THE DEVELOPMENT OF SEISMIC ZONES AND THE EVALUATION OF LATERAL LOADINGS FOR EARTHQUAKE RESISTANT DESIGN OF BUILDINGS IN PAPUA NEW GUINEA

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ABSTRACT:

The basis for modern earthquake resistant design can be considered to be a two stage process the objectives of which can be summarised as follows:

1. Provide the structure with sufficient strength and stiffness to resist moderate earthquakes so that the frequency of occurrence of structural and non-structural damage is acceptably low, and

2. Ensure that the probability of collapse of the structure and the risk to life in a severe earthquake is acceptably low.

The first stage can be satisfied by seismic zoning to ensure that the risk of damage to structures of similar structural type is acceptable and approximately uniform over the whole country and by restricting interstorey deflections under moderate earthquakes.

The second stage can be satisfied by the use of structural type factors.

In particular, this study explains how these principles were used to develop seismic zones and evaluate lateral loadings for Earthquake Resistant Design for Buildings in Papua New Guinea.

1.0 SYNOPSIS:

This paper presents the results of a study commissioned by the Department of Works and Supply of Papua New Guinea to develop a zoning map and load co-efficients to be presented in conjunction with a general review of the PNG Building Regulations and in particular with an Interim Code of Practice for Earthquake Loadings and the Design of Earthquake Resistant Structures.

The procedure presented has also been used to derive seismic zones and determine seismic lateral design loadings for building construction in Indonesia.

The aim of the method is to divide Papua New Guinea into seismic zones so that over the whole region the expected frequency of occurrence of structural damage from earthquakes for similar structural types is approximately uniform. The expected frequency of occurrence of damage can be directly related to the expected frequency of occurrence of elastic acceleration response in a building (for a given structural period and site condition) by the choice of design load levels within each zone.

The expected frequency of occurrence of a building's elastic acceleration response at any particular site is calculated from:

(a) The expected frequency of occurrence of shallow (i.e. depth less than 65km) earthquakes of different magnitudes near the site (i.e. say within 300km), and

(b) The attenuation of building structural elastic response with distance from an earthquake of any particular magnitude.

By inverting the frequency of occurrence of the building's peak elastic acceleration response, the return period of that peak response is found.

At this point in the method a decision must be made as to how the zones and therefore the loads are to be defined. Either an acceptable return period for the onset of structural damage is chosen and the appropriate loads adopted, or the loads are chosen (i.e. to agree with existing code levels) and the implied return period for onset of structural damage checked to see whether it is acceptable. The onset of structural damage has been defined in this paper as the point at which yield can be expected to have occurred in a significant number of primary elements. Primary elements have been defined as those elements in the structure whose failure would result in a substantial loss in load carrying capacity.

The zoning map presented in Section 5 is based on the 20 year return period of peak acceleration response for short period structures on soft ground as this is believed to be typical of many structures in Papua New Guinea.

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2.0 DETERMINATION OF THE EXPECTED FREQUENCY OF OCCURRENCE OF SHALLOW EARTHQUAKES:

2.1 General

The method of approach is to:

(a) Determine the historical frequency of occurrence of shallow earthquakes in PNG.

(b) Make an assessment of the expected future occurrence of earthquakes.

(c) Produce a map of expected frequency of shallow earthquakes of magnitude 7.0 or greater throughout Papua New Guinea.

(d) Use of recurrence relationship to calculate the expected frequency of occurrence of earthquakes above any magnitude, or within any magnitude interval.

2.2 HISTORICAL FREQUENCY OF OCCURRENCE OF SHALLOW EARTHQUAKES:

The historical occurrence of earthquakes was based on an earthquake listing provided by the Bureau of Mineral Resources in Australia. Plots of Major Shallow Earthquakes (< 65km in depth) and Major Deep Earthquakes (>65km in depth) obtained from this listing for the period 1900→1978 are given in Figures 1 and 2 respectively.

The more complete a listing of earthquakes, the more valuable it is for making estimates of future seismicity. From a simple study of earthquake frequencies it was concluded that the list of shallow earthquakes would contain practically all earthquakes of magnitude 7 or more since 1900, of magnitude 6 or more since 1930 and of magnitude 5 or more since 1960. From the listings amended for the above time periods contour maps showing the average historical frequency of occurrence per century per square degree were produced for earthquakes above magnitude 7.0, above magnitude 6.0 and above magnitude 5.0. These maps are presented in Figures 3, 4 and 5.

The average frequency of occurrence for each of these maps has been calculated by summing the numbers of earthquakes within the specified magnitude ranges within each one degree square, and assuming that this frequency is to apply at the centre of the respective square, contours have been drawn by linear interpolation.

In any region the number of earthquakes "N" above some stated magnitude "M" is found to follow a law of the type, $\log N = a - bM$, where a and b are constants. If it is assumed that for any period of record the slope of the line, b, will remain constant it is possible to relate the frequency of earthquakes above any magnitude to the frequency of earthquakes above any other magnitude. A value of $b = 1.0$ was used in this study.

This value was selected as being representative for the type of seismic activity in Papua New Guinea on the basis of references 12 and 13.

Using this relationship the contours of frequency of occurrence of earthquakes of magnitude greater than5.0 and also for magnitudes greater than 6.0 were adjusted to give an implied value for magnitude 7.0 or greater. Thus, it was possible to use the more statistically reliable distribution (in terms of numbers of events) of smaller magnitude earthquakes to fill in the gaps left in the distribution of larger earthquakes which although recorded over a longer period are small in number.

By overlaying the maps derived above showing frequency of occurrence of earthquakes of magnitude 7.0 or greater as determined from each magnitude range, it was possible to construct a further map illustrating the expected frequency of shallow earthquakes of magnitude 7.0 or more per square degree per century based primarily on epicentral data.

Rationalisation of the map to account for geological data was then carried out.

2.3 ASSESSMENT OF THE FUTURE OCCURRENCE OF EARTHQUAKES:

The final assessment of future occurrence of earthquakes was made by amending the map of expected occurrence of earthquakes from epicentral data (determined in 2.2 above) to take into account geological data. In general terms for Papua New Guinea this involved adjusting the contours so that they followed the trends of generally accepted seismotectonic plate boundaries.

From this final map (refer Figure 6) it is possible to once again apply the relationship described in section 2.2 and thus determine the expected frequency of occurrence of earthquakes greater than any magnitude value, or by further manipulation, the expected frequency of occurrence of earthquakes within any magnitude interval at any selected point on the map.

The specification of Figure 6 together with appropriate 'b' value therefore provides a complete statement on the estimate of expected earthquake occurrence in Papua New Guinea.

3.0 ATTENUATION OF BUILDING ELASTIC RESPONSE WITH DISTANCE FROM AN EARTHQUAKE:

3.1 General

The seismicity of Papua New Guinea is described in Section 2 by the estimated average expected occurrence of earthquakes of varying magnitude. This information alone is insufficient to determine the seismic risk to structures. This is because moderate to large earthquakes will cause damage to structures at varying distances from their epicentres depending on the earthquake magnitude and the type of ground condition near to the structure and other parameters.

The risk at any point is therefore also dependent on the likelihood of earthquake occurrence at surrounding points. Furthermore, the relative risk between points is measured in terms of relative probability

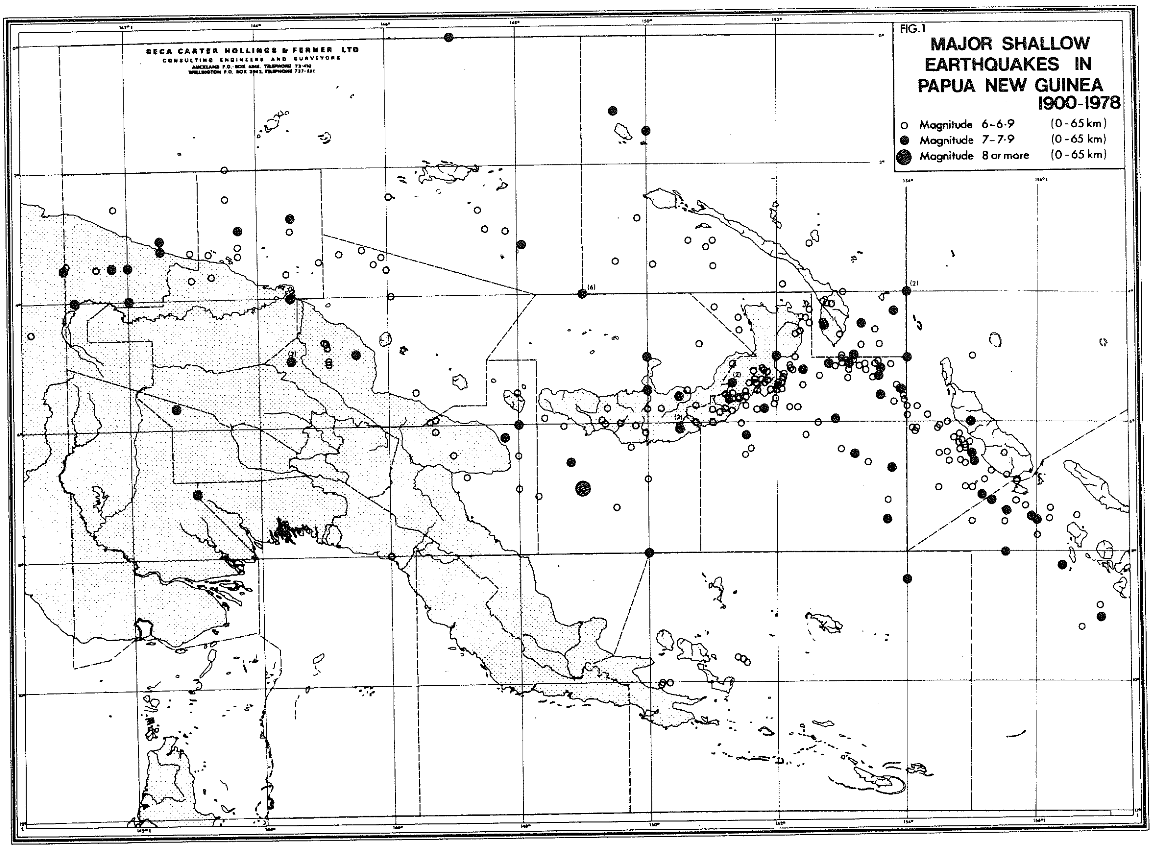
FIG. 1
MAJOR SHALLOW EARTHQUAKES IN PAPUA NEW GUINEA
1900-1978
Magnitude 6-6·9 (0-65 km)
Magnitude 7-7·9 (0-65 km)
Magnitude 8 or more (0-65 km)
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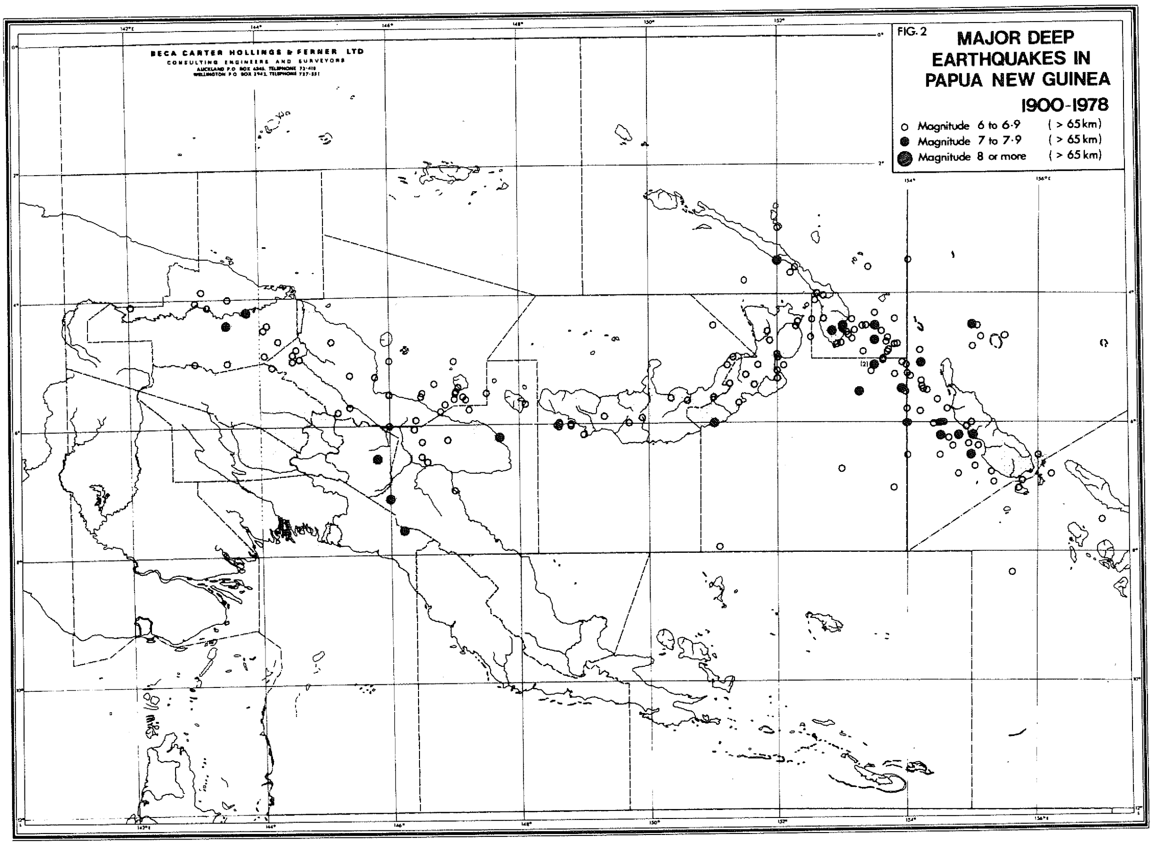
FIG. 2
MAJOR DEEP EARTHQUAKES IN PAPUA NEW GUINEA
1900-1978
Magnitude 6 to 6·9 (> 65 km)
Magnitude 7 to 7·9 (> 65 km)
Magnitude 8 or more (> 65 km)
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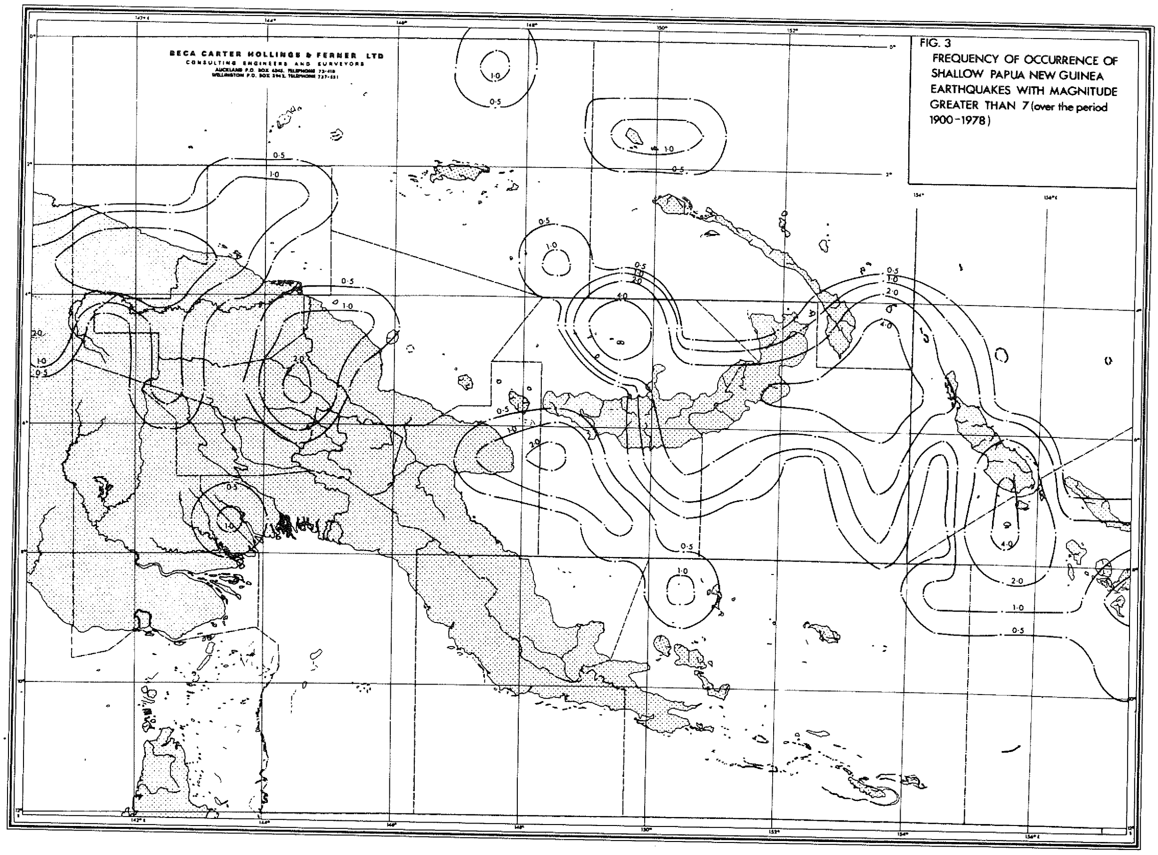


FIG. 3
FREQUENCY OF OCCURRENCE OF SHALLOW PAPUA NEW GUINEA EARTHQUAKES WITH MAGNITUDE GREATER THAN 7 (over the period 1900–1978)

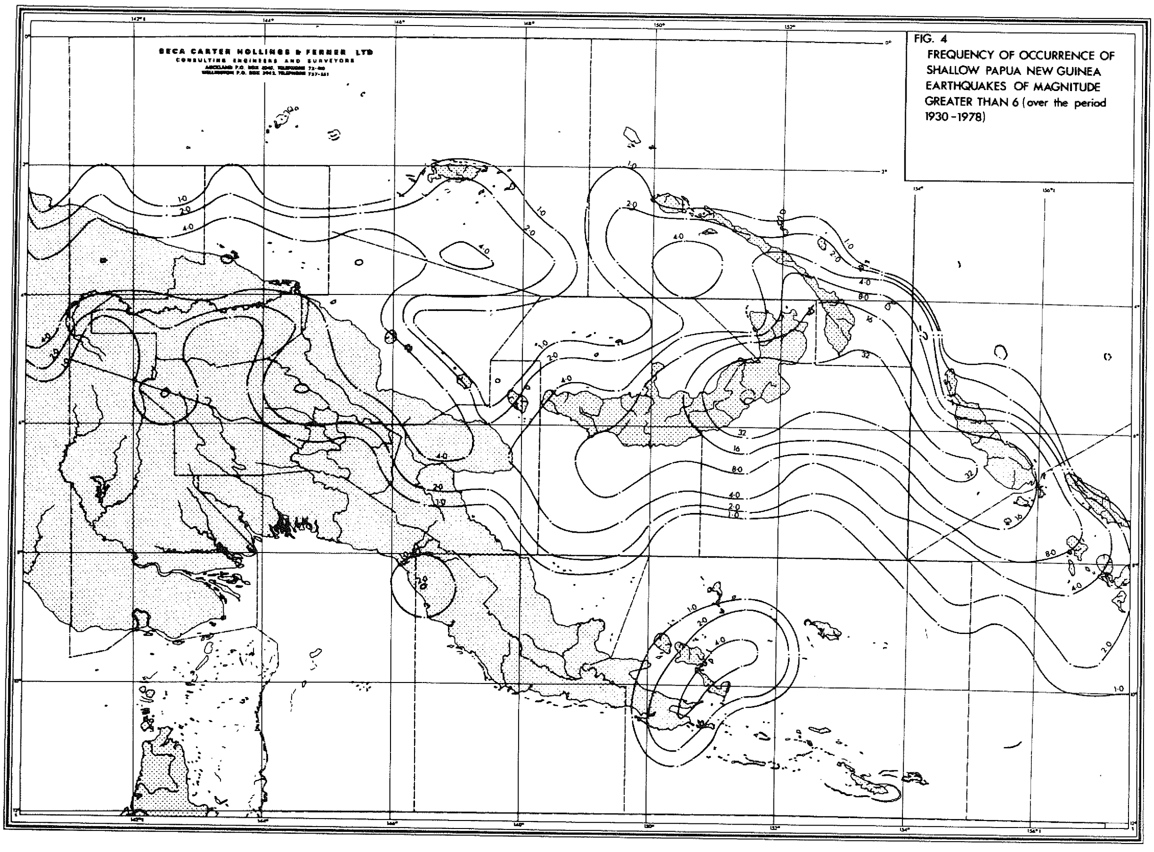


FIG. 4
FREQUENCY OF OCCURRENCE OF SHALLOW PAPUA NEW GUINEA EARTHQUAKES OF MAGNITUDE GREATER THAN 6 (over the period 1930–1978)

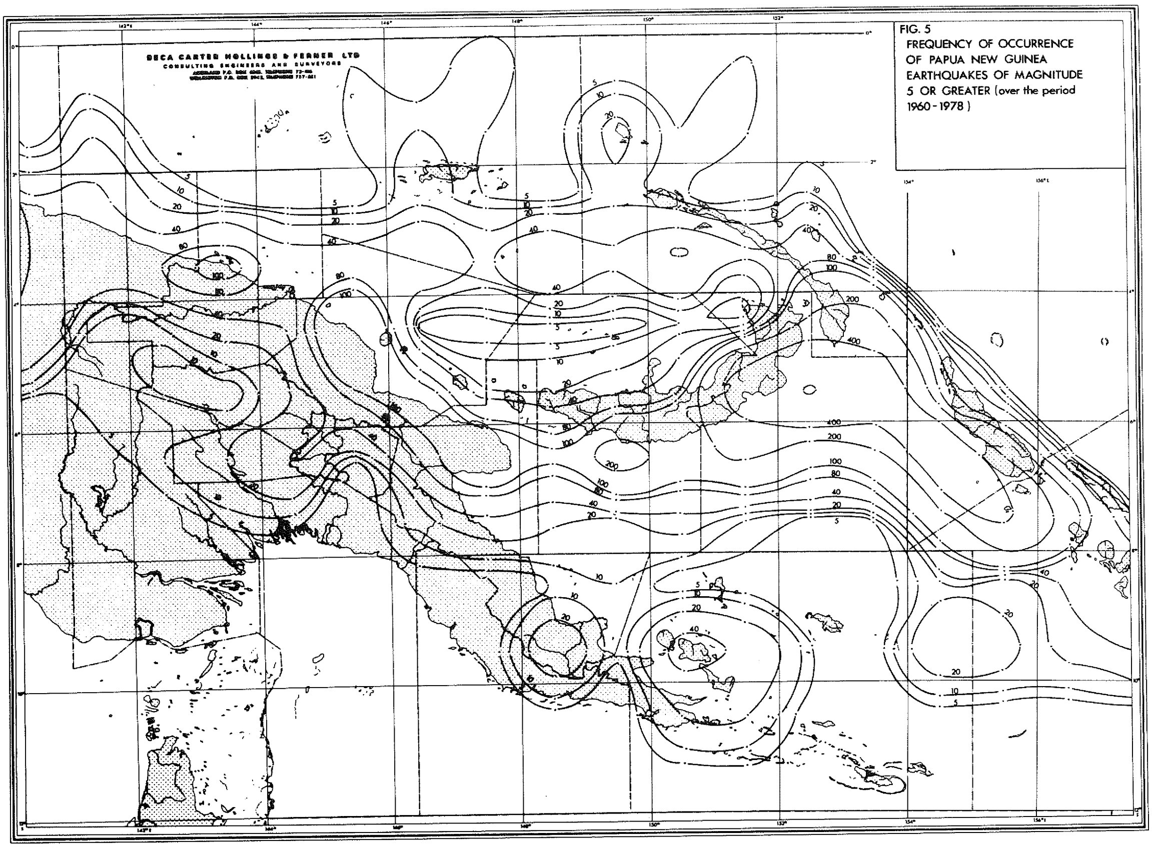


FIG. 5
FREQUENCY OF OCCURRENCE OF PAPUA NEW GUINEA EARTHQUAKES OF MAGNITUDE 5 OR GREATER (over the period 1960-1978)

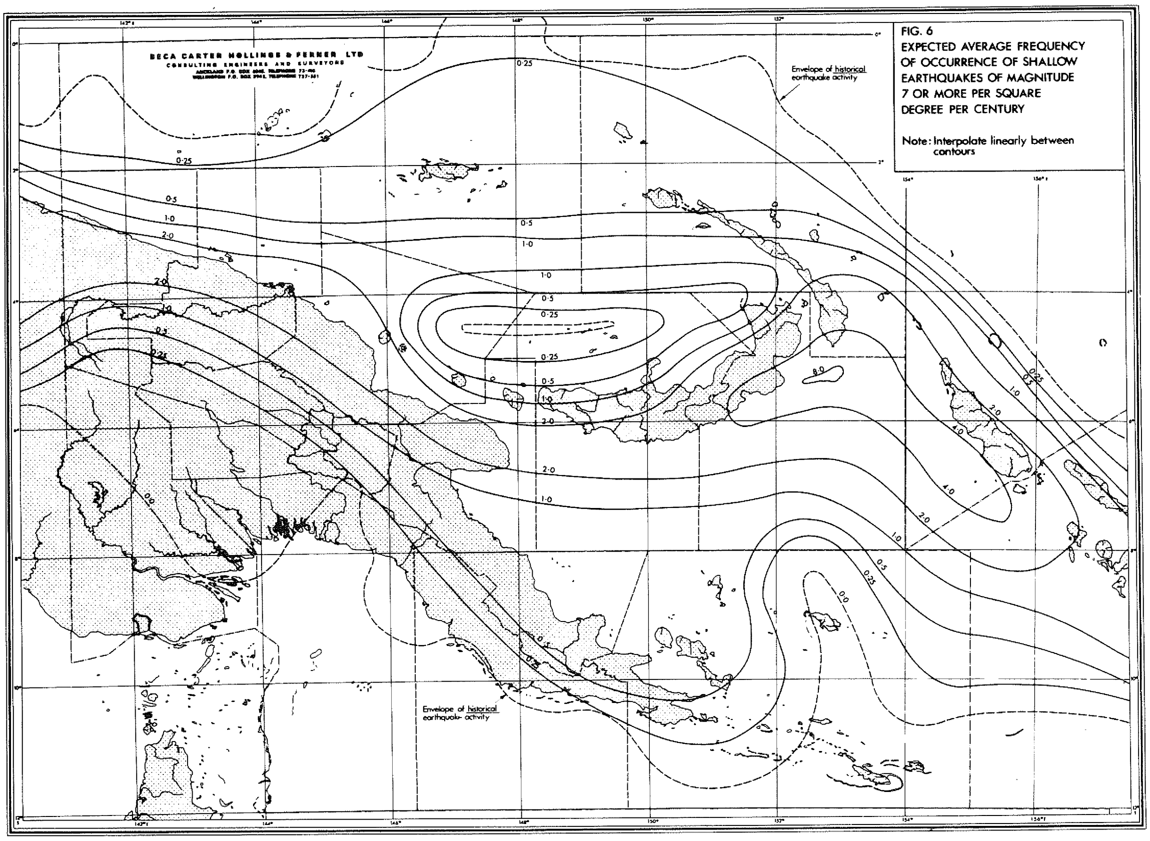


FIG. 6
EXPECTED AVERAGE FREQUENCY OF OCCURRENCE OF SHALLOW EARTHQUAKES OF MAGNITUDE 7 OR MORE PER SQUARE DEGREE PER CENTURY

Note: Interpolate linearly between contours of occurrence of a level of structural response in similar structures. It is therefore necessary to combine earthquake occurrence with a statement relating structural response to earthquake magnitude, distance and ground condition, to gain a more complete picture of the seismic risk to buildings.

Elastic horizontal acceleration response spectra determined from earthquake strong-motion were selected as a suitable measure of the effect on buildings of earthquake ground motion in Papua New Guinea. The advantage in using elastic acceleration response spectra is that they provide by definition the peak horizontal acceleration and therefore the peak lateral force in a single degree of freedom oscillator of varying natural period and specified damping. It is usual to assume that response spectra give, to a close approximation, the peak lateral force in most elastic structures. Although the peak lateral force is a limited measure of the extent of earthquake damage (particularly in ductile structures) it does more precisely indicate whether a structure has been stressed past the elastic limit, and therefore is an indicator of the possible onset of structural damage.

It is common to construct response spectra for discrete levels of viscous damping (0, 2, 5, 10, 20% are usual). Various factors appear to be important in determining the level of damping in buildings including the degree of excitation of the structure, the degree of separation of structural and non-structural elements, the material of construction and the nature of construction (bolted, cast in-situ, prefabricated etc). Structural response may also be altered by the absorption of energy by yielding of the foundation soil. The relative level of earthquake risk is not greatly influenced by altering the damping ratio and in this study 5% damping has been chosen as an average value appropriate to normal construction in Papua New Guinea.

The production of response spectra from strong-motion records is a lengthy process. Partly for this reason, very few Papua New Guinea strong motions have been analysed. It has been necessary therefore to adopt typical average acceleration response spectra for Papua New Guinea on data from regions outside Papua New Guinea.

3.2 DETERMINATION OF THE RELATIONSHIP BETWEEN STRUCTURAL RESPONSE DISTANCE AND GROUND CONDITION (ATTENUATION RELATIONSHIPS)

In a previous study[1] of over 130 strong motion records from 90 earthquakes from 15 countries, a statement on the attenuation with distance of structural response (in terms of expected average elastic peak acceleration of short and long period structures), on soft and hard ground for earthquakes of varying magnitudes, was determined. This relationship is shown in Figure 7. In the absence of data from Papua New Guinea earthquake records the relationship shown in Figure 7 has been adopted as the best available for the purposes of this study.

4.0 DETERMINATION OF THE EXPECTED FREQUENCY OF OCCURRENCE OF ACCELERATION RESPONSE (SEISMIC RISK):

4.1 General

The expected frequency of occurrence of acceleration response at a site (for a given structural period and site condition) was determined directly from the appropriate attenuation curves (discussed in 3.0 and the estimates of earthquake occurrence near the site (discussed in 2.0).

In its simplest form the calculation was as follows: Consider a particular site at which we require the estimate of risk:

1. Select a site condition (soft or hard) and a particular structural period (simplified in this study to less than 0.5 secs and over 2 seconds for soft ground and less than 0.25 secs and over 2 secs for hard ground).

2. Select a level of elastic acceleration response say 0.2g.

3. Select an earthquake size, say magnitude 6½ ± ¼.

4. From the attenuation curves (Figure 7) find the radius of the circle (whose centre is at the chosen site) in which magnitude 6½ earthquakes cause a response of 0.2g or more, say 70km (i.e. short period structure on soft ground).

5. From the earthquake occurrence data find the expected frequency of occurrence of earthquakes of magnitude 6½ ± ¼ within 70km of the site.

6. Repeat steps 3, 4 and 5 for all the different magnitude intervals (i.e. 5½ ± ¼, 6 ± ¼, 6½ ± ¼ ....., 8 ± ¼, 8½ ± ¼ in this study) and add the expected frequencies.

   The result is the expected frequency of occurrence of an elastic acceleration response of 0.2g or more in structures with 5% damping with the selected period of vibration and site condition at this particular site.

   Two refinements to the calculation procedure are necessary before the return period is correctly determined. The first involves an adjustment to take account of the elliptical nature of the contours of equal shaking isoseismals. The second refinement takes account of the errors introduced by using deterministic attenuation relationships. Further discussion of these points is however beyond the scope of this paper. The return period of this response then follows directly by inversion of the frequency.

7. Repeat steps 2. → 6. for a number of different elastic responses (i.e. 0.05, 0.10, 0.15, 0.20, 0.30, 0.40, 0.60, 0.80, 1.00, 1.20g in this study).

8. Repeat steps 2. → 7. for a number of different sites on a grid covering Papua New Guinea.

The calculation procedures described above were carried out by computer. By interpolation it is now possible to draw contours of response for any given return period.

The following four maps of structural response for a 20 year return period are presented:

Figure 8 Short (i.e. up to 0.5 secs) period structures on hard ground

Figure 9 Long (i.e. over 2 secs) period structures on hard ground

Figure 10 Short (i.e. up to 0.25 secs) period structures on hard ground

Figure 11 Long (i.e. over 2 secs) period structures on hard ground

5.0 SELECTION OF SEISMIC ZONES FOR BUILDING CONSTRUCTION IN PAPUA NEW GUINEA:

5.1 General:

In the context of this study, seismic zones are regions where, for a particular structural type, the expected frequency of occurrence of structural damage from earthquakes is approximately uniform. Seismic zoning maps were constructed from Figures 8, 9, 10 and 11 for different structural periods and ground conditions. However, for administrative and for practical reasons it was considered desirable to have only one map in a loadings code. The zones in this study have been based on a return period of 20 years, however the zone boundaries are unlikely to change significantly for other choices of return period.

5.2 DEVELOPMENT OF SEISMIC ZONES:

In this study seismic zones for building construction were delineated from the contour map for short period structures founded on soft ground (Figure 8). There were two main reasons for this choice. Fir stly, it can be assumed that most structures in Papua New Guinea are most likely to be founded on soft ground in river valleys and that most would tend to have a low natural period of vibration. Secondly, detailed inspection of the relative level of response between sites in Figures 9, 10 and 11 has shown that the adoption of Figure 8 as a basis for the zoning map does not lead to inaccuracies above those inherent in the risk calculation procedure. This means that zones based on other combinations of soil stiffness and building period will not differ significantly from those proposed.

Seismic zones were derived from Figure 8 by defining suitable intervals of acceleration response. For example a seismic zone could be defined by the 0.15g and 0.20 g contours on Figure 8.

It can be seen from Figure 8 that the seismic risk varies considerably across Papua New Guinea. Extremely high risk areas include Bougainville and Buka Islands and the eastern region of New Britain. The very south of Western New Guinea would be an area of very low risk along with Louisiade Island arch. Areas of moderately high risk would include the northern coast of Papua New Guinea and for moderate risk the central region of Papua New Guinea including the D'Entrecasteaux Islands.

By selecting appropriate contour intervals to from zone boundaries the following areas were chosen:

Area	Interval of Maximum Acceleration Response for 20 year Return Period. (Refer Figure 8)	Representative Response
1	0.68g or greater	0.75
2	0.54g - 0.68g	0.61
3	0.4g - 0.54g	0.47
4	Less than 0.4g	0.29

In defining the areas above, some attempt was made to match the boundaries to zone boundaries in existing documents[11].

The representative response levels for the four areas is also illustrated in the right-hand column. If the frequency of the potential onset of structural damage was the only criteria, the lateral load levels in each zone for stiff structures on soft ground would be proportional to these values.

Figure 12 illustrates the four areas. The figure can be considered as a possible seismic zoning map based on scientific evidence. Different maps with fewer or more zones could have been prepared and the boundaries could have been altered by adjusting the response intervals for each area.

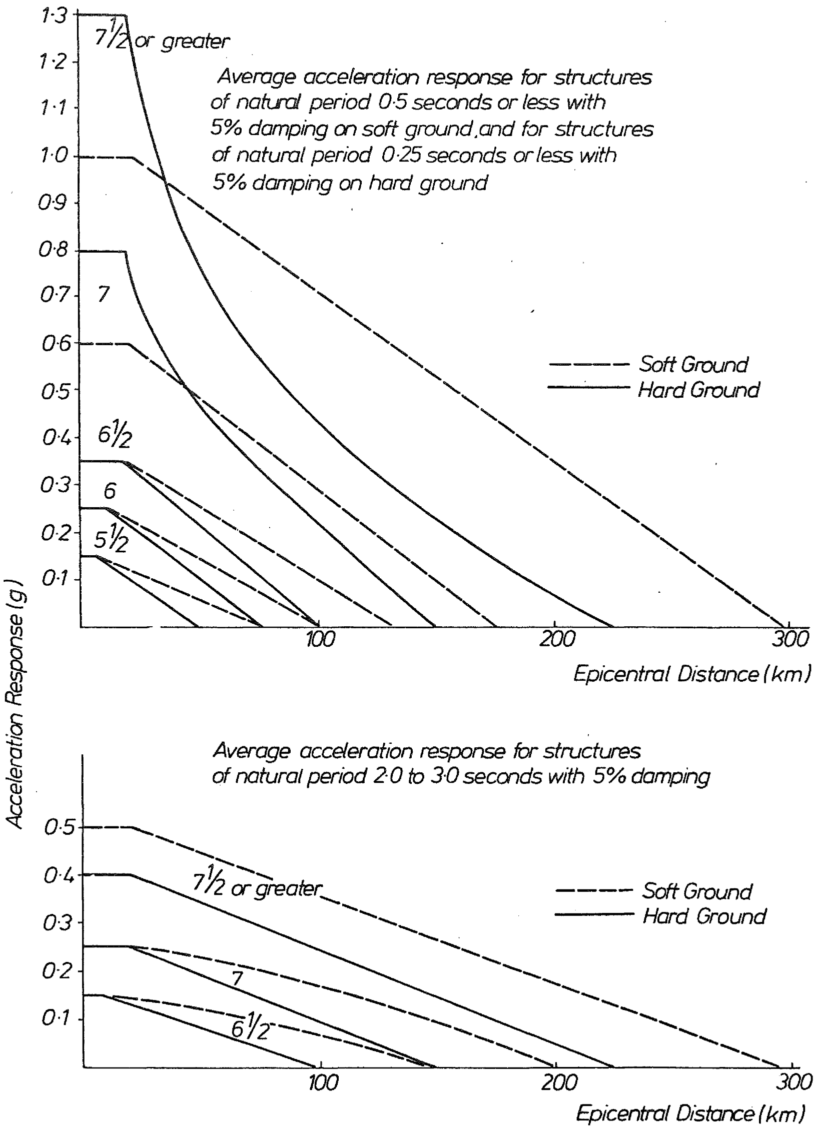
It should be noted that the use of Figure 8 as a basis for zoning does not mean that Figures 9, 10 and 11 are of no further value. In Section 6.0, factors are derived from a comparison of Figure 8 with Figures 9, 10 and 11 and these factors are subsequently used to define relative lateral loads for long period structures on soft and hard ground and for short period structures on hard ground.

Figure 13 indicates the zoning map finally adopted for the Papua New Guinea Loadings Code. It is administratively expedient to rigorously define zones and this means that the boundaries do not necessarily follow exactly those shown in Figure 12. The deviation of the south-western boundary between Zones 3 and 4 to include Kiunga, Mendi and Kerema in Zone 3 results from recent studies by the PNG Geophysical Observatory (unpublished) which indicate that some magnitude 6 earthquakes may have occurred prior to 1950 which have remained undetected and uncatalogued. It was decided that the above amendments should be made until further data is available to assess historical activity in this area. The zoning map could then be changed if appropriate.

6.0 REVIEW OF PROCEDURES FOR DETERMINING SEISMIC LOADING PROVISIONS:

6.1 Historical Development

Lateral "earthquake loads" were first



ATTENUATION OF AVERAGE PEAK HORIZONTAL ACCELERATION RESPONSE AT 5% DAMPING ON SOFT AND HARD GROUND FOR EARTHQUAKES OF VARYING MAGNITUDE (For intermediate structural periods, curves can be obtained by interpolation) FIGURE 7

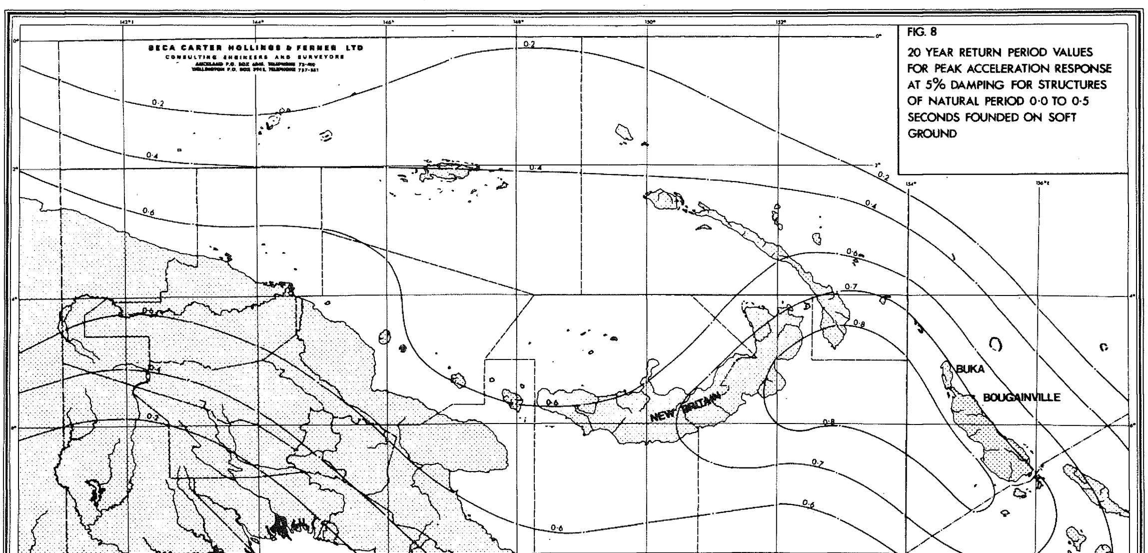


FIG. 8
20 YEAR RETURN PERIOD VALUES FOR PEAK ACCELERATION RESPONSE AT 5% DAMPING FOR STRUCTURES OF NATURAL PERIOD 0·0 TO 0·5 SECONDS FOUNDED ON SOFT GROUND

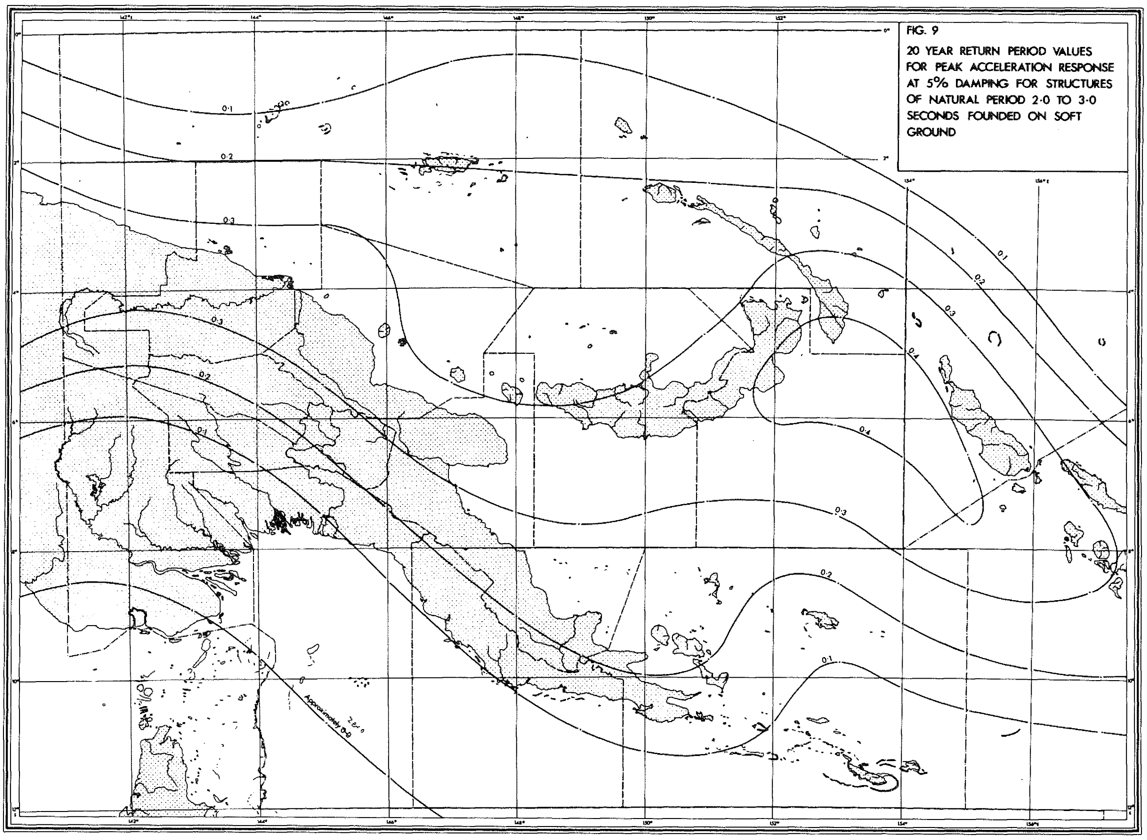


FIG. 9
20 YEAR RETURN PERIOD VALUES FOR PEAK ACCELERATION RESPONSE AT 5% DAMPING FOR STRUCTURES OF NATURAL PERIOD 2·0 TO 3·0 SECONDS FOUNDED ON SOFT GROUND

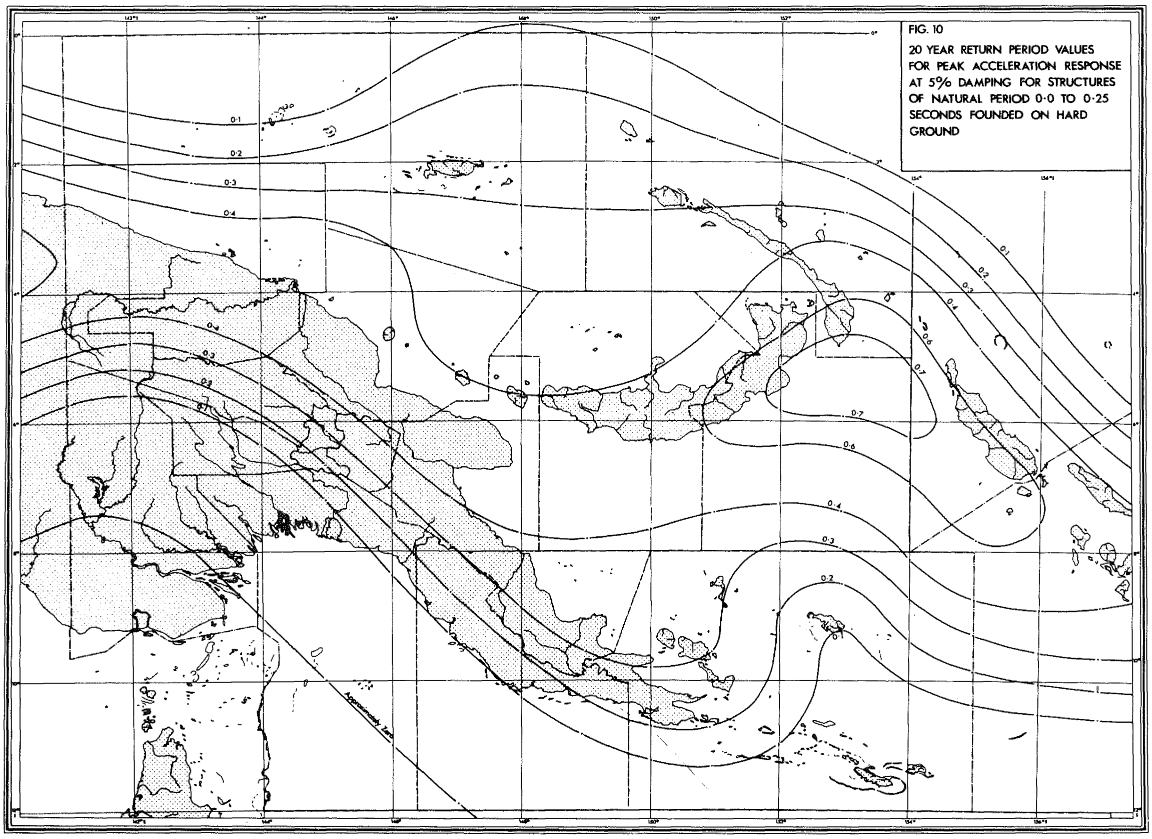


FIG. 10
20 YEAR RETURN PERIOD VALUES FOR PEAK ACCELERATION RESPONSE AT 5% DAMPING FOR STRUCTURES OF NATURAL PERIOD 0·0 TO 0·25 SECONDS FOUNDED ON HARD GROUND

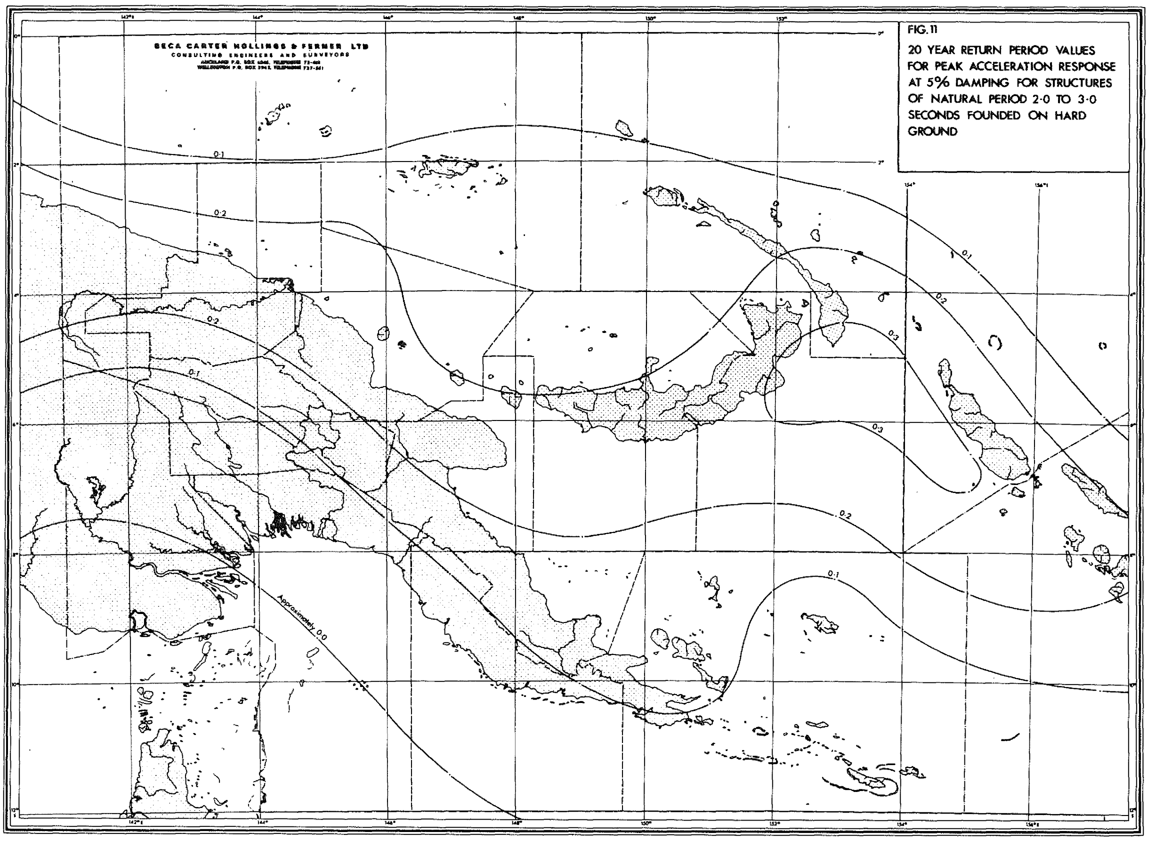


FIG. 11

20 YEAR RETURN PERIOD VALUES FOR PEAK ACCELERATION RESPONSE AT 5% DAMPING FOR STRUCTURES OF NATURAL PERIOD 2·0 TO 3·0 SECONDS FOUNDED ON HARD GROUND

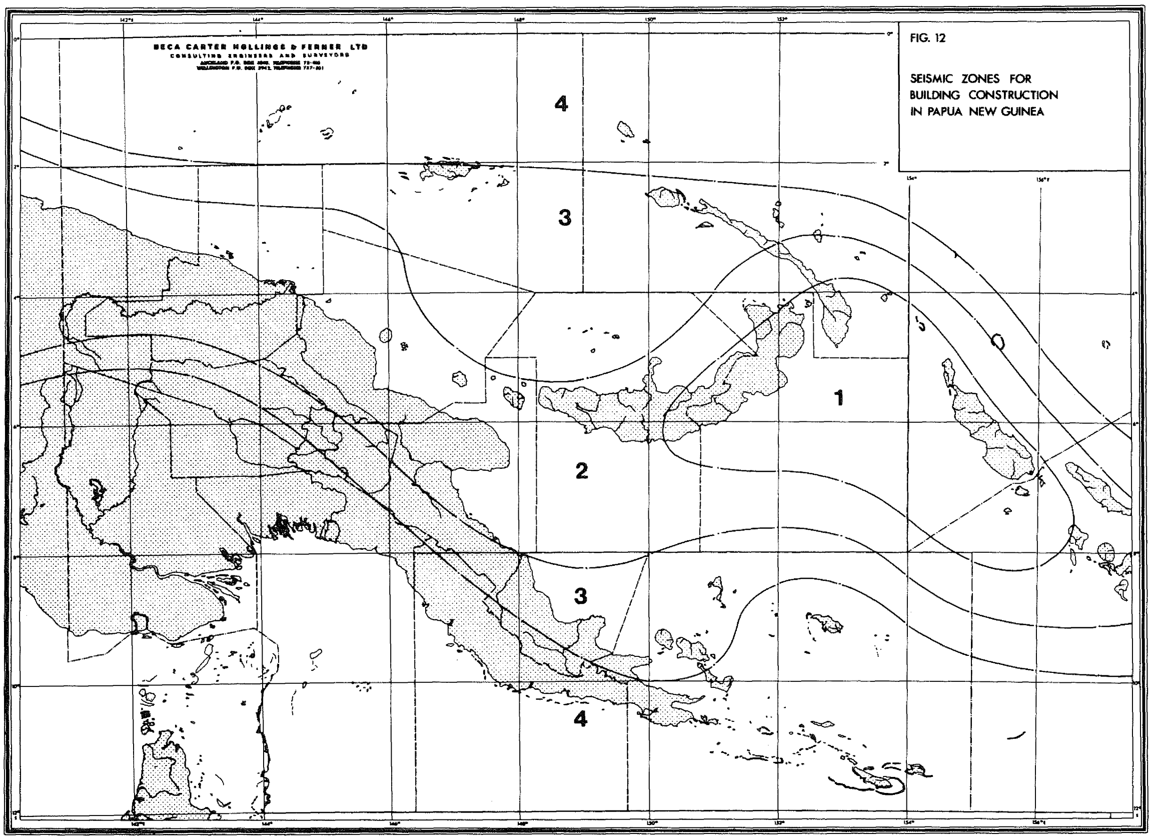


FIG. 12

SEISMIC ZONES FOR BUILDING CONSTRUCTION IN PAPUA NEW GUINEA included in codes of practice in the United States and in New Zealand in the 1930's following damaging earthquakes at Santa Barbara (California) in 1925 and Murchison and Napier (New Zealand) in 1929 and 1931. The loadings were defined as CW where W was the weight of the structure (including some live load) and C was a coefficient normally less than 0.10.

Strong-motions recorded in several moderately large earthquakes (Long Beach 1933, Imperial Valley 1940, Kern County 1952) encouraged considerable research into earthquakes and their effects on buildings, and in the late 1950's the Structural Engineers Association of California (SEAOC) published a detailed seismic design code[2]. The code specified that the lateral earthquake load should be KWC; where C was now dependent on building period, and K, the structural type factor varied from 2/3rd's to 1-1/3rd depending on the type and arrangement of the resisting elements. K was introduced to reflect the historical experience that some building systems performed better than others in earthquakes. In 1961 a zoning factor was included and the code was adopted in the American Uniform Building Regulations.

An important text by Blume Newmark and Corning[3] on the design of multi-storey concrete buildings was published in 1961. This book demonstrated that buildings could be subjected to loads considerably in excess of those prescribed in the code. To ensure survival of the structure when subject to these very severe earthquakes it was necessary to carefully detail the areas in which damage would occur and to dissipate seismic energy while still maintaining the vertical and lateral load carrying capacity. This ability of a structure was referred to as ductility.

Code changes since 1961 have been directed towards the incorporation of adequate ductility to ensure survival of structures in large earthquakes. Recent examples of such changes are the 1975 amendments to SEAOC[4] and the subsequent publication of ATC-3[5], and the 1976 revisions to the seismic provisions in the New Zealand loadings cods NZS 4203[6].

6.2 State of the Art

A brief review of present practices and procedures in determining lateral loadings for the seismic design of buildings is presented in this section. Section (a) outlines the basis of several widely used codes for earthquake resistant design. Section (b) discusses the dynamic response of simple systems and the determination of inelastic seismic response.

(a) Basis of Code Provisions

Statements of principles in earthquake resistant design regulations are invariably general in nature. Three examples are given below:

"The earthquake provisions of this standard are based on the assumption that during its lifetime a building in zone A will probably experience (a) one or more earthquakes of high intensity and long duration, and (b) several earthquakes of moderate intensity and duration. It is further assumed that probability of such occurrences is less in zone B than zone A, and less in zone C than zone B".

(NZS 4203:1976[6])

" ... structures designed in conformance with the provisions and principles set forth herein should, in general be able to:

1. *Resist minor earthquakes without damage;*
2. *Resist moderate earthquakes without structural damage, but with some non-structural damage;*
3. *Resist major earthquakes, of the intensity of the strongest experiences in California, without collapse, but with some structural as well as non-structural damage".*

(SEAOC, 1975[4])

" ... structures designed in accordance with (the code's) provisions should be able to resist minor earthquakes without damage and resist catastrophic earthquakes without collapse".

(NBC of Canada, 1970[7])

It is generally accepted that it is not economic to design a structure to perform elastically (and therefore remain undamaged) when subjected to the worst earthquake expected at its site. The basis for modern earthquake resistant design is therefore a two stage process, the objective of which can be summarised as follows:

1. Provide the structure with sufficient strength and stiffness to resist moderate earthquakes so that the frequency of occurrence of structural and non-structural damage is acceptably low.
2. Ensure that the probability of collapse of the structure in its useful life in a severe earthquake is acceptably low.

A moderate earthquake can be defined as one which has a high probability (say 60%) of occurrence in the assumed life of a structure while a severe earthquake can be defined as one which has a low probability (say less than 15%) of occurrence in the same period.

6.3 Recommended Method for Determining Code Design Loads

The design lateral load can be simply defined by the formula;

$$V = C\ I\ K\ W_t$$

Where V is the total horizontal seismic base shear and W_t is the weight of the structure (including some live load),

C is the basic seismic coefficient which depends on the risk of potentially damaging ground motions at the site, the natural period of vibration of the structure and the stiffness of the foundation soils. (Objective 1)

I is the importance factor, which takes a value of 1.0 for typical structures. Higher values are used for important structures to provide a decrease in the probability of damage from the implied by the value of "C".

K, the structural type factor, is intended to reflect the potential performance of different types of structures and materials in severe earthquakes. Structures with substantial ductility and capable of dissipating energy at a substantial number of locations are assigned K = 1.0, and K increases as the available ductility decreases (Objective 2)

Values for C, I and K are developed in the next section.

7.0 DETERMINATION OF LATERAL DESIGN LOADINGS FOR NORMAL PAPUA NEW GUINEA STRUCTURES:

7.1 General

A description of the derivation of the values for the lateral design load coefficient (C), the importance factor (I), and the structural type factor (K) is given in this Section.

The values were based on an acceptable average return period for the onset of structural damage of 20 years. It must be noted that the level of "damage" expected in the event with an average return period of 20 years is very minor. It is an onset or threshold level after which there would be just perceptable permanent effects - say minor cracking of no structural significance. Also because of the number of necessary assumptions throughout the load calculation and the tendency for these to be conservative, it is likely that the loads will lead to a greater average return period than 20 years for this onset level. The choice of the 20 year return period is thus considered to be realistic for Papua New Guinea. Lateral loads derived for this return period tend to be very similar to those derived for equivalent zones in existing documents[11]. Clearly, if it was deemed that this particular level of risk was inappropriate the factors could be adjusted. For an assumed life of a structure of 30 years earthquake motions of the order referred to above have a probability of approximately 80% of occurring in the life of the structure. This is consistent with the definition of a moderate earthquake.

It should be noted that the 20 year return period is for those structural types having a K factor of one. These are types having the minimum permitted total base shear for design. For other structures with a K factor greater than one, the return period of damage will be greater than 20 years. For example, a K factor of 2 changes the expected return period of onset of structural damage for that structural type to approximately 100 years.

7.2 The Frequency of Occurrence of Potentially Damaging Ground Motions in Papua New Guinea

The expected average frequency of occurrence of specified levels of 5% damped, horizontal acceleration response spectra have been presented in Section 4 as Figures 8, 9, 10 and 11. The adopted zoning map is given in Figure 13.

Average spectra for soft and hard ground conditions for varying return periods of onset of structural damage in structures with K factor one, have been calculated for each seismic zone, ground condition and period range. These spectra are illustrated in Figure 14.

7.3 Determination of the Lateral Design Load Coefficient (C)

The purpose of C in the loading expression (see Section 6.3) is to ensure that the return period for the onset of structural damage in structures of K factor one, is acceptable. C is therefore dependent on the expected frequency of occurrence of potentially damaging ground motions, and will differ between seismic zones as illustrated in Figure 14. C is also dependent on the natural period of vibration of the structure and on site conditions.

The acceleration response (in g's) associated with the onset of structural damage in typical Papua New Guinea structures designed with a lateral design load coefficient C can be represented by $f_1 \times f_2 \times C^*$.

Where f_1 is an additional strength factor which is an estimate of the ratio of the specified lateral strength and the actual lateral strength at first yield, and f_2 is a factor which takes account of the increase in lateral deflection and lateral load beyond first yield to cause inelastic deformation in a significant number of primary structural elements.

Although the respective contributions of each factor may vary between building types a combined value appears to fluctuate less. A combined $(f_2 \times f_1) = 3.0$ appears representative, although this may be considered too high for structures with few energy dissipating regions.

The spectra for soft and hard ground with the return period of 20 years have been divided by $(f_1 \times f_2) = 3.0$ to give C values for each zone and structural period combination, and these are illustrated in Figure 15.

Note that the 20 year return period is an indicative period. The return period of the response taken as representative for each zone with the exception of zone 4 has been assessed as varying between approximately 15 and 30 years at the most and least seismic boundaries of the zones. It is also related to peak structural response which is achieved in only some structures and for a very short period of time.

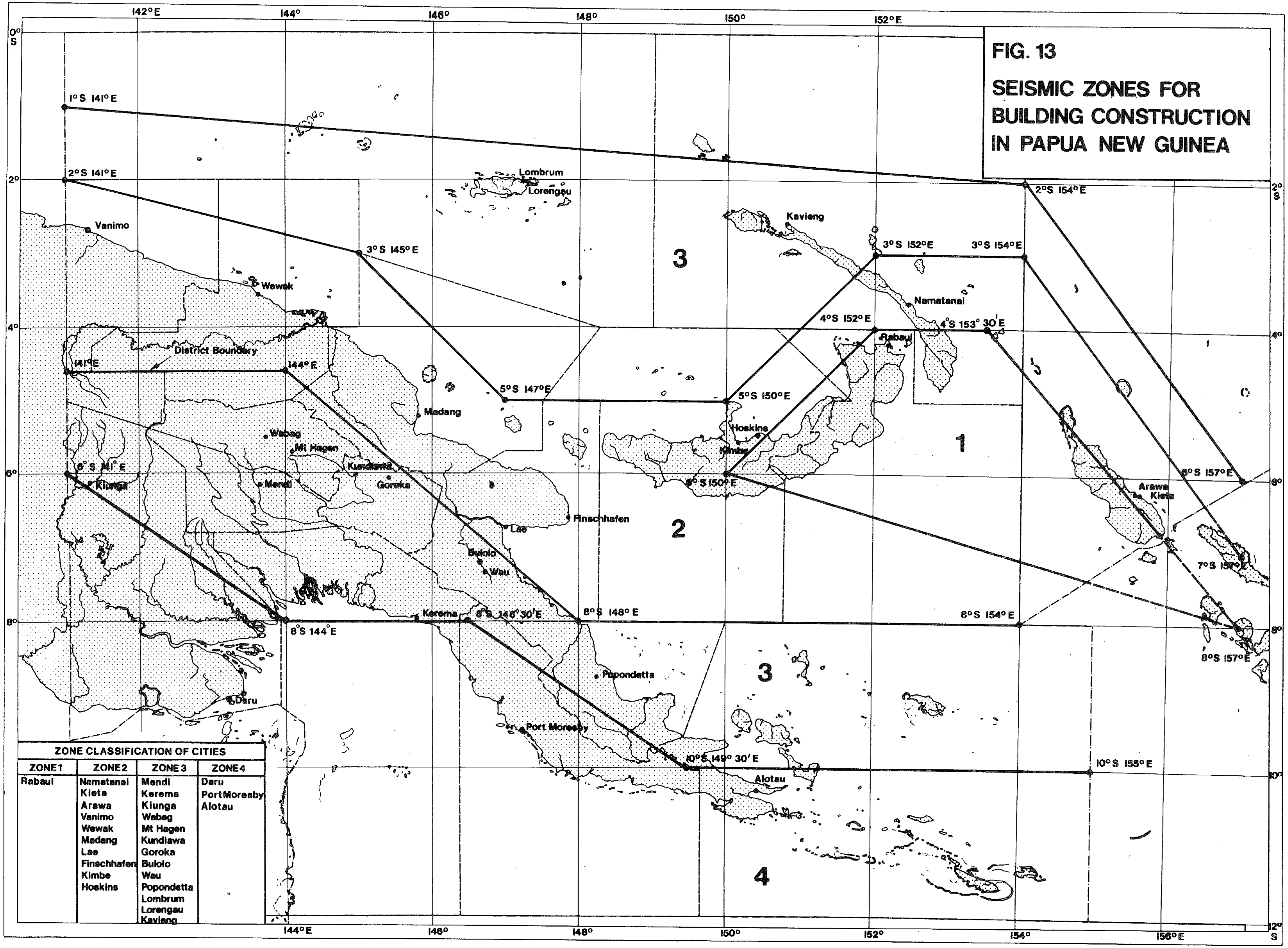
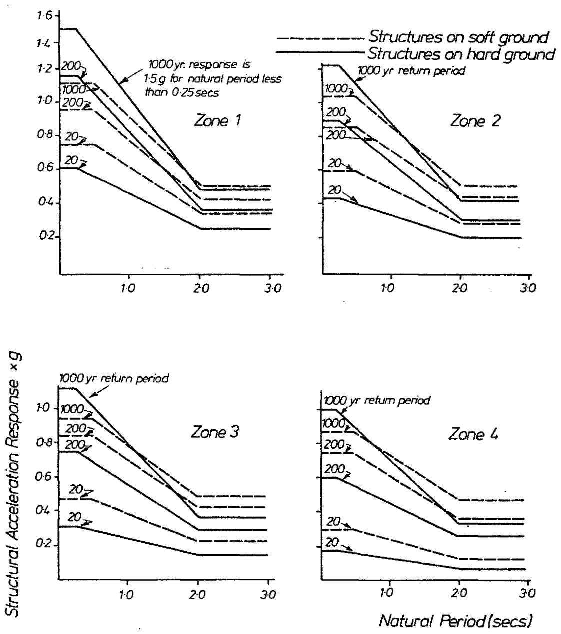


FIG. 13

SEISMIC ZONES FOR BUILDING CONSTRUCTION IN PAPUA NEW GUINEA

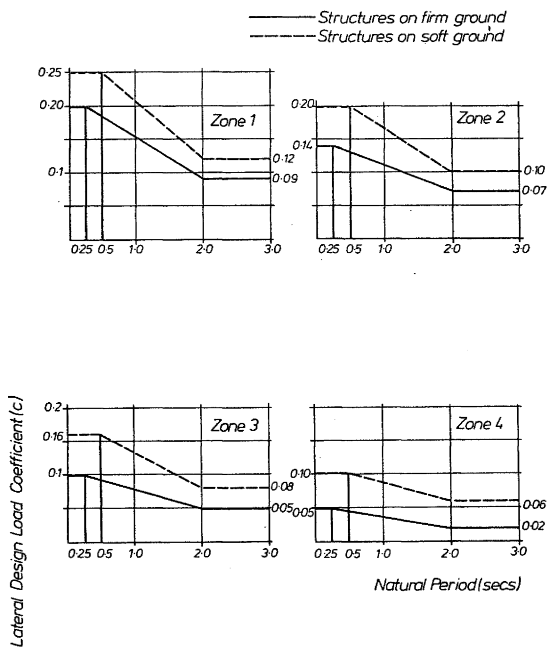
ZONE CLASSIFICATION OF CITIES

ZONE 1	ZONE 2	ZONE 3	ZONE 4
Rabaul	Namatanai	Mendi	Daru
	Kieta	Kerema	Port Moresby
	Arawa	Kiunga	Alotau
	Vanimo	Wabag	
	Wewak	Mt Hagen	
	Madang	Kundiawa	
	Lae	Goroka	
	Finschhafen	Bulolo	
	Kimbe	Wau	
	Hoskins	Popondetta	
		Lombrum	
		Lorengau	
		Kavieng	



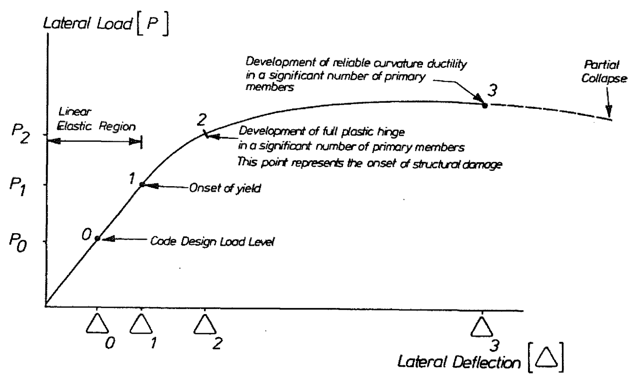
STRUCTURAL RESPONSE SPECTRA FOR VARYING RETURN PERIODS OF STRUCTURAL DAMAGE IN STRUCTURES WITH K FACTOR ONE

FIGURE 14



BASIC SEISMIC COEFFICIENT 'C'

FIGURE 15



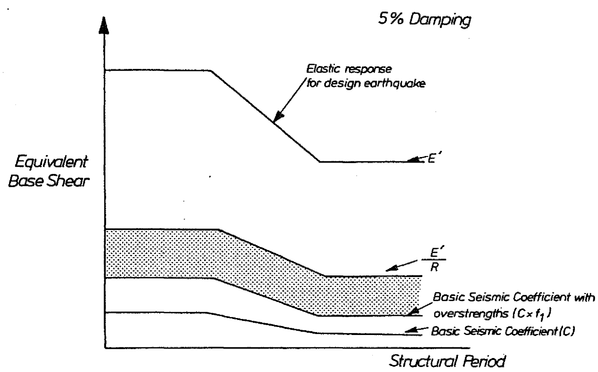
$$f_1 = \frac{P_1}{P_0} = \frac{\Delta_1}{\Delta_0}$$

$$f_2 = \frac{\Delta_2}{\Delta_1}$$

$$\text{Ductility } \mu_a = \frac{\Delta_3}{\Delta_1}$$

SIGNIFICANT STAGES ON LOAD-DEFLECTION CURVE FOR BUILDINGS

FIGURE 16



K is defined as $\frac{E'}{R} \div C \times f_1 = \frac{E'}{C \times f_1 \times R}$

ILLUSTRATION OF DERIVATION OF K

FIGURE 17

7.4 Determination of Importance Factor (I)

The desired level of earthquake protection will vary depending on the importance of the facility and the consequences of failure. The importance factor, I, is used to increase the lateral design load to achieve the required degree of acceptable risk. For normal structures I is 1.0.

To illustrate an increase in C from .10 to .12 for zone 4 in Papua New Guinea corresponds to an increase in return period from 20 to 30 years for stiff structures on soft ground. Similar calculations for other return periods, soil conditions and seismic zones lead to generalised values for I.

Change in return period	Importance Factor (I)
.67 times	0.8
1.5 times	1.3
2 times	1.5
5 times	2.0

Strictly, for a given percentage change, I depends on the seismicity, the natural period and on site conditions, but the variation does not appear to be significant and an average value was adopted.

NOTE: I and K are assumed to be 1.0.

7.5 Effect of Building Materials

The various building materials used in construction will tend to exhibit different properties when subject to earthquake loads. One of these properties affecting the risk will be the level of viscous damping in the elastic structure. As discussed in section 3.1, 5% damping was assumed in the derivation of the 'C' factor. If, for example, the damping in the structure is only 2%, then the frequency of occurrence of damage will be greater because the design load level would have been under-estimated. The reverse is also true. The expected frequency of occurrence of damage in a structure with a level of damping exceeding 5% would be less than that assumed in the derivation of the design loads. However the change in design load level for changes in damping with different materials is not considered significant enough to warrant introducing a further parameter into the method for seismic load assessment. This effect was however considered qualitatively in the determination of the K factor.

The building materials used will also have an effect on the inelastic dynamic properties of a structure. The materials affect the ductility available (refer section 7.6 (c)) and after damage has occurred the damping will tend to increase which will reduce the response of the structure. This increase in damping is of particular significance in ensuring the survival of structures of low ductility and has been considered in preparation of table 1.

7.6 DETERMINATION OF THE STRUCTURAL TYPE FACTOR (K)

(a) Introduction

The purpose of the structural type factor in the loading expression (see section 6.3) is to take account of the expected difference in performance of varying structure types and materials in severe earthquakes. The K factor provides sufficient laterial strength to ensure that demands do not exceed available ductility in severe earthquakes. Structures with substantial ductility and which are capable of dissipating energy in a significant number of members will have low K factors. Structures which have few energy dissipating mechanisms (for example masonry structures and cantilever type structures) need to have higher lateral strength to provide adequate protection in severs shaking. These structures therefore have higher K factors.

K can be seen to be dependent on both the configuration of the structure and the material of construction.

In the following sections K is calculated for the major structural types covered by the PNG code recommendation. In order to qualify for a given K factor this recommendation requires that structures meet certain structural type restrictions. These relate to member layout etc.

(b) The Design Earthquake Response (E')

It was proposed that the design earthquake response, which is the maximum intensity of shaking considered in the design of the structure, was that which has a low probability (say 10-15%) of occurring in the assumed structure life. Buildings should survive the design earthquake response without the ductility demand on the primary structural elements exceeding that.

Thus a structure with an assumed structure life (alternatively called the design life) of 30 years would be designed to withstand without available ductility being exceeded, a design earthquake response which had a return period of not less than 200 years. This does not necessarily mean collapse for larger earthquakes than the design earthquake - it does mean a greater demand for ductility than that which can be relied on. This greater demand may well be met in many structures.

(c) Structure Ductility Factors

The available structure ductility (denoted μ_a) can be assessed for structures depending on the structural configuration and the amount of ductility incorporated in the structural elements and joints. Representative values for μ_a for the major structural types covered by the PNG code recommendations are given in the fifth column of Table 1.

The detail provisions for ductile frames are expected to ensure values of 6½ or more for μ_a. At the lower end of the scale elevated tanks where diagonal bracing resists the lateral loads are assigned values of μ_a = 2½.

Figure 16 illustrates the load deflection curve for a typical building. The curve is significantly different to the ideal bilinear curve discussed in section 6.2 when procedures for evaluating the reduction factor R were outlined. For buildings the structure ductility factor μ_a can be assumed to apply to the curve between points 1 and 3. Thus μ_a can be defined as the ratio of the lateral deflections at the points 3 and 1.

(d) Method of Determination of Structural Type Factor K

The determination of the structural type factor 'K' required a great deal of judgement. Aspects considered were:

a. the effect of the building material

b. the structural type

c. any restrictions such as allowable height and detailing requirements

The procedure, which is tabulated in Table 1, may be summarised as follows:

(i) List μ_a factors

(ii) Assess the likely average natural period of vibration of each structural type

(iii) Assess the reduction factors. Typically the reduction factors may be taken as $R(\mu_a) = \sqrt{2\mu_a - 1}$ for natural periods less than 0.5 seconds and $R(\mu_a) = \mu_a$ elsewhere (see section 6.2). However it will be realised that the division between the use of $\sqrt{2\mu_a - 1}$ and μ_a is not as distinct as the above statement would imply. For structural periods close to 0.5 seconds therefore a value of $R(\mu_a)$ between the two limits is more applicable. An increase in damping with increasing damage will also increase the effective reduction factor. This is because the elastic response to the design earthquake will reduce with increased damping. For simplicity however this effect has been included in the assessment of $R(\mu_a)$ rather than in reducing the level of the design earthquake response. The sixth column in Table 1 gives a qualitative indication of the assumed effect of increased damping for each structural and material type.

(iv) Reduce the elastic response of the design earthquake by dividing by R and multiply the lateral design load co-efficient, by the structure additional strength factor f_1 (see section 7.3).

(v) If the required reduced response level $\frac{E'}{R}$ is more than the real strength $f_1 \times C$ provided, a K factor is required to make up the difference.

i.e. $K = \frac{E'}{R} \times \frac{1}{f_1 \times C} = \frac{E'}{f_1 \times C \times R}$

This is illustrated in figure 17.

In this way the value of K can be directly related to the design earthquake response. This formula can be rearranged to give $E' = K \times f_1 x C x R$. A summary of the calculation of K factors for zone 4 is presented in Table 1. The calculation was based on the design earthquake response for zone 4 as it is within this zone that the Capital of PNG; Port Moresby is located. Rather than recalculate the K factors for each zone it was decided to use the value for zone 4. A check was carried out to ensure that the implied return period for the design earthquake response in zones 1 to 3 was greater than or equal to 200 years. In each case the return period was found to be greater than 1000 years or a probability of occurrence in the 30 year life of a structure of less than 3%. This low occurrence was accepted in zones 1 to 3 rather than raise the loads in zone 4.

8.0 CONCLUDING COMMENTS:

The authors believe that the procedures used in this study provide a logical method of Seismic Zoning and lateral load determination consistent with modern earthquake design philosophy; namely designing to ensure that the risk of damage under moderate earthquakes is acceptable and then checking to ensure that the risk of collapse under severe earthquakes is also acceptable. Simplified (cook-book type) approaches to seismic risk assessment should however be treated with caution. Earthquake occurrence data is usually only available for a relatively short historical period and it is not possible to confidently state a clear relationship between the various earthquake parameters (magnitude, distance, depth, duration, geology of epicentre etc) and structural response. Results must therefore be mediated with reason and caution to ensure that suggested load levels reflect the past performances of structures.

9.0 ACKNOWLEDGEMENTS:

The assistance of the Director of Bureau of Mineral Resources, Canberra in making available the Bureau's extensive list of earthquakes for this study is gratefully acknowledged.

The assistance of the staff of the Geological Survey Division of the Papua New Guinea Department of Minerals and Energy is also gratefully acknowledged.

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1. Bentley, R.J. Average Estimates of the Attenuation with Distance of 5% Damped Horizontal Acceleration Response Spectra. Proc. Second South Pacific Regional Conference on Earthquake Engineering, May 1979.

ITEM NO FROM FIG 3-4 ANNEXURE 1 PART 4	STRUCTURE TYPE (1)	CONSTRUCTION MATERIAL OF THE SEISMIC ENERGY DISSIPATING ELEMENTS	ASSUMED AVERAGE NATURAL PERIOD OF VIBRATION (sec)	μ_a	LEVEL OF DAMPING INCREASE LIKELY DUE TO DAMAGE (2)	REPRESENTATIVE R	$R \times f_1 \times C$ Hard Ground Zone 4	ZONE 4 DESIGN EARTHQUAKE RESPONSE (200 year Return Period) Hard Ground	K(4)	IMPLIED RETURN PERIOD FOR DESIGN EARTHQUAKE RESPONSE, E' i.e. $E' = k \times f_1 \times C \times R$ (Hard Ground) (5) Zones 1 to 3
1	DUCTILE MOMENT RESISTING FRAME COMPLYING WITH CLAUSE 3.4.4.1 ANNEXURE ONE - Part 4	(i) Reinforced Concrete	0.8	6	M	6½	0.44	0.5	≈1.0	> 1000yr
		(ii) Steel	0.8	6½	L	6½	0.44	0.5	≈1.0	> 1000yr
		(iii) Timber*								
2	DUCTILE CANTI-LEVER SHEAR WALL COMPLYING WITH CLAUSE 3.4.4.2 ANNEXURE ONE - Part 4	(i) Reinforced Concrete	0.7	6	L	6	0.41	0.52	≈1.2	> 1000yr
		(ii) Timber*								
		(iii) Reinforced Masonry	0.5	4	M	4½	0.31	0.52	≈1.5	> 1000yr
3	DUCTILE COUPLED SHEAR WALL COMPLYING WITH CLAUSE 3.4.4.3 ANNEXURE ONE - Part 4	(i) Reinforced Concrete	0.8	6	M	6½	0.44	0.5	≈1.0	> 1000yr
4	CANTILEVER SHEAR WALL OF LIMITED DUCTILITY COMPLY-ING WITH CLAUSE 3.4.4.4 ANNEXURE ONE - Part 4	(i) Reinforced Concrete	< 0.25	4	H	4½ (6)	0.38	0.6	≈1.5	> 1000yr
		(ii) Timber*								
		(iii) Reinforced Masonry	< 0.25	3	H	3½ (6)	0.30	0.6	≈2.0	> 1000yr
5	CANTILEVER SINGLE STOREY FRAME COMPLYING WITH CLAUSE 3.4.4.5 ANNEXURE ONE - Part 4	(i) Reinforced Concrete	0.5	6½	L	3½	0.27	0.55	≈2.0	> 1000yr
		(ii) Steel	0.5	6½	L	3½	0.27	0.55	≈2.0	> 1000yr
6	DIAGONALLY BRACED FRAME COMPLYING WITH CLAUSE 3.4.4.6 ANNEXURE ONE - Part 4	(i) Reinforced Concrete	< 0.25	4	M	3	0.26	0.6	≈2.5	> 1000yr
		(ii) Steel	< 0.25	4	L	3	0.26	0.6	≈2.5	> 1000yr
		(iii) Timber*								
7	CABLE STAYED CHIMNEYS AND ELEVATED TANKS ON 4 OR MORE DIAGONALLY BRACED LEGS COMPLYING WITH CLAUSE 3.4.4.7 ANNEXURE ONE - Part 4	(i) Reinforced Concrete	< 0.25	2	H	2½ (7)	0.21	0.6	≈3.0	> 1000yr
		(ii) Steel	< 0.25	2	M	2½ (7)	0.21	0.6	≈3.0	> 1000yr

NOTES:

1. Refer Annexure One - Part 4 for STRUCTURE TYPES.
2. The level of viscous damping increase due to damage has been assessed as follows:

   L - small increase in viscous damping with damage

   M - medium increase in viscous damping with damage of approximately 30 → 70%

   H - high increase in viscous damping with damage of in excess of 70%

   (These increases are as recommended in reference 14).
3. f_1 is assumed to be 1.7
4. The calculation sequence $\left(K = \dfrac{E'}{R \times C \times f_1}\right)$ leads to K factors approximately equal to those given.
5. High return periods implied for the design earthquake response are likely where the zone is close to a region of high seismic activity. Therefore the seismic design load level (return period equal to 20 years) is high compared with that with a very large return period.
6. It has been assumed for cantilever shear walls of limited ductility that severe damage may occur at levels of deformation not necessarily involving yielding of the reinforcement. Thus the resulting increase in damping would tend to have more effect in terms of load limitation.
7. It has been assumed that seismic loads in cable stayed structures will be limited principally by damping.

* denotes items referring to similar constructions in timber. Design requirements for timber were not available at the time of preparation of this table and therefore appropriate K factors could not be assessed.

TABLE 1. CALCULATION OF K VALUES

(Refer section 7.6(d) for explanation)

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15. Papua New Guinea Standard "Code of Practice for General Structural Design and Design Loadings for Buildings - Part 4 Earthquake Loadings" (in preparation).