# RECOMMENDATIONS FOR THE SEISMIC DESIGN OF PETROCHEMICAL PLANTS

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SYNOPSIS:

Difficulties in applying the seismic design provisions of the NZ Standard Code of Practice for General Structural Design and Design Loadings for Buildings, NZS 4203:1976, to multi-component petrochemical facilities lead to the preparation of the document: Seismic Design of Petrochemical Plants - Volume 1 : Recommendations and Volume 2 : Commentary.

This paper explains the basis of that document. The philosophy that is used provides for a consistent level of earthquake protection to the various components of a petrochemical facility according to the importance of the component and the potential hazard associated with its failure. This is achieved by establishing design load levels based on assessed seismic risk and structure reliability. An important part of the philosophy is the minimising of seismic risk by the elimination or modification of potentially hazardous situations at the conceptual design stage.

The derivation of basic seismic design coefficients from a seismicity study of New Zealand is described and the analysis and detailing procedures adopted in the Recommendations for structures and equipment in petrochemical facilities are discussed. Particular attention is given to explaining the application of capacity design principles to ductile structural forms.

## 1. INTRODUCTION:

### 1.1 Background

As New Zealand embarked on the major energy developments in 1980, there was concern that the seismic design provisions of the New Zealand Standard Code of Practice for General Structural Design and Design Loadings for Buildings, NZS 4203:1976[1], would not be suitable for the design of major petrochemical facilities. In the first instance NZS 4203 was written for the general run of commercial and industrial buildings and contains no specific guidance for the specialised structural and plant components associated with petrochemical facilities. This is acknowledged in clauses 1.1.1 and C1.1 of NZS 4203 where special industrial equipment is excluded from its detail provisions and a special study is required. In the second instance the concepts of "risk" and "importance" contained in NZS 4203 for individual buildings can not be readily related to multi-component facilities.

In determining an appropriate level of earthquake protection for these facilities differing interpretations and applications of NZS 4203 are possible and problems were anticipated between the client, the designer and the administering authorities.

In June 1980 the Ministry of Energy commissioned the preparation of a document for guidance on the seismic design of petrochemical plants. This was carried out by a small team from the Ministry of Works and Development with the assistance of A.G. Gillies from En-Consult Technology Ltd and resulted in the publication in March 1981 of Seismic Design of Petrochemical Plants - Volume 1 : Recommendations and Volume 2: Commentary[2].

A "draft for comment" was issued in October 1980 to a wide range of engineers. Acting on the comments received, the March 1981 issue was completed.

The urgent need for a working document meant the rigorous but time-consuming "committee" approach was not appropriate.

The purpose of this paper is to explain the basis of the Recommendations, to provide for their discussion and to assist in their interpretation and application. In addition it has become clear that, particularly outside New Zealand, the concepts of capacity design included in the Recommendations are not well understood. Some effort has therefore been made in the paper to explain the capacity design procedures which were solidly founded in NZS 4203.

### 1.2 Approach

Early consideration was given to preparing a supplement to Part 3. Earthquake Provisions of NZS 4203, to provide an interpretation and additional provisions suitable for petrochemical facilities. However, NZS 4203 in covering general structures, is already a complex document to use. It was considered its expansion to cover the special conditions, structures and plant components of petrochemical facilities would have been unwieldy and complicated.

It was therefore decided to prepare a complete and specific set of recommendations which could be used in place of Part 3 of NZS 4203. This allowed recent seismic design developments to be included in the

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Recommendations and enabled the development of a logical and consistent approach to the seismic protection of components and the plant as a whole in place of the single building concepts of "risk" and "importance" contained in NZS 4203.

The means of establishing the seismic design load levels is therefore significantly different from that used in NZS 4203. A concept of assessed seismic risk and structure reliability modifying an elastic response design spectrum is used in place of the factored design spectrum of NZS 4203.

The application of the design loads and the detailing requirements generally follow the approach of NZS 4203. Specific guidance is provided for structural and plant components commonly found in petrochemical plants.

On one other significant area the Recommendations depart from the provisions of NZS 4203. In clause 3.2.1, NZS 4203 requires that all buildings, with minor exceptions, be designed to possess ductility. The Recommendations allow any structure to be designed to the elastic response seismic loading provided it does not contain equipment with a higher seismic risk classification than that of the structure itself. Structures so designed are not required to be specifically detailed for ductility. However, in recognition of the possibility of the design earthquake being exceeded, the designer is nevertheless required to ensure that in the overload condition the structure will fail in a controlled manner. By this is meant that the designer must identify the failure mechanism and be satisfied that the identified critical components will not fail in a brittle manner.

Elastic design of structures with the above provisos, has been allowed because for many prefabricated units, the shipping loads for which they are designed exceed the elastic design earthquake loads. Requiring special seismic detailing in addition to such high strength was not considered reasonable.

However, it is stressed that the intention throughout the Recommendations has been to promote the use of structures designed and detailed for ductility. No matter what design earthquake is chosen there is always the possibility of the occurrence of a more severe event. The ability of such structures to survive the disaster level earthquake without catastrophic collapse is without question better than an "elastically" designed structure designed for substantially larger loads. That is, ductile structures provide a better quality of seismic protection. In addition, particularly for major structures, the use of the smaller seismic coefficients appropriate for a ductile structure result in a lighter and therefore cheaper structures.

### 1.3 Status

The Recommendations represent the results of a special study of the seismic design requirements for petrochemical plants in New Zealand. They are recommended by the Ministry of Works and Development as a means of compliance with the requirements of clause 1.1.1.2 of NZS 4203.

The client and/or designer will need to establish acceptance of the Recommendations for their particular application from the administering authorities.

While the Recommendations have been specifically written for petrochemical plants, they are equally applicable in principle to other multi-component industrial facilities.

At this stage the Recommendations have been formally adopted for the NZ Synthetic Petrol Plant by the Ministry of Energy (for the client) and the Marine Division of the Ministry of Transport and the Clifton County Council (as administering authorities). It is likely they will also be adopted for the New Zealand Steel expansions at Glenbrook.

## 2. PHILOSOPHY:

### 2.1 General

The following objectives form the basis of the design philosophy for the Recommendations:

(a) protection of life of the public in the region of the plant;

(b) protection of life of the plant operators;

(c) protection of the environment;

(d) minimising disruption to the operation of the plant following a moderate earthquake, i.e. an earthquake with a return period 1-2 times the design life of the plant;

(e) control of damage to critical components under a major earthquake, i.e. an earthquake with a return period 4-6 times the design life of the plant;

(f) safe shutdown of the plant to a passive state under the maximum probably event, i.e. an earthquake with a return period of the order of 1000 years.

To meet these objectives the principle has been adopted of achieving a consistent level of seismic protection throughout the plant by providing a hierarchy of strengths between the various structural components. This is achieved by choosing an earthquake return period separately for the design of each component according to its importance to the plant as a whole and to the risk associated with its failure.

For each seismic zone in New Zealand, an elastic response spectrum was derived for a basic design return period earthquake.

The design lateral load for each component is obtained by modifying the appropriate elastic response spectrum, firstly for the chosen earthquake return period, and secondly for the capability of the component to dissipate seismic energy by post elastic deformation without significant loss of strength, i.e. its ductility capability.

This procedure for determining the design lateral load is based on the work undertaken by Berrill, Priestley and Chapman[3] for the Bridge Design Study Group of the NZ National Society for Earthquake Engineering. The approach is also in line with recent overseas codes[4].

The basic elastic response spectra for the regional zones were derived from a detailed seismicity study of New Zealand following the principles set out by Jury et al[5].

This approach provides the designer with an appreciation of the likelihood of the design force levels being exceeded during the life of the structure and provides a level of protection throughout the plant consistent with the importance or potential hazard associated with each component.

## 2.2 Plant Design Principles

Petrochemical plants are generally expensive, one-off facilities and the consequences of serious disruption from earthquakes may be of national importance. There are many areas where seismic risk can be minimised by eliminating or modifying potentially hazardous situations at the conceptual design stage - often at little or no additional cost. Accordingly the Recommendations require that the overall plant layout be considered, at an early stage of design, from the point of view of minimising possible damage from earthquake attack.

It is stressed that overall plant layout is an important aspect of seismic design philosophy. It has been found this has not previously been part of the conceptual stage of plant design. Traditionally this has been the province of the process engineers. Some adjustment of the traditional preliminary design team will be required to include the services of a seismic engineer. It is not intended that seismic considerations dominate the design process, but simply that where opportunities exist for reducing seismic risk they should be taken.

In this respect the Recommendations contain particular requirements for:

(a) the geotechnical assessment of the proposed site for the facility;

(b) the consideration of the overall plant layout;

(c) the selection of structural forms appropriate for the dispersion of earthquake induced forces;

(d) the location and fixing of mechanical equipment in the structures;

(e) the protection of control systems.

In considering the overall plant layout, each component is assigned a seismic classification according to its importance and the risk associated with its failure. In assessing the classification, both the functional operation and the relative location of the component within the plant are to be considered. The effect of this is to identify potentially hazardous situations which are assigned a higher risk classification, unless the hazard can be reduced. In the Recommendations the seismic classification is related directly to the earthquake return period to be used for the design of the component. A consistent level of seismic protection for the various plant components is thereby achieved.

## 2.3 Design Life and Design Earthquake

A design life of 25 years for petrochemical facilities has been taken as the basis of the Recommendations. The basic design earthquake has been chosen as the seismic event with a 150-year return period. This is the return period associated with the objective (e) in Section 2.1 to control damage to critical components under a major earthquake. Under such an earthquake, many plant components are expected to sustain significant damage. Brittle structures designed to this earthquake will be at the point of incipient failure while a structure designed for ductile post yield behaviour will have been subjected to significant excursions of plastic deformation. There is an approximately 15% probability that the plant will be subjected to an earthquake of greater magnitude within the 25-year design life.

It is therefore important that structural detailing should be such that the damage occurs in a controlled manner and that damaged components should be able to be repaired and returned to service within a short period of time.

For important or high risk components, a longer design return period is appropriate to reduce the extent of the damage during the 150-year earthquake and to reduce the probability of the design force levels being exceeded within the life of the structure.

Some consideration was given to using a two-tier approach of designing for an "operating" level and a "disaster" level earthquake. However, it was concluded that this would unnecessarily complicate the design process. Designing for the basic 150-year event meets the objective of minimising damage under more frequent earthquakes and catastrophic collapse of a structure under a more severe earthquake will be avoided provided the structure has been adequately detailed. The analysis and detailing requirements contained in the Recommendations are aimed at achieving this state.

It is noted the basic 150-year return period corresponds approximately with the level of earthquake implied by the basic design levels of NZS 4203[1].

## 2.4 Analysis Procedures and Detailing Requirements

With some modifications the equivalent

static force method of analysis and the detailing procedures of NZS 4203 are adopted. Specific guidance on ductility capability and detailing requirements for typical structures is contained in the Recommendations.

## 2.5 Seismic Performance Qualification

In order to achieve a consistent level of protection for the structural components of the plant and the equipment they support, the Recommendations give requirements for the seismic performance qualification of all equipment and control systems. It is the responsibility of the designer to specify the level of qualification required for each item of equipment and to ensure that the purchased item meets the requirements.

## 3. SEISMIC STUDY, BASIC DESIGN CURVES, RETURN PERIODS

### 3.1 General

Quantifying seismic risk in terms of earthquake recurrence intervals is an established way of expressing seismic design criteria in a code format[3,4,6]. Statistical techiques developed over the past decade permit the prediction of spectral accelerations for specified return period based on historical earthquake records and geological data. A detailed study carried out over a region provides the basis for establishing the two prime components of a seismic design code. One is the definition of seismic zone boundaries for the region and the other is the derivation of the elastic spectral acceleration curve for each defined zone.

### 3.2 Seismicity Study

The detailed study of seismic risk in New Zealand was carried out with the assistance of the Earthquake Engineering Group, Beca Carter Hollings and Ferner Ltd. From this study a set of contour maps for spectral acceleration variations over New Zealand was generated for the chosen 150-year return period event.

Two key periods were established, considered to represent the junction points for structures categorised as 'short period' and 'long period'. Two types of subsoil were defined, classified as 'hard' and 'soft' ground, to provide for the influence of subsoil flexibility at a given site.

The key periods were defined as:

Hard Ground:

(i) Short period: $T = 0.25$ sec
(for $T < 0.25$ sec)

(ii) Long period: $T = 2.0$ sec
(for $T > 2.0$ sec)

Soft Ground:

(iii) Short period: $T = 0.5$ sec
(for $T < 0.5$ sec)

(iv) Long period: $T = 2.0$ sec
(for $T > 2.0$ sec)

Four contour maps representative of the conditions (i) to (iv) above were derived for the 150-year return period spectral accelerations. These maps represented smoothed acceleration contours with discrete peaks being ignored. The contour maps are given in Figures A(i) to A(iv).

### 3.3 Definition of Seismic Zone Boundaries

The contour intervals and orientation of the spectral acceleration maps provide a basis for the definition of seismic zone boundaries. Using the three zone principle, the results compared well with the zone boundaries of NZS 4203 and only slight changes were adopted similar to those presented by Berrill, Priestley and Chapman[3]. The zone map is given in Figure B.

One important deviation from NZS 4203 was the decision to adopt a graduated subdivision of zone B as proposed by Berrill et al[3]. The primary reason for this was to avoid anomalies near the zone boundaries where, over a short geographic distance, a sharp change of seismicity could be implied. The graduations represent a uniform gradation between zones A and C as much as they represent results from the contour maps of the seismicity study. It is incorrect to conclude that the variation of seismicity in Zone B has been derived with the precision implied by the intervals given in the zone map.

### 3.4 Derivation of Basic Horizontal Seismic Coefficients

From the spectral acceleration levels on the contour maps, plots for the basic elastic horizontal seismic coefficients were derived. These are the $\mu = 1$ curves for zones A and C given in Figure C. For zone B the seismic coefficient is given by $\beta$ times the zone A curve.

Representative spectral accelerations for zones A and C were determined for the key period intervals ($T = 0.4$ secs and $T = 2.0$ secs). The design elastic spectrum was derived by connecting the two points with straight lines. For very long period structures ($T > 2.0$ secs) a further drop off in intensity was justified on the basis of observed correlation with spectra of recorded earthquakes. For very short period structures ($T < 0.4$ sec) the plateau at the peak acceleration level has been maintained as in NZS 4203.

Some averaging of the predictions for spectral accelerations is reflected in the design curves recommended for each zone. Depending on the site location within zone A for example, the design coefficient may slightly overestimate the spectral accelerations for stiff (short period) structures supported on hard ground. Conversely, for similar structures located on soft ground, the design event accelerations may be underestimated. A site specific seismicity study is recommended where local site subsoils may be classed as particularly soft. For zone C, the design spectrum has been raised above the level implied by the seismicity study, in part to match more closely with

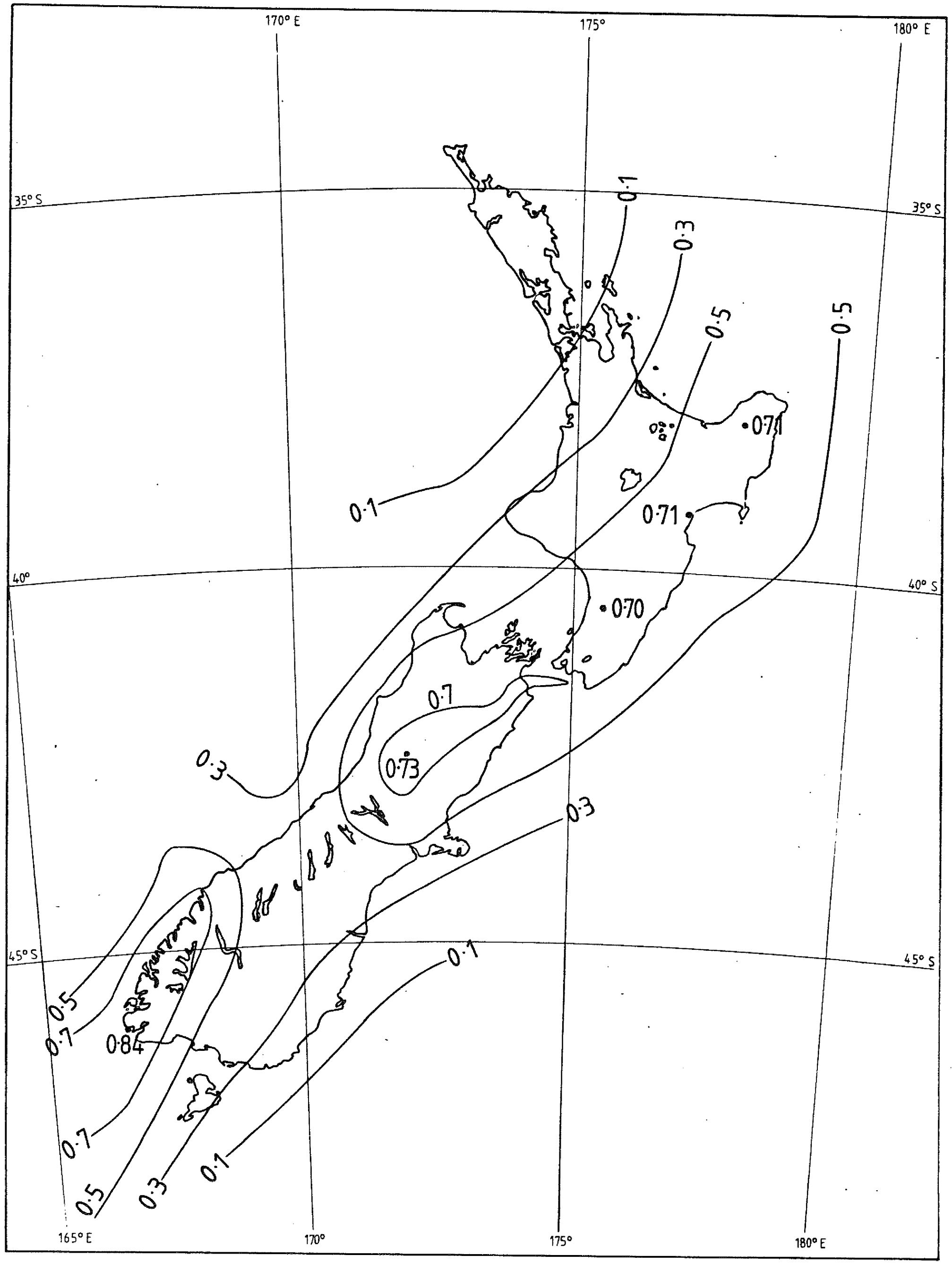


Figure A(i) HARD GROUND, SHORT PERIOD, 150 YEAR RETURN EARTHQUAKE.

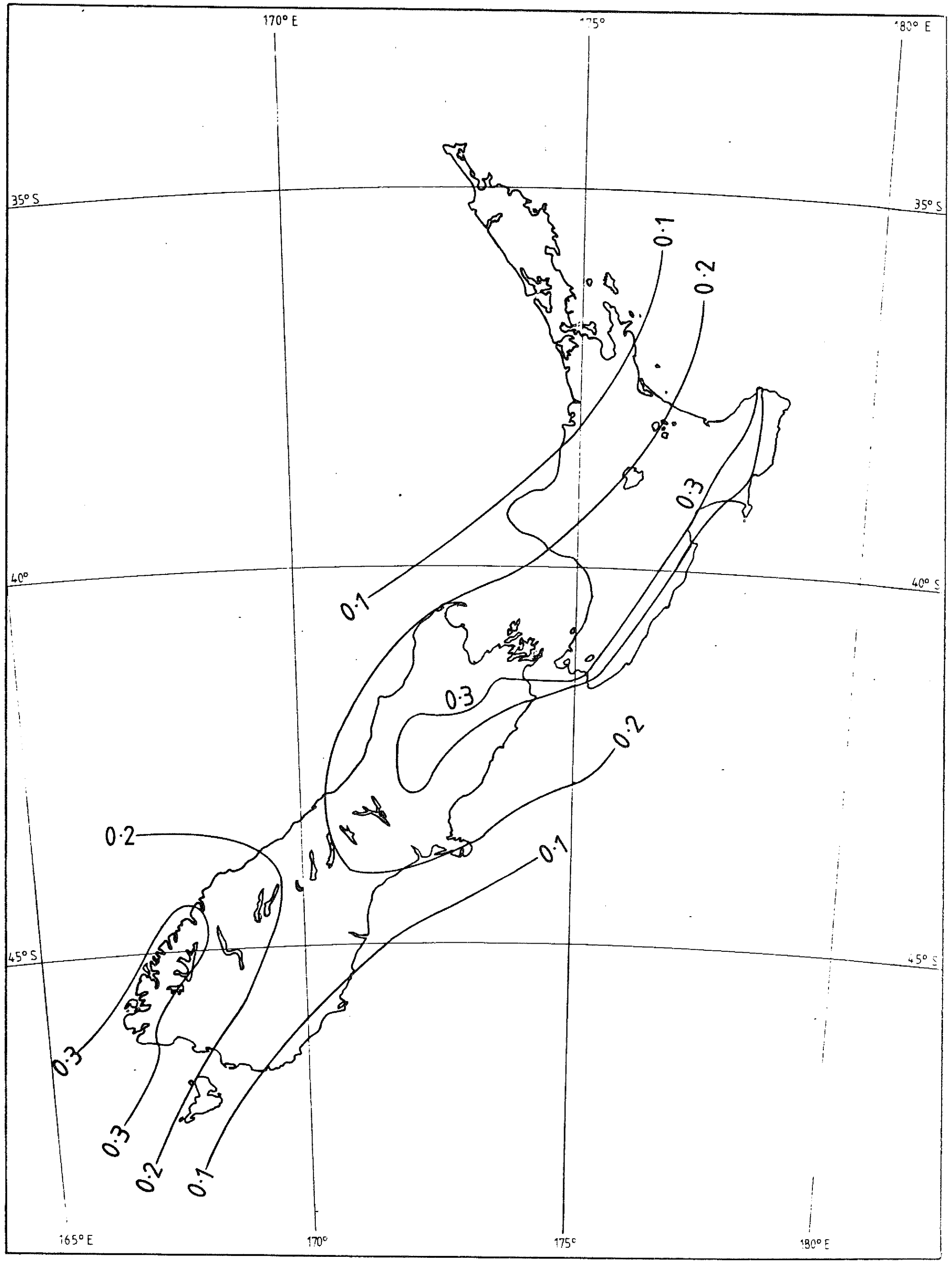


Figure A(ii) HARD GROUND, LONG PERIOD, 150 YEAR RETURN EARTHQUAKE.

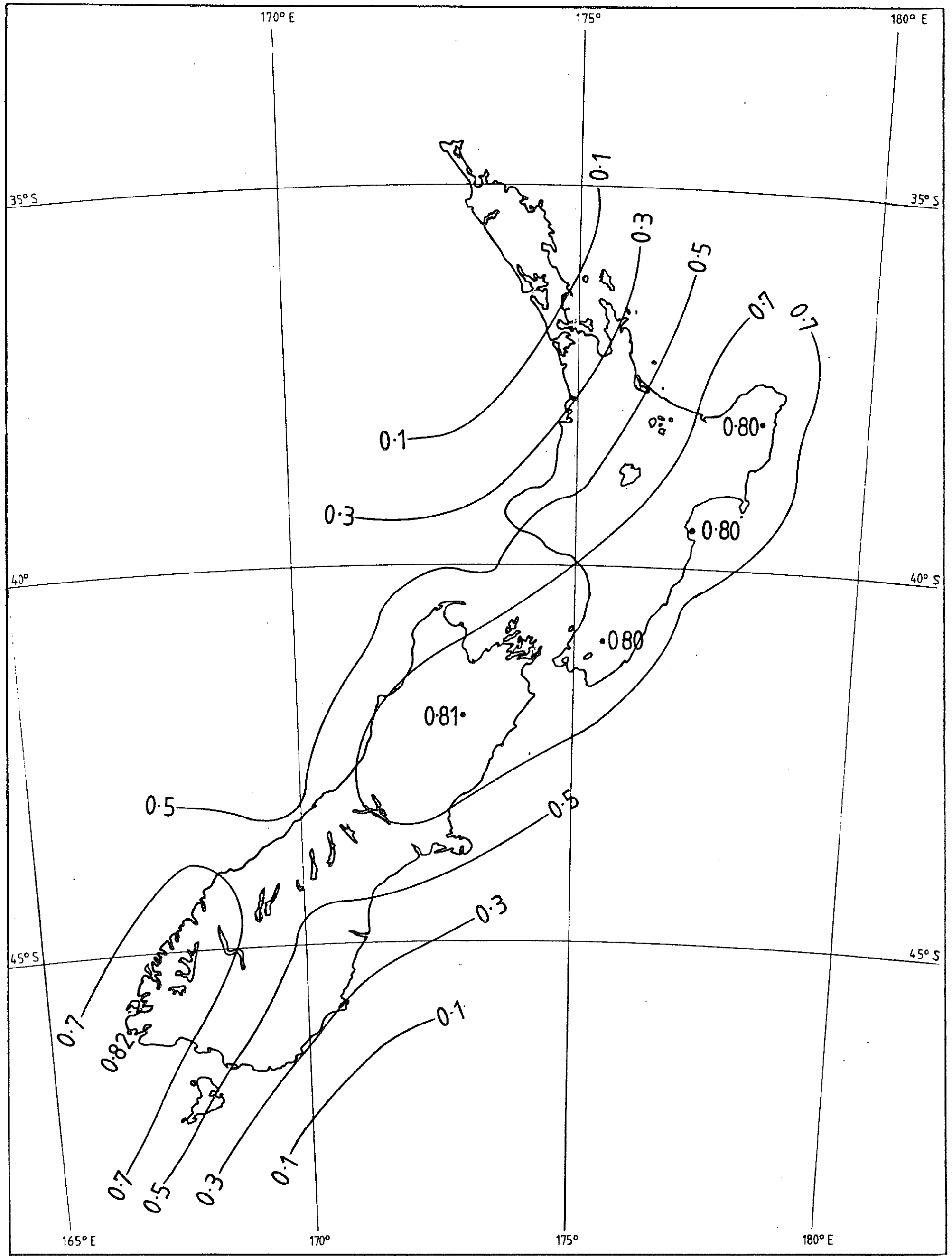


Figure A(iii) SOFT GROUND, SHORT PERIOD, 150 YEAR RETURN EARTHQUAKE

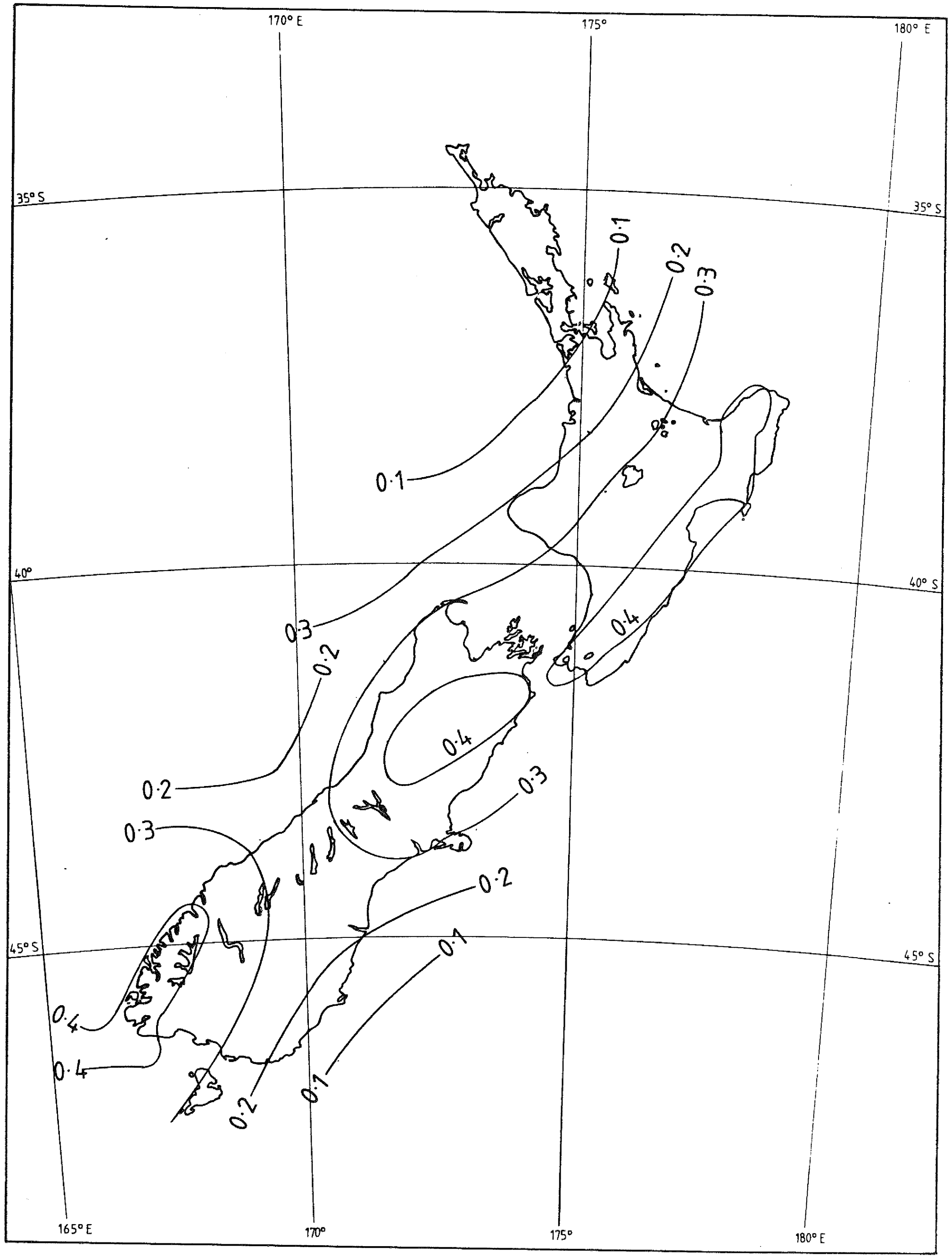


Figure A(iv) SOFT GROUND, LONG PERIOD, 150 YEAR RETURN EARTHQUAKE.

NZS 4203 but primarily to recognise the uncertainty about the accuracy of the attenuation expressions used to assess the effect of distance earthquakes.

### 3.5 Modification of the Elastic Seismic Coefficients for Ductility

In NZS 4203, the structure type factor, S, takes into account the differing earthquake performance of various structural systems. In an effort to better acquaint the designer with the seismic performance characteristics of common petrochemical plant structures, the Recommendations relate the structural system to a ductility capability, $\mu$. In this context, $\mu$ is the structure displacement ductility defined as the ratio of the displacement at the top of the structure than can be sustained under inelastic cyclic loading without significant degradation of strength to the corresponding displacement at yield.

Structures are assessed for their ability to achieve a $\mu$ factor of 2, 3 or 4. This is similar to the approach taken in other standards and recommendations.[3,4]

To derive the design seismic coefficients for structures with ductility capabilities of $\mu$ = 2, 3 or 4 the elastic coefficient curves have been scaled using the "equal energy" theorem for short period structures (T < 0.4 secs) giving:

$$C_{H\mu} = \frac{C_{HE}}{\sqrt{2\mu - 1}} \qquad (1)$$

and the " equal displacement" theorem for long period structures (T > 2.0 secs) giving:

$$C_{H\mu} = \frac{C_{HE}}{\mu} \qquad (2)$$

The basic seismic coefficients for $\mu$ = 2, 3 and 4 are given in Figure C. A minimum basic coefficient for any structure has been arbitrarily set at 0.05, as shown for zone C, to ensure that all structures are designed for at least a minimal earthquake loading.

### 3.6 Modification of the Basic Seismic Coefficients for Return Period

The design return period for each structure is assessed taking into account its importance to the plant as a whole and the safety hazard associated with its failure. Longer return periods imply greater design loads and a smaller probability of the design load being exceeded within the life of the structure.

The scaling factors, $Z_H$, to modify the basic (150-year return period) seismic coefficient for the chosen design return period has been taken from the work of Peek[7] and are appropriate for New Zealand seismic conditions. A plot of $Z_H$ against return period, as given in Table 2.1 of the Recommendations, is presented in Figure D.

Assuming a Poisson distribution for the occurrence of earthquake events, the relationship between design life, t, earthquake return period, $t_s$ and the probability, p, of the earthquake being exceeded during the design life, can be approximated by:

$$t_s = \frac{1}{1 - (1-p)^{1/t}} \qquad (3)$$

A plot of this relationship is given in Figure E.

From this figure it can be seen that for a 25-year design life, the probability of the 150-year return period being exceeded is approximately 0.15. For a very important and high risk structure, an acceptable probability of exceedance may be 0.025. The design return period then becomes 1000 years and from Figure D, $Z_H$ is 1.5.

Alternatively, for a 50-year design life, to achieve the basic level protection of the 0.15 probability of exceedance, a design return period of 300 would be required with an associated $Z_H$ factor of 1.15.

## 4. GENERAL ANALYSIS PROCEDURES

### 4.1 General

The detailed requirements are given in full in the Recommendations and Commentary. Generally they follow the equivalent static lateral force method of analysis of NZS 4203. Only the major components of interest are covered in this paper.

It is noted the Recommendations cover only seismic design. Design for other loadings (gravity, wind etc) is to be to the requirements of NZS 4203.

### 4.2 Seismic Load Combinations

The design load combinations follow the prescription of NZS 4203 and allow both the strength method and the alternative (working stress) methods of design:

Either strength method

$$U = 1.0D + 1.3\,L_o + E$$

$$U = 0.9D + E$$

For 'ductile' structures (those designed for inelastic deformation of yielding elements) these load combinations prescribe the required dependable strength ($\phi < 1$) for the yielding elements. The required strength for the non-yielding elements is subsequently assessed using capacity design principles, as discussed in Section 5, to ensure they are strong enough to remain elastic under the maximum likely forces resulting from simultaneous yielding of the yielding components.

For 'elastic' structures (intended to remain elastic under the design seismic load) these loadings prescribe the required dependable strength for all elements in the structure.

Or alternative method

$A = D + L_o + 0.8E$

$A = 0.7D + 0.8E$

The structures are designed to resist the imposed loading without exceeding the member allowable stresses. The alternative method is suitable for use in the design of 'elastic' structures only

where $D$ = dead load

$L_o$ = live load under normal operating conditions

$E$ = earthquake load

Where appropriate, $L_o$, the live load under normal operating conditions includes temperature effects and internal pressures as well as the weight of vessel contents and platform live loads. Where live loads are of short duration only, e.g. maintenance or test loading, they need not be considered as part of $L_o$ or be used in assessing E in the above equations.

For consistency between the vertical gravity load and the horizontal earthquake loading on containment vessels it is recommended that the dead load component of the second of the equations in either the strength or alternative method load combinations should include both vessel weight and weight of contents under normal operating conditions. If the vessel is empty for a significant proportion of the time, the second equation with the earthquake load E based on the dead weight of the vessel only, should also be considered.

The imposed forces in the members of the structure are determined by an elastic analysis of the structure under the prescribed loading combinations. For structures with at least one axis of symmetry, the seismic loads are taken to act non-concurrently in the direction of the principal axes. For other structural forms, the most severe loading directions are established and the design carried out assuming non-concurrent loading in these directions.

### 4.3 Horizontal Seismic Base Shear

The earthquake loading for each structure (except for the design of in-structure equipment - see Section 6) is based on a horizontal seismic force (base shear) given by:

$$V = C_{H\mu}\ Z_H\ W \qquad (4)$$

where

$C_{H\mu}$ = basic horizontal seismic coefficient

$Z_H$ = design earthquake return period coefficient

$W$ = total structure weight $(D+L_o)$ above ground.

### 4.4 Basic Horizontal Seismic Coefficient, $C_{H\mu}$

The ductility capability, $\mu$, is assessed for the structure from Table A.

For the fundamental period of the structure and the geographic location $C_{H\mu}$, is obtained from Figures B and C as discussed in Section 3.4.

The use of ductility capability factors, $\mu$, greater than those listed in Table A is permitted only where they can be substantiated by experimental evidence. The use of $\mu>4$ is not permitted to ensure that damage to the structure does not occur during smaller, more frequent seismic events.

For the structural types listed in Table A, the $\mu$ factors were assessed for the structure's ability to maintain strength and dissipate seismic energy under cyclic deformations beyond the yield point. The values generally reflect the SM combination for structural types in NZS 4203.

### 4.5 Return Period Coefficient, $Z_H$

The design return period for each structure is determined from the seismic classification given in Table B. It is expected that most structures would fall into the C and D classification. The A classification is expected to apply only to critical control components.

In choosing a classification for a component of the plant, consideration should be given to the nature and consequences of its failure. For example where the failure of a component would result in its collapse or rupture with the consequent release of quantities of hazardous material, it would warrant a higher classification than if the failure represented a buckling or even severe distortion of the component without complete collapse or rupture; i.e. the consequences of such a failure would be less.

### 4.6 Distribution of Horizontal Seismic Force

For regular structures the horizontal earthquake design forces are obtained by distributing the total base shear, V, over the height of the structure according to the equation:

$$F_x = V \frac{W_x h_x^{\ k}}{\Sigma W_x h_x^{\ k}} \qquad (5)$$

where

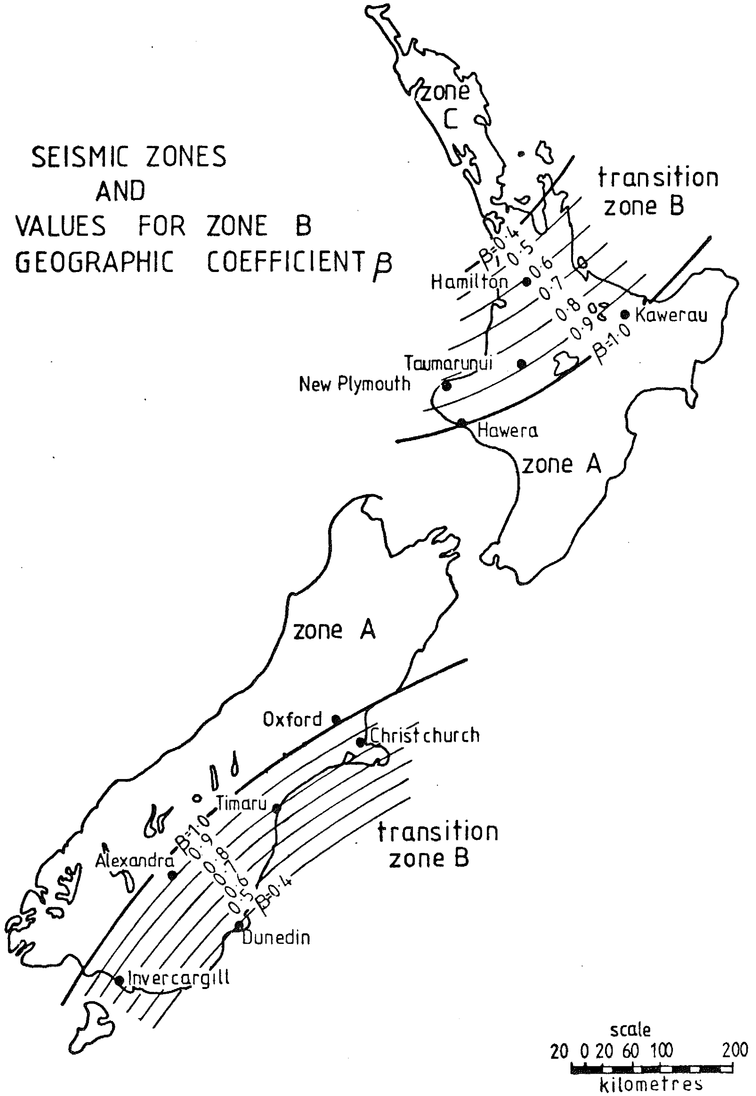
$W_x$ = total operating weight $(D + L_o)$ at level x

$h_x$ = height of level x

and

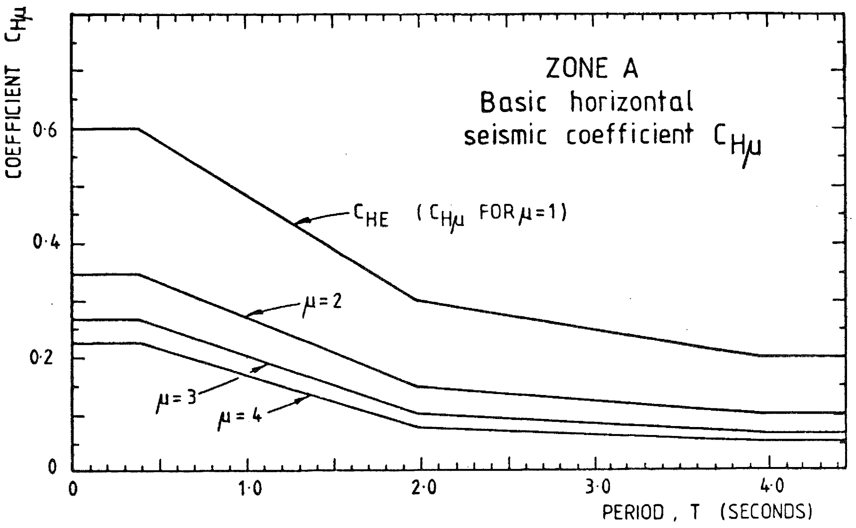
$k$ = an exponent related to the fundamental period of the structure

SEISMIC ZONES
AND
VALUES FOR ZONE B
GEOGRAPHIC COEFFICIENT β

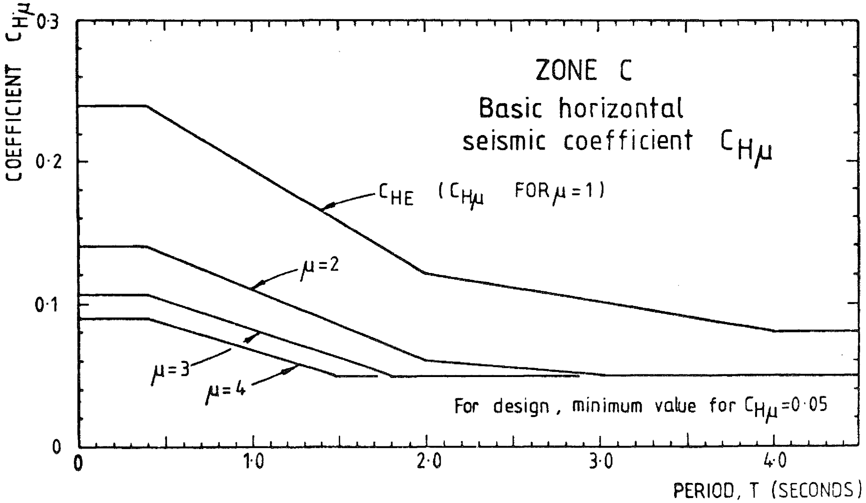


SEISMIC ZONES AND GEOGRAPHIC COEFFICIENT β

FIGURE B: REPRODUCED FROM FIGURE 2.1 OF THE RECOMMENDATIONS



BASIC HORIZONTAL SEISMIC COEFFICIENT $C_{H\mu}$ FOR ZONE A (150 YEAR RETURN PERIOD EARTHQUAKE)



BASIC HORIZONTAL SEISMIC COEFFICIENT $C_{H\mu}$ FOR ZONE C (150 YEAR RETURN PERIOD EARTHQUAKE)

FIGURE C: REPRODUCED FROM FIGURES 2.2 AND 2.3 OF THE RECOMMENDATIONS

| Item | Description of Structural Type | | μ |
|---|---|---|---|
| 1 | All structural types which are designed to remain elastic under the design earthquake loading | | 1 |
| 2 | Braced frames (reinforced concrete or structural steel) | Single storey tension yielding | 3 |
| | | Multi-storey tension yielding | 2 |
| | | Single or multi-storey tension and compression yielding | 3 |
| 3 | Reinforced concrete shear walls | Ductile coupled shear walls | 4 |
| | | Ductile cantilever shear walls | 3 |
| | | Shear walls with a height to width ratio less than 2.0 designed so that brittle shear failure will not occur | 2 |
| 4 | Reinforced masonry shear walls | | 2 |
| 5 | Ductile moment resisting frames | Reinforced concrete or structural steel | 4 |
| | | Prestressed concrete | 3 |
| 6 | Concrete or steel tanks on the ground | | 2 |
| 7 | Chimneys or stacks * | | 2 |
| 8 | Guyed chimneys or guyed stacks | | 1 |
| 9 | Interconnecting pipework | | 1 |
| 10 | Low pedestal supports for pipework | | 1 |
| 11 | Vessels supported on a ductile pedestal or skirt or where hold-down bolts are designed for tension yield under the design earthquake loading | | 3 |
| 12 | Structures designed for foundation rocking | | 2 |

* Chimneys or stacks greater than 20 m high shall be subject to modal analysis (Section 2.6).

Ductility Capability, μ

TABLE A : REPRODUCED FROM TABLE 2.3 OF THE RECOMMENDATIONS

| Class | Description | Design Return Period |
|---|---|---|
| A | (i) Components critical to safe shutdown which must remain operative during and after an earthquake; or<br>(ii) components whose failure represents a severe hazard beyond the confines of the plant; or<br>(iii) in a plant of national economic importance components which cannot be readily repaired, replaced or bypassed following an earthquake. | 1000 years |
| B | (i) Components whose failure represents a hazard to life within the confines of the plant; or<br>(ii) Components which cannot be readily repaired, replaced or bypassed following an earthquake. | 500 years |
| C | Components critical to the working of the plant which can be readily repaired, replaced or bypassed following an earthquake and whose failure does not represent a significant hazard to life within the plant. | 150 years |
| D | Components other than in Class A, B and C above, whose failure represents minimal hazard to life safety. | 75 years |

Guidelines for Design Earthquake Return Periods Appropriate to the Seismic Classification for Components in a Plant with a Design Life of 25 years.

TABLE B: REPRODUCED FROM TABLE 2.2 OF THE RECOMMENDATIONS

$Z_H$ is then determined from Table C which is a tabulation of Figure D discussed in Section 3.6.

| Earthquake Return Period (years) | $Z_H$ |
|---|---|
| 25 | 0.62 |
| 50 | 0.75 |
| 75 | 0.83 |
| 100 | 0.90 |
| 150 | 1.00 |
| 250 | 1.12 |
| 500 | 1.30 |
| 1000 | 1.50 |
| 2000 | 1.73 |
| 3000 | 1.89 |
| 5000 | 2.10 |

Return Period Coefficient, $Z_H$

TABLE C : REPRODUCED FROM TABLE 2.1 OF THE RECOMMENDATIONS

For the seismic classification, importance and risk are assessed from economic as well as social considerations. For a 25-year plant life, the probability of the design loads being exceeded during the 25 years is given below for the A, B, C and D classifications of Table B:

| Seismic Classification | A | B | C | D |
|---|---|---|---|---|
| Earthquake Return Period (years) | 1000 | 500 | 150 | 75 |
| Probability of Exceedance | .025 | .05 | .15 | .30 |
| Design Return Period Coefficient, $Z_H$ | 1.5 | 1.3 | 1.00 | 0.83 |

These probabilities are considered to represent an acceptable level of risk for the given classifications.

For a facility with a design life other than 25 years, the appropriate return period coefficients are chosen, using the procedure discussed in Section 3.6, to achieve the same probability of exceedance for each seismic classification. For example, for a facility with a design life of 50 years:

| Seismic Classification | A | B | C | D |
|---|---|---|---|---|
| Earthquake Return Period (years) | 2000 | 1000 | 300 | 150 |
| Probability of Exceedance | .025 | .05 | .15 | .30 |
| Design Return Period Coefficient, $Z_H$ | 1.73 | 1.5 | 1.15 | 1.0 |

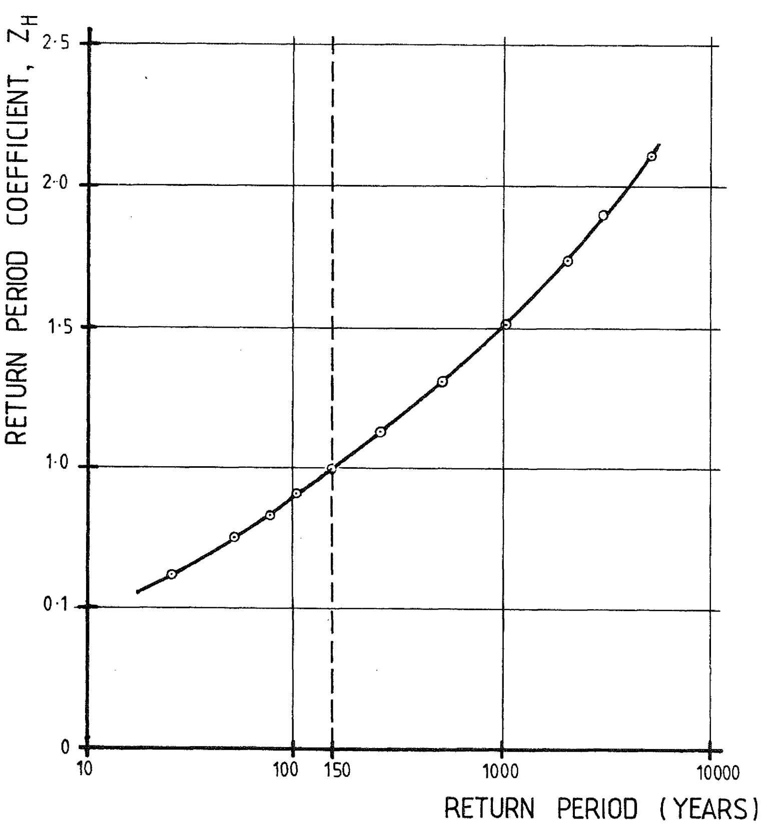
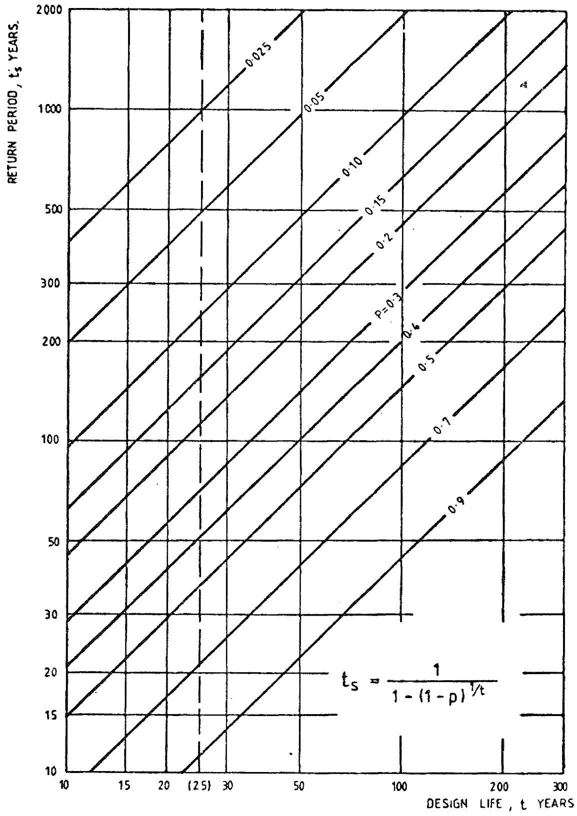


FIGURE D : RELATIVE EARTHQUAKE INTENSITY VS RETURN PERIOD (AS GIVEN BY TABLE 2.1 OF THE RECOMMENDATIONS)



RELATIONSHIP BETWEEN RETURN PERIOD $t_s$ DESIGN LIFE t AND PROBABILITY OF EXCEEDANCE p

FIGURE E : REPRODUCED FROM FIGURE C2.1 OF THE COMMENTARY

$k = 1.0$ for structures with $T < 0.5$ sec

$k = 2.0$ for structures with $T > 2.5$ sec

For structures with $0.5 < T < 2.5$, k may be taken as 2.0 or as determined by linear interpolation.

This equation has been adopted from ATC 3-06[4] and has been found to provide a more representative distribution than that given by the corresponding NZS 4203 prescription, particularly for long period structures.

Exceptions for this distribution are liquid storage tanks on the ground and for guyed structures.

For structures without distinct levels, care is required to ensure the model of lumped masses chosen adequately represents the dynamic properties of the structure.

## 4.7 Modal Analysis

For 'irregular' structures, defined as structures where the storey stiffness or structure weight at adjacent levels vary by more than 30%, the above distribution of forces is not adequate and the Recommendations require a modal analysis to determine the seismic load distribution. Requirements for the modal analysis are given in the Recommendations.

## 4.8 Torsion

The effects of torsion due to actual and accidental eccentricity between the centre of mass and the centre of rigidity of the structure are taken into account by assuming the lateral load at each level acts at the more unfavourable eccentricity given by:

$$e = e_s \pm 0.1b \; * \qquad (6)$$

The effects of torsion need not be considered for axisymmetric structures such as chimneys or vertical cylindrical vessels

*Footnote

The lateral stiffness of the individual frames of the structures which typically form the components of a petrochemical plant can be determined more precisely than in a building. Also the distribution of dead weight and operating live loads can be more accurately assessed. These considerations have led to the conclusion that the ±0.1b accidental eccentricity term is unduly harsh and it is suggested that a more appropriate term for skeletal frame structures would by ±0.05b.

## 4.9 Deflections

The actual deflections of a structure under the design earthquake loading are obtained by multiplying the elastic deformations calculated from the design loads by a factor, λ, given by Table D.

| Type of Structure | $\lambda$ |
|---|---|
| Structures designed to remain elastic under the design earthquake loading ($\mu=1$) | 1.0 |
| Structures designed for a ductility capability, $\mu>1$ | |
| (i) Vessels with hold-down bolts designed for tension yielding, or braced frames with tension yielding braces | $1.5\mu$ |
| (ii) all other structural types | $\mu$ |

Modification Factor, λ for Computing Deformations under the Design Earthquake Loading

TABLE D : Reproduced from Table 3.1 of the Recommendations

The 1.5 factor is introduced to account for the "slop" experiences by "tension yielding only" structural systems after 1 or 2 post yield cycles.

It is noted that if the structure is designed to be stronger than that required by the design earthquake loading, the ductility demand will be less and the deflections calculated from Table D will be conservative for structures with $T < 2.0$ second.

The Recommendations provide guidance on allowable deflections and required structural separation under earthquake loading. Should seismic deflections greater than the prescribed limits be tolerable in terms of the function of the plant component, the designer should ensure that structural integrity is maintained under the additional $P - \Delta$ moment induced into the system by large lateral deflections.

The Recommendations require the design of pipework and other connections between separate structures to accommodate the sum of the deflections computed from Table D for each structure.

This can become a critical design condition particularly for pipework which is required to remain elastic under such deformations. Given the low probability of the peak structure deformations occurring simultaneously it is suggested that the interconnections be designed for an imposed displacement of plus or minus the square root of the sum of the squares of the calculated deflections at the points of attachment.

This suggestion would constitute a revision to clause 3.2.2.2 of the Recommendations which has been found in practice to be overly restrictive.

# 5. CAPACITY DESIGN AND DETAILING PROCEDURES

## 5.1 Capacity Design Principles

The capacity design approach follows

that prescribed by NZS 4203.

For ductile structures, the design seismic loading is reduced from the appropriate elastic response loading by virtue of the fact that the structure will dissipate seismic energy by inelastic deformation of selected elements. The selected elements are required to form a rational yielding mechanism and under the design earthquake the mechanism will have been subjected to a significant number of cycles of inelastic deformation.

There are two aspects to the capacity design procedure. One is that the selected yielding elements must be detailed so that they will dissipate seismic energy while still retaining their strength under the maximum expected deformations. The other is that all components not part of the chosen yielding mechanism (and therefore not detailed to sustain inelastic deformations) must be strong enough to ensure they do not inadvertently yield, interrupt the chosen mechanism and cause premature failure of the structure.

The design loading applied separately in each direction of the structure prescribes the dependable strength ($\phi < 1$) for the elements of the yielding mechanism. The yielding elements are then detailed according to the ductility requirements of the appropriate materials code (or are designed following sound engineering principles where such requirements do not exist).

The formation of the selected yield mechanisms represents the upper bound load to which the non-yielding elements can be subjected. All those elements which are intended to remain elastic are therefore designed to resist, at their ideal strength ($\phi=1$), the maximum member actions resulting from possible overstrength in the yielding elements. The non-yielding elements are designed, in effect, for a further load case where the load E, prescribed in Section 4.2 is replaced by member actions resulting from overstrength in the yielding elements. Because extensive yielding will occur under the design earthquake, the maximum member actions are to be assessed considering the effects of simultaneous hinging of all beams or yielding of all diagonal braces framing into the elements under consideration from all directions.

## 5.2 Overstrength Assessment

The overstrength assessment of the yielding elements should consider possible sources of material overstrength (eg. strengths greater than specified, strain hardening, material confinement) and member overstrength (eg non-structural components contributing to member strengths). Where these cannot be readily assessed, a minimum overstrength factor of 1.25 is recommended.

## 5.3 Elastic Structures

The Recommendations do not require capacity design principles to be used in the design of structures intended to remain elastic under the design seismic load ($\mu=1$). However, the structure is nevertheless to be designed to avoid brittle failure under possible overload conditions. This can generally be achieved by assessing the failure mechanism and checking that brittle elements (eg bolts in shear) do not participate. To permit this assessment, it is stressed that the failure mechanism for 'elastic' structures must be identified as part of the design process.

## 5.4 Specific Detailing Requirements for Ductile Structures

The Recommendations contain specific requirements for the detailing of a number of structural forms commonly used in petrochemical facilities. Comment on some of these is as follows:

*Tall Tubular Vessels:* Ductility may be introduced into the support system for tall tubular vessels by means of a yielding skirt or by a ring of yielding hold-down bolts. In either case, care is required to ensure the mechanism will be ductile. Guidance on the appropriate yield length for the hold-down bolts is given in the Recommendations.

*Braced Frames:* Ductile behaviour can be achieved by designing the braces for tension yielding only or for both tension and compression yielding. Connections must be designed for overstrength in the braces. The design of braces capable of yielding in compression can be difficult and an alternative is to design for ductile eccentric connections.

*Thin Walled Storage Tanks:* The dynamic impulsive and convective effects of liquid movement are to be considered in designing thin walled storage tanks for earthquake loading. Such structures are specified to have a ductility capability $\mu = 2$, on the basis that a load limiting mechanism will be provided by anchorage bolt yield (in the case of small tanks) or by uplift and yielding of the base, together with an associated rocking of the tank on the ground (in the case of large tanks). Seismic resistance for large tanks may be provided by anchorage bolts, but foundation requirements to resist the anchorage bolt loads will normally preclude their usage.

The provisions of API 650, Appendix E[8] are the only codified procedures available for the seismic design of thin walled steel tanks, and subject to a number of modifications to overcome their shortcomings have been adopted in the Recommendations. It must be recognised that the API 650 approach is empirical and also that the imposed limits on shell stress, etc, correspond to allowable material stresses. The tank overturning moment for use in the API 650 approach is therefore evaluated from the Recommendations by the alternative method, i.e. at the 0.8E level rather than at the design earthquake level E. The section of the Recommendations dealing with thin walled steel tanks is currently under review with the aim of introducing capacity design principles into the API 650 approach.

*Pipework:* The Recommendations require that pipework be designed to remain elastic under the design seismic loading. To achieve this the pipework will need to be flexible with earthquake deformations being accommodated by flexure at loops or bends. The Recommendations contain guidance on the detailing of supports and connections to be compatible with this approach.

## 5.5 Rocking Foundations

The concept of rocking foundations is attractive as a means of limiting load on the structure and for dissipating seismic energy - particularly for relatively squat rigid structures. However, there has been insufficient work done to adequately codify the conditions under which such a mechanism is appropriate. The Recommendations allow for structures to be designed to rock at a design load corresponding to $\mu=2$ provided the designs are adequately investigated. References for the design of rocking foundations are given[9,10].

## 6. IN-STRUCTURE EQUIPMENT LOADING

## 6.1 General

Different procedures are prescribed in the Recommendations to evaluate the design loading for items of equipment located in or on a structure depending upon whether the mass and stiffness characteristics of the equipment is likely to have a significant effect on the seismic response of the structure. For 'heavy' items of equipment, defined as having a mass greater than 10% of the total operating weight of the structure or greater than 20% of the total operating weight at the level in the structure at which the equipment is located, a modal analysis is prescribed. For 'small' items of equipment, the design loading is determined from an empirically based formula.

## 6.2 Equipment Design Loading

The horizontal seismic loading for 'small' items of equipment is given by:

$$F_e = C_e Q A_r W_e \qquad (7)$$

where

$C_e$ = equipment horizontal seismic coefficient, see Section 6.3

$Q$ = equipment importance factor

$A_r$ = resonance amplification factor

$W_e$ = operating weight of the equipment

The equipment design loading is used primarily for the design of the connections between the equipment and the structure in which it is located. Note that the equation for $F_e$ does not contain a factor relating to the structural type of the 'part' (c.f. $S_p$ in NZS 4203) as no allowance has been made for a reduction in the design loading resulting from ductility in the equipment support system. In most instances, the design for equipment connections that can withstand the elastic spectral acceleration for the level in the structure at which the equipment is located will not prove too costly.

The equipment importance factor, Q, assumes the value of $Z_H$, the return period coefficient appropriate to the seismic classification of the equipment as assessed from Table B. In an 'elastic' structure, the seismic classification for the structure must not be less than that assigned to the equipment. However, in a 'ductile' structure, the structure seismic classification may be up to one category less than that assigned to the equipment because of the greater assurance that actual collapse of the structure is unlikely to occur under overload conditions; eg, a vital Class A item of equipment may be located in a Class B structure, provided the structure is ductile. However, as the seismic loading on an item of equipment located in a ductile structure will be governed more by the yield level for the structure than the intensity of the earthquake loading, the value used for the equipment importance factor should not be less than that assigned to the structure.

## 6.3 Equipment Seismic Coefficient and Resonance Amplification Factor

Following the guidelines given by Kelly[11], the equipment seismic coefficient is given by:

$$C_e = C_{HE}(\text{equipment}) + \frac{h_x}{h_{cg}} NC_{H\mu(\text{structure})} \quad \text{for } T_e/T < 3.0 \qquad (8)$$

$$= C_{HE}(\text{equipment}) \quad \text{for } T_e/T > 3.0$$

For flexibly mounted equipment, $C_{HE}$ (equipment) is taken as the appropriate ground-based elastic seismic coefficient as given by Figure C. For rigidly mounted equipment ($T < 0.05$ second), $C_{HE}$ (equipment) may be replaced by a factor, G, representative of peak ground acceleration, G=0.35 (Zone A), G=0.35β (Zone B) or G=0.14 (Zone C).

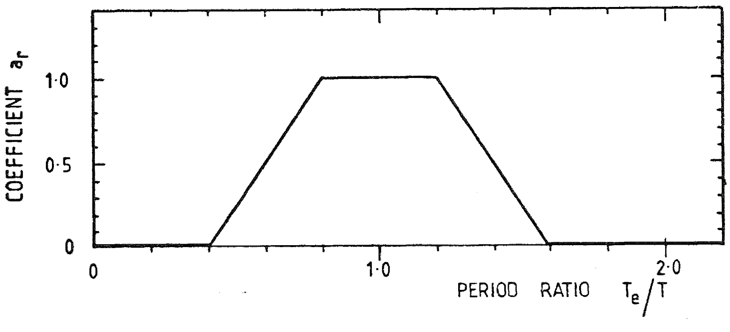
The amplification of the ground based response of the equipment by the yielding characteristics of the structure and its location in the structure is accounted for by the second term in the first of the above equations. The factor N, which equals 1.25 for ductile structures, takes into account probably structural overstrength.

Kelly's finding that the seismic response of flexibly mounted equipment is not significantly influenced by the seismic response of the structure if $T_e$ is greater than about 3 times the period of the structure, is accounted for in the second of the above equations.

The resonance amplification factor, $A_r$, is given by:

$$A_r = (1 + a_r \frac{h_x}{h}) \qquad (9)$$

where $h_x$ is the height at which the equipment is located, h is the total structure height and $a_r$ is given by:



The resonance amplification factor, $A_r$, takes account of partial resonance effects which occur when the earthquake input motion to the equipment, as filtered by the structure, has a dominant frequency close to that of the equipment. The findings of Reference 11 that 'resonance' effects are more pronounced at higher levels in the structure are incorporated in the equation for $A_r$.

## 7. SEISMIC PERFORMANCE QUALIFICATION FOR ITEMS OF EQUIPMENT AND CONTROL SYSTEMS

### 7.1 General

In addition to the design of items of equipment and control systems for the loadings prescribed in Section 6, the Recommendations require performance qualification to assess how the workings of the equipment will be affected under earthquake conditions.

### 7.2 Levels of Seismic Performance Qualification

The level of seismic performance qualification required depends upon the importance and/or risk of the equipment. Three levels of performance qualification are prescribed in the Recommendations:

| | |
|---|---|
| Level I | : Dynamic Testing - Class A items of equipment |
| Level II | : Analysis - Class B items of equipment |
| Level III | : Qualitative Assessment - Class C and D items of equipment. |

For equipment which is not critical to plant operation and whose functional disruption by an earthquake would not create a significant safety hazard, the seismic performance may be assessed qualitatively. Judgement can often suggest where modifications to traditional configurations, eg a simple redistribution of weight, can significantly improve seismic performance. The Recommendations require all equipment to be performance qualified at Level III before the more cost-intensive qualification at Level I or II is undertaken.

Items of equipment whose disruption may result in a significant safety hazard, or items which may be needed soon after an earthquake and must therefore be protected from extensive damage, would normally be performance qualified at Level II.

Dynamic testing is an expensive exercise and should only be prescribed for items of equipment essential to safe shutdown of the plant or for emergency equipment. Typical items in this category include automatic fire protection systems, communication systems and control systems associated with plant shutdown. A number of dynamic testing procedures are outlined in the Recommendations. It is stressed that testing should be carried out with the equipment mounted in a manner which closely represents its intended service mounting and that all external and internal operating loads and functions should be simulated during the test.

### 7.3 Fail Safe Isolation Valves

Where equipment is fitted with fail safe valves which will isolate an item of equipment even in the event of loss of the control system, the requirements for seismic performance qualification of the equipment may be waived. Should the process exceed safe operational limits as the result of disruption by earthquake or any other cause, the monitoring equipment will detect a process malfunction and isolate the equipment by activating the fail safe valve. To provide assurance that the fail safe valve will not malfunction or that the monitoring equipment will not give erroneous readings as a result of the earthquake shaking (i.e. give a reading indicating that the process is within operational limits when it is not) seismic qualification of the valve and the monitor will be required at a level appropriate to the classification of the equipment it protects.

## 8. SPECIAL STUDIES:

The Recommendations provide the designer with a set of readily applied seismic design criteria. However, situations may arise where the provisions are not appropriate or where specialised information is available to override them. The Recommendations permit special studies to be undertaken to provide, by more rigorous analysis, a better understanding of the seismic performance of a structural system or to arrive at design parameters more applicable to a given site or plant component.

The conditions that apply to special studies are that:

(a) the design philosophy embodied in the Recommendations is retained, and

(b) the objectives, procedures and conclusions of any special study used in the design are submitted for review (and acceptance by) the approving authorities.

The areas where special studies may be undertaken are specified in the Recommendations. These are:

(i) site specific seismicity - a detailed study of local site conditions, geological features and earthquake history may be used to establish basic horizontal seismic coefficients appropriate to the site;

(ii) risk analysis - alternative design earthquake return periods may be used if supported by a properly constituted risk analysis of the economic and social consequences of possible earthquake damage;

(iii) ductility capability and detailing for ductile behaviour - the results of experimental studies may be used to modify the ductility assessments and detailing requirements of the Recommendations;

(iv) structure response to seismic loading - dynamic time-history analysis may be used to supplement the equivalent static force and modal analysis procedures given in the Recommendations.

## ACKNOWLEDGEMENTS:

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The contribution of Dr R.W.G. Blakeley who set the framework for the study and supervised its early stages, is also acknowledged.

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## NOTATION:

| | |
|---|---|
| $a_r$ | resonance coefficient (function of $T_e/T$) |
| A | alternative method design loading combination |
| $A_r$ | resonance amplification factor |
| b | horizontal dimension of structure, measured perpendicular to the direction of the applied load |
| $C_e$ | equipment horizontal seismic coefficient |
| $C_{H\mu}$ | basic horizontal seismic coefficient |
| $C_{HE}$ | basic horizontal seismic coefficient for structures designed to remain elastic under the earthquake loading ($\mu$=1) |
| D | dead load |
| e | eccentricity |
| $e_s$ | horizontal distance, measured perpendicular to direction of applied load between, centre of mass and centre of resistance |
| $F_e$ | equipment design horizontal seismic force |
| $F_x$ | design horizontal seismic force at level x in a multi-level structure |
| G | coefficient representing peak effective ground acceleration during a 150-year return period earthquake |

| | |
|---|---|
| $h$ | total height of the structure |
| $h_{cg}$ | height of the centre of mass of the structure |
| $h_x$ | height of level x in a multi-level structure |
| $k$ | an exponent related to the fundamental period of the structure |
| $L_o$ | live load under normal operating conditions |
| $N$ | factor used in the prescription for equipment loading to account for probable overstrength in a ductile structure |
| $P$ | probability of occurrence of an earthquake equal to or greater than the design earthquake within the design life of the structure |
| $Q$ | equipment importance factor |
| $t$ | design life of the structure |
| $t_s$ | earthquake return period |
| $T$ | fundamental period of the structure |
| $T_e$ | fundamental period of the equipment |
| $U$ | strength method design loading combination |
| $V$ | seismic base shear |
| $W$ | total structure weight $(D+L_o)$ above ground |
| $W_e$ | operating weight of the equipment |
| $W_x$ | total structure weight $(D+L_o)$ at level x in a multi-level structure |
| $Z_H$ | design earthquake return period coefficient |
| $\beta$ | seismic geographic coefficient for Zone B |
| $\Sigma$ | summation sign |
| $\lambda$ | modification factor for structural deformations |
| $\phi$ | strength reduction factor |
| $\mu$ | ductility capability of the structure |