE FACTOR.

3.1 INTRODUCTION

The S_p factor was introduced to account for effects that were not addressed directly by other design spectra factors. The Commentary to the Loadings Standard [2] presents two specific arguments for the introduction of the factor.

1. The design spectrum indicates the likely level of a single peak response. However "a single peak displacement will not necessarily lead to damage", rather "it is more appropriate to consider a number of cycles of motion of sustained high response".

2. To account for observations that "buildings, on average, perform better than can be predicted by calculation using simplified analyses".

The Standard listed a number of sources which may lead to the actual performance of buildings being better than that predicted by analysis as;

(i) Higher material and member strengths and better performance;

(ii) Greater redundancy of the structure than assumed (it was referring to non-structural members);

(iii) Beneficial effects arising from geometric changes, which reflect the simplifications that may be used by designers in modelling the structures;

(iv) An increase in damping and period after the onset of damage;

(v) Energy dissipation from elements not considered in the design as contributing;

(vi) Damping from radiation of energy associated with the interaction of the structure with the ground.

To help support its claim that the above sources were valid the Standard quoted two reports, namely, a 1982 Applied Technology Council report by Matthiesen and Joyner [15] "An investigation of the correlation between earthquake ground motion and building performance", and a 1982 Earthquake Engineering Research Institute publication, "Earthquake Design Criteria", by Housner and Jennings [16].

The applicability of these two reports is first reviewed then assessments are made of each of the sources highlighted by the Loadings Standard as to contributing to enhancing the performance of buildings.

3.1.1 Applied Technology Council (ATC-10) Report [15]

A review of the report reveals that its emphasis was on the correlation of peak ground acceleration and the corresponding code design forces, with reference to building performance. The study discusses the influence on building performance of factors such as peak ground acceleration, earthquake duration and vibration frequency, and structure dynamic characteristics, energy dissipation and strength. A detailed section of the study investigates the apparent ability of buildings to sustain seismic motion in excess of the design seismic levels. This investigation involved the evaluation of the 'actual' capacities of selected representative buildings relative to their 'design' capacities and supported the inclusion of the sources listed above as possible reasons why buildings, at least of North American design, would perform better than expected when referred to their nominal design strengths.

3.1.2 Earthquake Engineering Research Institute (EERI) Report [16]

A section of the report looks at the recorded behaviour of 14 multi-storey concrete buildings in the 1971 San Fernando Earthquake. A comparison is made between the base shear calculated from the recorded response of the buildings and the design base shear specified by the Los Angeles building code. The comparison showed that the calculated base shears were two to three times the design shears. Despite this, only a few of the buildings suffered structural damage and this was typically at a low level.

The EERI report [16] assigns the differences between design and observed strength primarily to the limitations of the building codes and the factors of safety incorporated into the codes. The report emphasises that the variation between design and observed strength is not uniform, but varies due to factors such as structural form, structural detailing, and soil-structure interaction. The EERI report therefore appears to highlight factors, which can be accounted for explicitly in the standards, rather than through the use of the Structural Performance factor, and/or suggests the use of an S_p factor that takes different values for different structural forms.

From a review of the seismic provisions of the design standards, the above referenced reports and related documents three additional sources that could contribute to the S_p factor have been identified. These are;

(vi) Structural hysteretic form;

(vii) Structural system redundancy;

(viii) Structural system reliability.

The following sections assess the contributions of the above sources to the S_p factor.

3.2 Response Duration Effects

It is widely recognised that the duration of excitation is an important factor in the assessment of structural capacity and damage [12,15,16,17]. Tests on structures have shown that damage increases with the number of large displacement cycles [18,19]. The repeated high-level deformations, which arise in an earthquake, and cause much of the structural damage, are closely linked to the duration of the strong ground motion. For this reason they are referred to as duration effects.

The effects of duration are reflected in the definition of the structural ductility factor, as required by the Loadings Standard [1,2], which requires a structure to be capable of sustaining the specified deformation ductility for four complete load cycles, which involves eight load reversals. Hence this definition may be seen as a measure of cumulative ductility demand in addition to peak displacement ductility.

In contrast, design spectra specify the peak displacement ductility demand, and have no information on the number of repetitions of displacements involving inelastic deformation, that is the cumulative ductility demand. There is a feeling that these peak displacement ductility design spectra over-estimate the seismic demand [2,20,21].

There appears to be a need to reconcile the seismic demand specified by design spectra and the capacity implied by structural ductility factors obtained from tests. This could be achieved through the application of a Structural Performance factor.

3.3 STRUCTURAL STRENGTH

Modern material Standards allow for a realistic assessment of the ideal strength (nominal strength) of structural members. Strength reduction factors are then assigned to provide a level of safety considered appropriate. Over-strength of members in a structure is therefore already accounted for in the design process and should not contribute to the S_p factor.

In ultimate state design, the capacity of a structure may be significantly greater than that where the first members reach their nominal strength. Provided the structure has sufficient ductility it can typically sustain an increase in loads until a failure mechanism is formed. This 'structure over strength' can also accounted for explicitly in the design process. The New Zealand material Standards allow for the redistribution of elastic member forces, or in some cases plastic analysis, and hence this effect is covered. Additionally New Zealand Standards allow for the ductility demand to be calculated from the failure strength of the designed structure, rather than the initial required strength.

It is important that analysis and design methods used are consistent with assumptions made in the evaluation of seismic demand. Design spectra are developed assuming an elasto-plastic relationship, where yield strength is equal to the nominal strength. In contrast, the ultimate strength of structures may be significantly greater than the strength of the structure when the first member yields. Therefore in assigning the ductility of a structure, the designer is effectively defining a fit between the strength-displacement relationship of the structure and an ideal elasto-plastic relationship.

When the ductility factor of the structure is evaluated with the yield displacement defined as in Fig. 2-3, a best fit is made between the structure and an elasto-plastic system as shown in Fig. 3-1, elasto-plastic system 'a'. In this situation the inelastic design spectra demand corresponds to ultimate strength, and either plastic design methods or design using elastic-redistribution, are appropriate. If the structural ductility factor is evaluated with the yield displacement defined as the displacement at "first yield" the fit with the idealized elasto-plastic curve is less clear. It can be assumed that a design based on this definition is conservative, and this would be a justification for an S_p factor less than "1" if it occurred for all structures. This is of course not the situation. Many structures require only a few yielding members to form a mechanism and as a result, the lateral force sustained when the first member reaches its nominal strength approximately corresponds to the ultimate lateral resistance of the structure. This is also the case for more complex structures that can be designed so that all members yield at approximately the same displacement.

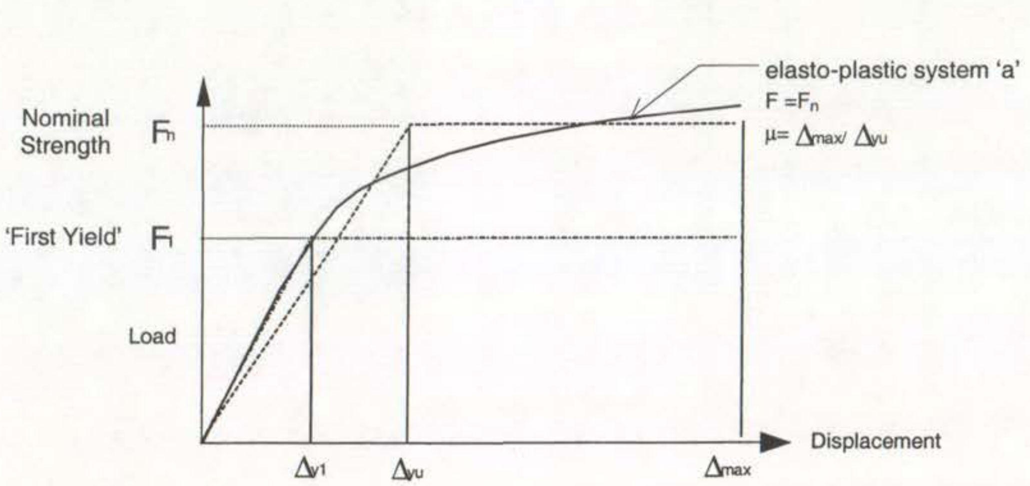

Figure 3.1 Equivalent Elasto-Plastic Systems

3.3 'NON-STRUCTURAL' STRENGTH

The structural model used in the evaluation of building capacity is typically a simplification of the actual building. A number of building elements that are deemed to add little stiffness or strength to the structure may be excluded from the model to simplify the analysis process. These 'non-structural' members may, however, contribute some level of seismic strength to a structure, but there is a large amount of variability in the strength provided to different structures by these members. As a consequence it is non-conservative to reduce the demand on all structures by including this as a possible source for the S_p factor. It should be noted that non-structural members are typically not designed or detailed to carry seismic forces. When subjected to reversing loading, unless specifically detailed, these members may lose their structural integrity and as a consequence cannot be relied upon.

3.4 GEOMETRIC CHANGES

The Loadings Standard [2] refers to beneficial effects of geometric changes, for example changes brought about by variation in stiffness due to centroid shifts, a mechanism that might occur in structures with large members. These effects would however, only apply to some buildings, and could be included in the analysis of the building if the designer proved them to be significant. As a consequence it is considered unwise to include these effects as a reason for reducing the value of the S_p factor for all buildings. A recommendation of this report is that it is better for the designer to include these types of effects in the analysis of the structure when justified.

3.5 NON-LINEAR BEHAVIOUR

Increases in effective damping and period after the on set of damage typically leads to a reduction in seismic demand. These effects are however, already accounted for through the use of non-linear design spectra, and therefore should not be accounted for with the S_p factor.

3.6 Damping

Design spectra are developed for systems with 5% viscous damping. The level of damping in a structures may vary from this 5%, due not only to contributions from non-structural elements, but also to the variation of damping with structural form. Consequently there is a case for making adjustments to the 5% damped design.

In the assessment of the effects of damping, it is important to recognise the different forms of damping. 'Hysteretic' damping is the damping associated with the non-linear behaviour of material in 'plastic hinge' zones in structural elements. 'Equivalent viscous' damping accounts for all other forms of damping in a building, such as cracking and rubbing of non-structural elements, radiation damping, and small amounts of yielding and cracking in structural members away from hinge zones [16].

Damping can increase significantly when buildings are subject to strong earthquakes [16,22]. However, much of the increase is due to hysteretic damping of the ductile structural elements. The effects of this hysteretic damping are already accounted for through the use of inelastic design spectra [23]. Therefore only the variations in viscous damping from the nominal 5% should be considered.

Studies indicate that the sensitivity of spectra to damping is dependant on total damping levels. For systems with a high level of damping, further changes in damping have little effect on response levels. This means that at levels of high ductility, where hysteretic damping may be equivalent to around 10% viscous damping [24], the effects of viscous damping on system response are small [25].

For ductile structures, there is a requirement for a reasonable amount of energy dissipation, so changes in viscous damping are therefore likely to have a small effect on response. Additionally, for ductile behaviour there are practical problems in assigning the total damping between the equivalent viscous damping and the effects of ductility [16].

It appears that adjustments of response for damping may only be significant for structures with low ductility levels, effectively structures designed elastically or for limited ductility [25,26]. Examples of cases where damping adjustments may be significant are the elastic design of open frames, where the level of damping from non-structural elements would be low, and the ductile design of pre-stressed systems, where the level of hysteretic damping is low. In these cases, there may be a need to increase the seismic coefficient to account for the low viscous damping level.

The effect of damping on system response is a matter that should be dealt with by the material standards. The current New Zealand Structural Steel Standard [27] provides estimates of damping levels and allows for the factoring of seismic coefficients for elastic and limited ductility structures using a relationship from the Japanese bridge design standard. The approach used by the Steel Standard appears consistent with the reviewed research. It should be noted that currently beneficial damping effects may utilised twice in a steel design, explicitly through the use of a damping factor and implicitly through the S_p factor.

3.7 FOUNDATION DAMPING

The seismic response of large massive structures can be reduced by the absorption of energy into the foundation. This energy is dissipated by "radiation" and for large rocking motions it may be absorbed through the non-linear behaviour of the soil. These damping processes do not uniformly influence the response of all buildings and for structures of small seismic mass may be of little consequence. Detailed soil-structure interaction analyses can be performed on structures where this phenomenon is known to be significant and there may be justification for a reduction of seismic design forces for other types of structures. This reduction would not be uniformly made across all structures but made dependent upon the massiveness of the structure, the soil type and the level of seismic design forces.

3.8 STRUCTURE HYSTERETIC FORM

The hysteretic form of a structure may vary significantly from the idealised elasto-plastic hysteretic form assumed for the in-elastic design spectra in the New Zealand Loadings Standard. If this variation in hysteretic form influences seismic demand, there may be a reason for the adjustment of seismic coefficients on the basis of structural type. Of particular concern are the effects of stiffness degradation and lower levels of hysteretic damping in real systems.

3.8.1 Studies of Hysteretic effects on seismic demand

Recent studies of hysteretic effects on response levels have suggested that the differences are generally of low significance. A review by Park (1989) [12] of research in the area concluded that the hysteretic loop shape typically does not have a major influence on inelastic earthquake response. A study by Moss et al [6] on the seismic response of low rise structures concluded that the shape of the hysteretic loop is not a major influence on dynamic response; test results showed the form of hysteretic model had a small effect on response and ductility demand compared with the form of the earthquake [25]. An explanation given for this was the reduction in the sensitivity of response to damping as the level of damping in a system increases. A study by Dean et al [7] on the seismic behaviour of plywood sheathed concluded that reduction in structure stiffness rather than increased hysteretic damping is the primary mechanism by which the seismic demand is reduced for ductile structures.

Mahin and Bertero [26] found the effects of hysteretic form on ductility demand typically to be small. Degrading systems were found to dissipate about the same amount of energy hysteretically as elasto-plastic systems. A reason given for this is that elasto-plastic systems only dissipate energy when full strength is reached, while stiffness degrading systems dissipate energy on most cycles after first yield. Kitayama [28] investigated the inelastic dynamic response of multi-storey reinforced concrete moment frames with plastic hinge behaviour modelled by stiffness degrading hysteretic loops. The effect of significant pinching on the response was found to be relatively small.

A study by Iwan [24] to estimate inelastic displacement response spectra from elastic spectra looked at the influence of hysteretic form. The results showed that degrading systems had a greater effective period at high ductility levels, but there was no significant difference in effective damping. Overall the study concluded that the sensitivity of the displacement spectra to hysteretic form was low.

A study by Deam [29] looked at producing response spectra for structures with pinched hysteretic behaviour. The study concluded that the behaviour of timber buildings does not support the theory that pinched hysteretic loops mean a greater seismic demand. Studies in fact suggest that pinched systems may actually have a lower seismic demand, due to the softening of the system. In comparison with elasto-plastic systems, peak accelerations decreased without corresponding increases in peak displacements.

It should be noted that in the reviewed studies, it is typically the yield strength and initial stiffness of the test system hysteretic loops, which are matched against those of the control elasto-plastic systems. The test systems therefore had a greater ultimate strength than the elasto-plastic systems. There appears to be some evidence that ductility demand reduces as a result of 'post yield' strength increases, though this is small in comparison to the other effects.

Reviewed studies did identify some hysteretic features, which appear to increase seismic demand. Park concluded that a structure must have some level of 'viscous' and hysteretic damping. Moss identified pinched systems with a short period and high ductility demand and systems with degrading strength as requiring a greater seismic demand. Mahin and Bertero identified hysteretic loops with negative post-yield stiffness. A study by Thompson and Park [30] found that prestressed concrete systems, which have narrow hysteretic loops, have approximately 30% greater displacement demand than reinforced concrete systems of similar initial strength, stiffness and damping. Studies by Fenwick and Davidson [31] on P-delta effects indicate that seismic demand increases with strength degradation for bi-linear systems. However they show that this is not necessarily the case for stiffness degrading systems. The effects of degrading stiffness alone are not clear; there is large variation for the cases considered, though it does appear that seismic demand is increased for periods less than 1.0 second when there is a high ductility demand.

Aoyama [32] presented the method for the evaluation of the seismic capacity of existing reinforced concrete buildings in Japan (medium to low rise buildings where the equal energy criterion applies). The Japanese standard for the seismic capacity evaluation of existing reinforced concrete buildings uses a basic seismic capacity index, E_o, which incorporates a ductility index, F. The ductility index is a function not only of ductility, but also of the characteristics of the structure hysteretic behaviour. This adjusts for variation from the ideal elasto-plastic model. The effects of hysteretic behaviour were studied using a model, which was effectively a elasto-plastic model with degrading stiffness, though on first loading there is a greater stiffness, corresponding to the pre-cracked state. The study indicated that for lower ductilities, strength requirements were lower than those implied by the equal energy assumption, while for high ductilities strength requirements may be higher.

3.8.2 Assessment of Hysteresis Effects

The reviewed studies generally indicate that the effects of hysteretic loop shape on seismic demand are small compared with other factors such as the variability of earthquake ground motion. Trends that are identified are:

- Greater seismic demand for 'pinched' looped low period systems at high ductility levels. However, it should be noted that P-delta effects may be reduced for these systems;

- Greater seismic demand for strength degrading systems, especially at higher ductility levels. However ductility requirements limit the use of this type of system;

- Some decrease in seismic demand for 'strain hardening' systems. This may in part compensate for the effects of stiffness degradation.

These trends suggest that there may be some argument for factoring the seismic coefficient for systems with high ductility, especially those with low ('equivalent viscous') damping. The factoring would be period dependant. It appears that the matter is best dealt with in the material codes. One possibility may be to review the limits on the ductility capacity of systems with undesirable hysteretic characteristics. Alternatively seismic coefficients could be adjusted according to the relative hysteretic damping levels and effective period shift.

3.9 STRUCTURAL REDUNDANCY

The Loadings Standard Commentary makes reference to structural redundancy provided by non-structural or secondary members. Structural redundancy may also be present in the principal seismic resisting structural system. When a number of load paths are provided in a seismic resisting system, it is possible that part of the system can fail without a significant reduction is the seismic capacity of the system as a whole. For example, with a failure of a member in a multi-bay frame redistribution of seismic actions can result in the structure still having significant seismic resistance. In contrast, the failure of a member in a single bay frame is likely to result in collapse of the frame.

At present there is no specific recognition of structural redundancy in the evaluation of seismic demand or capacity. As a result, structures with a greater level of redundancy effectively have a greater factor safety against collapse. This is not consistent with the philosophy of the New Zealand design standards of providing a uniform level of protection for structures with the same risk factor. This inconsistency may be addressed through the application of a structural performance (or redundancy) factor.

3.10 STRUCTURE SYSTEM RELIABILITY

The capacity of a structure is a stochastic variable. Material properties, analysis and design processes and construction standards, all provide a level of uncertainty. The uncertainty in these variables is recognised through the application of a strength reduction factor, ϕ, by the material standards. There is, however, no recognition of how the uncertainty from these variables contributes to the uncertainty in the overall performance of a structural system. The uncertainty in the capacity of a structure varies with the level of form of the structure. For example, in a multi-bay frame there may be a large degree of variation of the actual strength of individual bays. However, when averaged across the whole frame, the actual strength is likely to be close to the theoretical value (assuming no bias). The variation in uncertainty for different structural forms amounts to a variation in the reliability of the structural systems.

As with structural redundancy, variation in structural system reliability is not consistent with the philosophy of the New Zealand design standards of providing a uniform level of protection for structures. Structural redundancy and reliability are in fact closely related. Structural system reliability can be seen as a consequence of structural redundancy. It is therefore apparent that structural system reliability effects could be incorporated along with structural redundancy in a structural performance factor.

3.11 SUMMARY AND CONCLUSIONS

3.11.1 Summary

In the assignment of an S_p factor, the Loadings Standard makes reference to observations that "buildings, on average, perform better than can be predicted by calculation using simplified analyses". The referenced studies indicate that buildings have been subjected to excitation levels significantly greater than the relevant code levels without significant damage. However, on inspection of the studies it appears that this anomaly is principally due to two features of designs. Firstly, the seismic hazard specified was low, therefore seismic actions were not necessarily the factor that determined the required strength for all structural members. Secondly, the design processes were often simplified and conservative, therefore the design underestimated the structural capacity. For example no allowance may have been made for moment redistribution.

With the provisions of the current New Zealand limit state standards, it seems likely the seismic hazard and structure capacity may be assessed much more realistically than in the past; NZS 4203:1992 specifies 450 year return period inelastic hazard spectra and makes provision for dynamic analysis and inelastic design, while material codes effectively provide for plastic design. This situation reduces the need for the S_p factor to account for differences between observed and calculated behaviour.

The Loadings Standard highlighted a number of factors, which may contribute to the better performance of actual buildings. However, on inspection it appears that they may generally be accounted for directly in the design process through rigorous application of the design standards. Furthermore, the effect of these factors on seismic performance is likely to vary with the system ductility and limit state levels. Therefore a constant S_p factor would not be the most appropriate way to deal with them.

One of the specified sources of the S_p factor may require further consideration. Variation in equivalent viscous damping of structures at low ductility levels may lead to significant variations in response levels. However, as discussed in section 3.2.5, it appears that this matter may be suitably dealt with through the use of a damping coefficient, as currently specified in the current Steel Standard.

The Loadings Standard specifies a uniform S_p factor value of 0.67. A review of the sources specified by the Loadings Standard indicates that these alone do not provide a basis for the assigned S_p factor.

The use of response spectra corresponding to once attained maximum displacement in the design process does appear to provide a rational basis for a structural performance factor to reduce design forces. Studies indicate the performance of a structure is a function of both the magnitude of displacement and the repeated displacement involving inelastic deformation. There is however, a need to make a quantitative assessment of the repeated displacement effects on damage.

Three additional factors, which have not been addressed directly by the design standards and could contribute to an S_p factor, have been identified. An assessment of these factors indicated that two of these, namely, structural redundancy and structural reliability, needs to be addressed.

At present the Loadings Standard specifies a uniform S_p value for all systems. However, a review of the relevant literature, with reference to the S_p factor, indicates that building performance is likely to vary with the system ductility and limit state. A uniform S_p value therefore appears difficult to justify. The effect of ductility on the S_p factors sources is recognised by the Steel Structures Standard, which assigns an S_p factor of 1.0 to structures that are expected to respond to earthquakes in a non-ductile manner [22].

3.11.2 Conclusions

The review suggests that the majority of potential sources for the S_p factor are otherwise accounted for by the design standards. There are two major exceptions to this, namely, the number of repeated cycles involving inelastic deformation (response duration), and structure redundancy and reliability. The effects of these two sources need to be accounted for in the design process through the application of a structural performance factor, or other factor.

There is a need to quantify the effects of repeated inelastic deformation, and structure redundancy and reliability of the structural system on performance, so to assign appropriate structural performance factors. It is likely that these values will be a function of structural form, material, ductility and limit state.

The next stage for this study is to develop processes to evaluate the structural performance factors of the two identified sources. This involves the formation of evaluation procedures, the application of the procedures on sample structures systems, and the subsequent calibration of results to ensure consistent design levels. Structural performance factors could then be evaluated on the basis of material type, and be specified by the appropriate material standards.

Two of the reviewed sources, which need not be accounted for with the S_p factor, require further attention in developing the Design Standards. These are the level of equivalent viscous damping and the structure hysteretic form. Variations in the level of viscous damping can result in significant changes in seismic demand. This aspect is currently addressed in the Structural Steel Standard by the application of a damping factor. It is suggested that this approach be adopted for all forms of construction. Variations in the structure hysteretic form can influence structure performance in some cases. This matter is in part addressed indirectly by the material standards through the limits placed on the ductility of systems with undesirable hysteretic characteristics. It is suggested that this matter be reviewed. An alternative approach to placing limits on ductility is factor seismic coefficients according to the relative hysteretic damping levels and effective period shift.

4.0 EVALUATION OF S_P FACTOR FOR DURATION EFFECTS

The S_p factor provides a means of accounting for duration effects while retaining the seismic demand and capacity evaluation procedures currently in use. The term "duration effects' is used to describe the effects of repeated non-linear excursions of a simple system. Most of these would be at strain levels less than that associated with the design ductility level. However, during the passage of a design level earthquake it can be expected that the design level of strain will be exceeded. The S_p factor must effectively scale the current design spectra so that it reflects the system strength required to limit damage to sustainable levels, rather than to limit the peak ductility. The S_p factor is likely to be a function of structure form and materials, structure period, and ductility level.

4.1 MEASUREMENT OF RESPONSE DURATION EFFECTS

4.1.1 Sustained Response

There have been a number of studies, which have looked at the sustained level of response as a measure of seismic demand. Perez [33] and Kawashima [18,34] independently considered the response amplitude sustained for a specified number of cycles. Both studies produced elastic spectra giving the response amplitude as a function of the number of cycles in which the amplitude was matched or exceeded. The concept of an effective displacement response was proposed. Structural damage is a function of all inelastic displacements. For simplicity a single displacement may be taken to represent the total damage that is induced. This must have a value less than the peak displacement corresponding to a design spectra value and it in effect becomes the limiting design displacement. The Perez study showed that for structures with medium or long periods, the effective displacement response was typically significantly lower than the peak response. The Kawashima study produced an equation for a response duration factor, which related the peak response spectra to spectra for a specified number of cycles (the relationship was developed from an analysis of 394 components of ground motion!):

Given $S_A(T,h,N) = \eta(T,h,N) \times S_A(T,h,1)$ **Eq. 4.1**

where $S_A(T,h,N)$ represents the acceleration response spectrum for N cycles, as given by the Nth largest peak amplitude

and $S_A(T,h,1)$ represents the acceleration response spectrum for 1 cycle, that is, the conventional peak amplitude response spectra.

The response duration factor is given by:

$$\eta(T,h,N) = \frac{1}{1 + a(T,h)(N-1)} \qquad \textbf{Eq. 4.2}$$

where $a(T,h) = \dfrac{80h}{60h+1} \times 0.0815 \times T^{0.349}$ **Eq. 4.3**

where T is the period, h is the damping ratio, and N is the number of cycles for which the response is sustained.

The Kawashima [18] study report discussed the need to compare strength and loading as a function of the number of loading cycles in the evaluation of seismic performance. The study reported on cyclic load tests and shake table tests on reinforced concrete columns, which indicated that damage increases with the number of loading cycles. The most significant results were for shake table tests where test columns were excited by a ground acceleration record scaled to three different levels. Despite the different levels of excitation the test columns experienced similar peak accelerations. However, the damage sustained by the columns varied significantly. This supports the need to account for response duration, but it does not provide a means of relating repeated inelastic deformation and structural damage. Perez [33] noted the need for a means of correlating damage with response levels and duration.

While the Perez and Kawashima studies looked at the response of linear systems, Popov et al [20] looked at the sustained response of non-linear systems. They produced response spectra for an elasto-plastic system for a specified number of yield reversals (NYR). The plots show the spectra ordinates for 10 NYR to be 50% or less of those for 1 NYR. As the number of yield cycles can be seen as a measure of cumulative ductility, spectra based on yield cycles may be a more meaningful measure of seismic demand.

Spectra based on the number of sustained cycles provide information on the duration of high-level response. There is however, some question as to how such spectra can be used in the design process. Comparison of spectra for various numbers of cycles could show how the peak response relates to the sustained response (repeated displacements) and this perhaps could be used to determine the number of cycles for selected response ranges. To use this information would require knowledge of the system capacity as a function of number of cycles of response at different levels.

4.1.2 Cumulative Ductility

A means of accounting for duration effects is to measure cumulative ductility rather than peak ductility. The cumulative ductility demand of an earthquake on a system is given by the sum of the displacement ductilities for each load reversal. That is;

$$D = \sum_i \frac{\delta_i}{\delta_y} = \sum_i \mu_i \qquad \textbf{Eq. 4.4}$$

where δ_i is the total displacement for load reversal i, and δ_y is the yield displacement. A disadvantage of this is that elastic displacements contribute to the accumulated ductility but they induce virtually no damage.

An alternative form is:

$$D = \sum_i \frac{\delta_i - \delta_y}{\delta_y} = \sum_i \mu_i - 1 \qquad \textbf{Eq. 4.5}$$

This form is a more appropriate measure of damage as it measures the cumulative inelastic displacement demand.

A short coming of the cumulative ductility measure is that there does not appear to be a unique damage level associated with a given cumulative ductility, or accumulative inelastic ductility demand. Rather damage depends on the displacement ductility history of the structure. Studies have shown that a few load reversals to a high-displacement ductility produce greater damage than a many load reversals to a low ductility.

Failure of a plastic hinge zone generally occurs as a result of sustaining high strains together with the damage sustained due to the repeated application of these strains. It generally takes the form of a low cycle fatigue failure, and as such it depends on both the strain level and the strain history. As displacement ductility is at best only a very crudely related to strain neither equation 4.4 or 4.5 could be expected to define a damage realistic damage index.

4.1.3 Seismic Energy

Seismic energy provides a measure of response demand and capacity that accounts to some extent for response duration. Studies by Fenwick et al [17] on the response of bilinear-systems have shown that earthquakes that produce similar peak responses can have greatly different energy demands. This suggests there is a need to consider energy demand as well as peak accelerations.

Popov [20] looked at the seismic energy input of earthquakes and produced Seismic Energy Spectra as an alternative to acceleration response spectra. These spectra were developed by integrating response over the duration of an earthquake and consequently they inherently account for the duration effects. A more applicable measure of energy is the seismic hysteretic energy, found by integrating the area within the force-displacement response history of a system. As the hysteretic energy is expended in inelastic deformation the structure it may be seen as a measure of seismic damage. For an elasto-plastic system, the hysteretic energy in an earthquake is directly proportional to cumulative inelastic ductility demand (Eq. 4.5). The use of seismic energy therefore has the same limitations as the use of cumulative inelastic displacement demand.

The use of hysteretic energy as a measure does have the advantage of providing a means of accounting for different hysteretic relationships. However, studies by Fenwick et al indicate that energy demand for an earthquake is approximately independent of hysteretic shape [17].

4.1.4 Damage Indices

Damage indices provide a means of quantifying system performance in an earthquake. The structural ductility factor, cumulative ductility, and seismic energy are basic forms of damage indices. These measures do, however, have a number of shortcomings as discussed. A number of more complex damage indices have been proposed to take account of the response history. Often indices involve some form of weighted cumulative ductility, which recognises the severe damage caused by large displacements and perhaps the order in which they were sustained. There is the potential for damage indices to be used as a measure of seismic demand in place of the structural ductility factor, or to be used to factor the structural ductility factor spectra.

A theory used to develop a number of damage indices is the linear damage law proposed by Miner. This postulates that the damage produced by repeated loading at any level is directly proportional to the number of cycles at that level. Miner's rule uses stress as the measure of loading level. For assessing damage of ductile structures, peak deformation, or equivalently hysteretic energy, would be used as the loading measure. The damage index then is given by:

$$D = \sum_i \frac{N_i}{Nc_i} \qquad \textbf{Eq. 4.6}$$

where Nc_i is the number of cycles producing failure (100% damage) at a loading level S_i, and

N_i is the number of applied cycles at a loading level S_i.

The formula implies that the damage effect for a given number of cycles is independent on loading order. While a number of studies have looked at more rigorous damage theories, a generally reliable method is not available.

A ductility damage index calculated using the Miner's law can be seen as a measure of cumulative ductility. That is:

Given Nc_i, the number of cycles to failure at ductility μ_i :

$$Nc_i = 1/fn(\mu_i) \qquad \textbf{Eq. 4.7}$$

where $fn(\mu_i)$, a function of μ_i, is dependant on the structural properties of the system,

then the damage ratio, D, is given by:

$$D = \sum_i \frac{N_i}{Nc_i} = \sum_i N_i . fn(\mu_i) \qquad \textbf{Eq. 4.8}$$

Note that if the maximum number of cycles were inversely proportional to the inelastic displacement, then the damage ratio would be given by the cumulative ductility demand with no weighting. i.e.:

given $Nc_i = 1/(k.\mu_i)$ where k is a constant **Eq. 4.9**

then $D = k.\sum_i N_i.\mu_i$ **Eq. 4.10**

More realistically the maximum number of cycles would reduce at a greater rate. Cycle-displacement relationships are therefore likely to be in the form $1/\mu^n$ where n is greater than 1, or e^u. i.e.:

$$D = k.\sum_i N_i.\mu_i^n$$ **Eq. 4.11**

or $$D = k.\sum_i N_i.e^{a.\mu_i}$$ **Eq. 4.12**

Jeong and Iwan [35] applied Miner's law using the power form, estimating the number of cycles to failure, Nc_i, from:

$$Nc_i = c/\mu_i^{\ s}$$ **Eq. 4.13**

where c and s are constants which are properties of the structure form.

A number of damage indices use plastic displacement as a measure of demand in place of ductility. Wang and Shah [36] developed a cumulative plastic displacement damage index in which the rate of accumulation of damage is proportional to the damage that has already occurred:

$$D = \frac{e^{s\alpha_i} - 1}{e^s - 1}, \alpha = c\sum_i \frac{\delta_{mi}}{\delta_f}$$ **Eq. 4.14**

where c and s are constants,

δ_{mi} is the maximum deformation for cycle i,

δ_f is the deformation to failure in a single cycle test.

Stephens and Yao [37] developed a weighted cumulative plastic displacement damage index, which considered the ratio of the positive and negative cycles. The damage index is given by:

$$D = \sum_i \left(\frac{\Delta\delta_{pi}^{\ +}}{\Delta\delta_f} \right)^{1-br}$$ **Eq. 4.15**

where $r = \Delta\delta_p^{\ +} / \Delta\delta_p^{\ -}$

where $\Delta\delta_p^+$ and $\Delta\delta_p^-$ are the positive and negative plastic deformation increments for each cycle, $\Delta\delta_f$ is the value of $\Delta\delta_p^+$ in a single-cycle test to failure, and b is a constant.

The introduction of terms, which consider the direction of loading, is a significant extension of damage theory. For a number of structural types there may be some degree of independence between positive and negative cycle loading. Another approach is to consider is the order of the loading or deformation. An energy based damage index,, which considers both the direction and the order of loading, was developed by Kratzig et al [38]. A damage index is given for both positive and negative cycles:

$$D^{+} = \frac{\sum_i E^{+}_{p,i} + \sum_i E^{+}_{i}}{E^{+}_{f} + \sum_i E^{+}_{i}}$$ **Eq. 4.16**

where $E^{+}_{p,i}$ is the energy absorbed in the first cycle to a given level, and E^{+}_{i} is the energy of following cycles at lower load levels. An overall damage index is given by:

$$D = D^{+} + D^{-} - D^{+}D^{-}$$ **Eq. 4.17**

An alternative approach to using the complex cumulative demand damage indices is to specify a combined limit for cumulative demand (energy or ductility) and peak demand (ductility or displacement). This could be expressed viewed as a failure criterion, that is:

$$D = \left(\frac{E}{E_c}\right)^{n} + \left(\frac{\mu}{\mu_c}\right)^{n} : D \leq 1$$ **Eq. 4.18**

where E_c may be the energy capacity of the system at low ductility levels large number of cycles, and μ_c may be the displacement ductility capacity of the system for a single cycle.

Park and Ang [39] developed a damage index of this type;

$$D = \frac{\beta}{F_y.\delta_u}\int dE + \frac{\delta_m}{\delta_u} : D \leq 1$$ **Eq. 4.19**

δ_m is the maximum response deformation and dE is the incremental absorbed hysteretic energy. The displacement limit is given by the peak displacement for monotonic loading, δ_u. The energy limit is given by the energy capacity under monotonic loading, $F_y.\delta_u$, calibrated by a constant, β. Recommendations for evaluating these terms for steel and concrete structures were developed from a study, which analysed a large number of tests results.

The damage index of Park et al [40] may be expressed in terms of an equivalent maximum response, δ_m':

$$\delta_m' = \delta_m.\left[\frac{\beta}{F_y.\delta_m}\int dE + 1\right]$$ **Eq. 4.20**

or equivalently, an effective ductility factor, μ_m',

$$\mu_m' = \mu_m.\left[\frac{\beta.\delta_y}{F_y.\mu_m}\int dE + 1\right]$$ **Eq. 4.21**

Such a factor could be used in place of the peak ductility factor currently used in design processes.

The success of a damage index model depends on its reliable prediction of the damage state, the range of system and loading applications and its ease of application. Given the range of damage indices proposed, there are questions as to the accuracy of the indices in predicting damage. It is likely that the relative accuracy of the methods would depend on the nature of the application.

A study of Williams et al [41] looked at the ability of a range of damage indices to predict the damage of reinforced concrete elements loaded in combined shear and flexure. In this a simple peak ductility index was considered together with more complex relations. It was found that the damage indices all showed a high degree of scatter in predicting the damage state. The primary cause of damage of the test systems appeared to be the magnitude of deformation rather than the number of loading cycles. The study concluded that simple ductility based indices, such as peak ductility and the combined energy peak ductility index, provided the most reliable prediction of damage. It is important to note the parameters of the damage indices were not set by calibrating the models to the test data, rather recommended parameter values and values from the envelope of the cyclic-test force deflection loop were used. This approach was taken as it was considered to be a requirement for the practical application of the damage indices. This means that the limitations of the damage indices may be a result of difficulties in application rather the theory underlying the damage indices.

Studies by Park and Ang [39] to calibrate the combined energy peak ductility index illustrated the difficulty in obtaining a precise damage model. The calculated damage index parameters showed a high degree of scatter. The coefficient of variance of the reinforced concrete damage index model was approximately 0.5. In discussion of this feature the study noted that capacity under repeated cyclic loading is much less predictable than under monotonic loading. There also appeared to be an inherent variability in system parameters such as ductility.

4.1.5 Failure

The ultimate measure of the response of a structure to a specified level of loading is its state after the loading, which is survival or failure. In the ultimate limit state an appropriate definition of survival is the ability of a structure to sustain a further loading at a specified level. Performance then is measured by residual strength. If a full material model of a system is available, non-linear time history analysis could theoretically be used to determine the residual strength of a structure for a design earthquake. If this could be measured for 'typical structures', then the strength of the systems required by the time-history analyses could be compared with the strengths required by the code spectra to obtain an S_p scaling factor.

4.2 Evaluation Procedure

4.2.1 Outline of Procedure

The review in section 4.1 indicates that a suitable means of accounting for response duration effects is to use a damage index to assess the damage based strength demand on a system. The ratio of this to the ductility based strength demand gives the structural performance factor.

Duration effects (repeated cycles of inelastic deformation) could be evaluated on a structural type basis, and S_p factors could then be provided in a similar manner to the structural ductility factors, which are provided in the material standards. The designer therefore would be able to simply apply the appropriate S_p factor rather than go through the detailed evaluation process.

The procedure involved in calculating the S_p factor for each structural type would be:

I. Select a damage model and determine the model parameters, which correspond to the selected system structural type and structural ductility factor.

II. Evaluate the seismic coefficient for the selected system type and ductility using the damage model.

III. Calculate the S_p factor, given by the ratio of the design seismic coefficients corresponding to the damage capacity and the ductility factor.

The selection of damage model is a matter of debate, as it would depend not only on its accuracy, but also on the ease of calibration and the ease of calculation for a design seismic coefficient. The review of damage measures shows that there are complex indices, which will theoretically account for a variety of duration effects. However, these indices can be difficult to fit, and fitted models are likely to be quite specific in application.

A critical requirement of a damage model is that it should provide the same level of system reliability as that implied by the structural ductility factor for the standard cyclic loading procedure. That is, a system with a specific ductility factor, μ_f, should have the same probability of reaching the 'failure' state defined by the damage model after four cycles to ductility μ. If damage models achieve this then the level of system reliability will be consistent with that implied in the current loadings standard, and the structural reliability should have some independence from the damage model selected.

The fitting of a damage model is simplified if it can be directly related to the structural ductility factor. In this case the required structure design strength can be calculated from the ductility factors provided in the material standards. An example of such a model would be hysteretic energy. The hysteretic energy dissipated by a structure for a specified structural ductility factor, is equal to the area enclosed by the force-displacement history for the standard cyclic loading procedure.

The method for evaluating damage model seismic coefficients would be dependant on the model form. If the damage model can be related to peak ductility demand, then the peak ductility demand can be calculated directly and the existing design spectra can be used to determine the damage based seismic coefficients. As well as simplifying the process, this ensures a level of consistency for different structural types. If a damage model cannot be related to peak ductility, then it would be necessary to effectively produce new spectra as a function of the damage model parameters. This may allow for the use of more rigorous damage measures, but would be an involved process.

4.2.2 Proposed Damage Model

It is proposed that seismic damage be measured as a function of cumulative demand and peak demand, using a damage index similar to that developed by Park and Ang [39] Eq. 4.19:

$$D = \frac{\beta}{F_y.\delta_u}\int dE + \frac{\delta_m}{\delta_u} : D \leq 1$$

This damage index allows for the consideration of both peak response and duration effects. Damage indices of this type have been used extensively so data is available on their application and reliability. The additional advantage of this damage measure is that seismic coefficients may be determined using current code spectra.

Coefficients of the damage model may be determined from existing recommendations or may be evaluated from test data. The structural ductility factor may be used as a calibration point for the damage model. This would ensure a consistent definition of failure. The system seismic hysteresis energy may be obtained from the existing seismic hazard spectra.

The seismic demand on the system may be determined by varying the peak displacement ductility until the damage index reaches the failure threshold, or equivalently, when the effective ductility reaches the system structural ductility factor. This gives the 'actual' peak ductility capacity as defined in Eq. 4.21.

$$\mu_m' = \mu_m.\left[\frac{\beta.\delta_y}{F_y.\mu_m}\int dE + 1\right]$$

The system seismic demand coefficient may be obtained from the loadings standard hazard spectra using this calculated ductility capacity. This is then compared with the seismic coefficient obtained with the structural ductility factor to calculate the S_p factor.

An important factor in the use of the proposed damage model is the implied relationship between the structural ductility factor as defined by the Loadings Standard, and as defined by the standard test procedure. The ductility test procedure effectively determines the ductility factor by equating the cumulative ductility of a test specimen with that given by the Loadings Standard definition. As a result of this the peak ductility is typically greater than the ductility factor, while the hysteretic energy with the test procedure is higher for ductilities below three, and lower for ductilities above three. The two structural ductility factor procedures therefore imply **different_damage levels** when using the combined cumulative and peak demand damage index. This is likely to be a result of shortcomings in both the damage index theory and the ductility factor evaluation procedure. The difference between the two ductility procedures is reduced if the damage index measures cumulative ductility rather than hysteretic energy. Altering the test procedure so that hysteretic energy was equated would have a similar effect.

4.2.3 Alternative Damage Model

The use of a weighted cumulative demand damage index would result in a more rigorous procedure. It would also provide a means of making the standard ductility test procedure more compatible with the Loadings Standard definition. The use of a cumulative damage index does however, complicate the S_p factor evaluation procedure and may have accuracy problems as discussed earlier.

The damage model could be fitted from experimental data. Suitable calibration points might be the failure ductilities for one half cycle (peak displacement ductility) and four cycles (structural ductility factor. If the damage index is calibrated to the structural ductility factor, μ_f, defined by four cycles to displacement ductility, μ_d, then:

Given Eq. 4.11:

$$D = k.\sum_i N_i.\mu_i^n$$

calibrating with the structural ductility factor gives:

$$1 = k.8.\mu_d^n \Rightarrow k = \frac{1}{8\mu_d^n} = \frac{1}{8\mu_f^n} \qquad \textbf{Eq. 4.22}$$

therefore:

$$D = \frac{1}{8\mu_f^n}.\sum_i N_i.\mu_i^n \qquad \textbf{Eq. 4.23}$$

From this definition of damage the structural ductility demand of a system may be evaluated. The ductility demand is given by the minimum required structural ductility factor. That is:

minimise μ_f: $\mu_f^n = \frac{1}{8D}.\sum_i N_i.\mu_i^n ; 0 \leq D \leq 1$

Therefore μ_f is given by,

$$\begin{aligned} \mu_f &= \left[\frac{1}{8.1}.\sum_i N_i.\mu_i^n\right]^{-n} \\ &= \left[\frac{\sum_i N_i.\mu_i^n}{8}\right]^{-n} \end{aligned} \qquad \textbf{Eq. 4.24}$$

By comparison, the standard test method gives the structural ductility factor for the specified loading scheme as:

$$\mu_f = \frac{\sum_i N_i.\mu_i}{8} \qquad \textbf{Eq. 4.25}$$

that is, the ductility is not weighted.

Theory and test data indicates that a simple cumulative ductility model such as this is not realistic. Simple curve fitting exercises of test data indicate that a second power ductility model could provide a reasonable estimate of damage and could be used to evaluate effective ductility factors.

5.0 EVALUATION OF S_P FACTOR FOR STRUCTURE SYSTEM RELIABILITY AND REDUNDANCY

A structural performance factor could be applied to account for the varying levels of reliability and redundancy of structural systems, and it would reflect the relative reliable strengths and/or ductilities of the structural systems.

5.1 EVALUATION PROCEDURE

5.1.1 Reliability

To evaluate reliability structural performance factors a reference structural system, or systems, must be specified. A reliability structural performance factor of 1.0 would be assigned to these reference systems. A logical reference system would be a simple form of a common structural system. For example a single bay, single storey frame. With this the reliability of the structure would be close to that of the individual components. The suitability of such a system should however, be reviewed. In particular, there is a need to consider whether the current material standards imply some degree of structural system redundancy, or reliability, in their specification of structural ductility factors and strength reduction factors.

Reliability structural performance factors of specific structural forms can be evaluated by calculating the reliability of sample systems relative to the reference systems. The reliability of the sample structural systems would be evaluated through the use of event trees and reliability theory. The reliability of the systems would be a function of the statistical distributions of the material and member parameters. This information might be sourced from the relevant materials standards and IPENZ study groups. The process is somewhat simplified as it involves the calculation of relative rather than absolute reliability.

5.1.2 Redundancy

In the pure sense structural redundancy could be defined as the degree to which parts of a structural system may be removed (or fail) with out a reduction in the design capacity of the structural system. This form of pure redundancy would be rare in real structures. It is possible that it may occur for structures designed elastically. In this case the design capacity would be controlled by the first yield of one of the members. If this member failed the load would be redistributed and the design capacity would then be controlled by the first yield of one of the remaining members. It is possible that this capacity would equal or even exceed the original design capacity (provided the failure of the member did not result in an unacceptable reduction in the ability of the structure to carry gravity loads).

For ductile structures it is likely that a substantial portion of the structural members will have reached deformations corresponding to their design strength. Therefore the failure of a member is likely result in some reduction in total capacity, rather than just a redistribution of structural actions. The reduction in capacity may however, be proportionally small. As structures are actually required to sustain earthquake accelerations rather than forces, the reduction in capacity may not significantly affect the ability of the structure to withstand the design earthquake event.

As discussed in section 3.5, structural system reliability and redundancy are closely related. Both factors are a consequence of the existence of parallel (multiple) load paths in a structural system. Aspects of structural redundancy are therefore dealt with implicitly in reliability calculations. It is however, possible to account for structural redundancy explicitly in reliability calculations by considering both changes in strength and ductility. One method for achieving this for a sample structure is specified below:

1. Perform a push-over analysis of the sample structure. Structural displacement controlled by limits on the ductility of members as specified by the material codes (these ductility limits should account for cyclic effects). Calculate the strength and ductility of the structure at the maximum displacement. Evaluate the design standard seismic demand on the system for the calculated ductility. Determine the factor of safety of the calculated strength over the design standard demand.

2. Remove the structural members, which have reached their ductility limit, and therefore control the displacement limit of the structure. Continue the push-over analysis until any of the remaining members reach their ductility limit and calculate revised values of structure strength and ductility. Calculate the design standard seismic demand corresponding to the new calculated ductility, and the resulting factor of safety.

3. Repeat step (2) until one of the following occurs:

    a) Limiting members cannot be removed without an unacceptable reduction in ability to carry gravity load.

    b) A limiting displacement is reached

    c) A limiting lateral strength degradation is reached

4. Calculate the relative redundancy reliability given by the ratio of the maximum factor of safety from step (2) to the factor of safety from step (1)

The above process could be revised so that reliability rather than the factor of safety is evaluated at each step. The reliability of the system, accounting for redundancy effects, is then given by the maximum reliability from steps (1) and (2).

CONCLUSIONS

A structural performance factor (S_p) was introduced into the 1992 New Zealand Loadings Standard. A single value of 0.67 was used. The Commentary to this code gave a number of reasons why the use of this factor, which reduced the required seismic strengths, was justified. In this study the literature and reasoning behind the introduction of this factor has been examined. It is concluded that many of the reasons that were quoted as justifying the introduction of the Structural Performance factor are already allowed for in the design process, and consequently they should not be used to justify the use of the S_p factor. However, there are two issues, as described below, which can be used to justify the S_p factor.

1. The response spectrum, which is used to assess seismic design actions, predicts the peak displacement that is attained once during the design level earthquake. However, methods of assessing ductility levels for sub-assemblies require that the structural elements sustain the plus and minus displacement corresponding to the design ductility level for several cycles. As damage accumulates with inelastic displacements an approach, which considers a single peak displacement, is bound to be conservative.

2. Multiple load paths exist in indeterminate structures. In such structures a loss of load carrying capacity in one or more elements does not necessarily lead to failure as redistribution of structural actions can occur. However, this redistribution cannot occur in statically determinate structures. Clearly the extent of redistribution that can occur depends on the level of redundancy. This redundancy increases the robustness of the structure and its reliability of the structure. Allowance for this aspect of behaviour could be incorporated into an S_p factor or a redundancy factor as occurs in a number of other cods of practice.

The review of the literature included an assessment of how a number of other codes of practice allowed for the equivalent of an S_p factor. The most comprehensive treatment is contained in the IBC 2000 code. In this, the equivalent of the S_p factor is given for a wide range of structural forms and materials, with values ranging from 0.5 to 1.0. This code also includes a factor, which allows for the beneficial effects of redundancy of the structure.

A method of developing rational S_p factors is proposed by assessing damage levels using time history analyses and damage indices. It is concluded that a single factor for use with all limit states and design ductility levels cannot be justified. The value that is used should be specified in the materials code taking into account the ductility level together with the reliability and robustness of the structure.

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For further information contact:
Department of Civil and Resource Engineering, School of Engineering
The University of Auckland, Private Bag 92019, Auckland, New Zealand
Telephone: 64-9-373 7599 ext. 8166 or 5715 Facsimile: 64-9-373 7462