EARTHQUAKE RESISTANCE OF BUILDINGS AND MARINE RECLAMATION FILLS IN HONG KONG

GEO REPORT No. 16

W.K. Pun

GEOTECHNICAL ENGINEERING OFFICE
CIVIL ENGINEERING DEPARTMENT
HONG KONG

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PREFACE

In keeping with our policy of releasing information of general technical interest, we make available some of our internal reports in a series of publications termed the GEO Report series. The reports in this series, of which this is one, are selected from a wide range of reports produced by the staff of the Office and our consultants.

Copies of GEO Reports have previously been made available free of charge in limited numbers. The demand for the reports in this series has increased greatly, necessitating new arrangements for supply. In future a charge will be made to cover the cost of printing.

The Geotechnical Engineering Office also publishes guidance documents and presents the results of research work of general interest in GEO Publications. These publications and the GEO Reports are disseminated through the Government's Information Services Department. Information on how to purchase them is given on the last page of this report.

A. W. Malone
Principal Government Geotechnical Engineer
April 1995

EXPLANATORY NOTE

This GEO Report consists of two Technical Notes on the resistance of engineering structures in Hong Kong to earthquakes.

They are presented in two separate sections in this Report. Their titles are as follows :

SECTION 1 : A NOTE ON LIQUEFACTION OF MARINE RECLAMATION FILLS

W.K. Pun

This report was originally produced as GCO Technical Note No. TN 5/91

FOREWORD

This note documents a brief review of literature relating to liquefaction of marine reclamation fills due to seismic loading and vibrations from other sources. It is the first step of a research on the mechanics of hydraulic fills being carried out by the Special Projects Division. The contents herein will serve as useful reference for the research project.

This note was prepared by Mr W.K. Pun under the supervision of Dr P.L.R. Pang. Mr P.W.T. To provided useful information and suggestions during the course of this work. Dr J. Premchitt made valuable comments on a draft version of the note.

(Y.C. CHAN)
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CONTENTS

1. INTRODUCTION

Due to the anticipated large amount of marine reclamation for PADs projects, the Special Projects Division has been assigned a task of reviewing the properties of marine reclamation fills. This note is prepared to assist in this work.

In this note, the phenomenon of liquefaction is discussed and the relevant properties of marine sand fills are described. The results of a preliminary assessment of the liquefaction potential of the marine sand fill at two sites at the Kai Tak Airport are presented. A bibliography of selected useful references is given at the end of the note.

For the purpose of this note, the term 'marine reclamation fills' shall mean fills placed in the sea to form land.

2. LIQUEFACTION

2.1 Definition

According to Seed et al (1971), the term 'liquefaction' was originally introduced to describe failures in soil slopes in which the slope material fluidises and flows through a large distance. Later, the phenomenon of 'boiling' and settlement of level ground in sands during earthquakes are also referred to as liquefaction (of the sands in the ground). Castro (1975) distinguished between the former and latter phenomena as 'liquefaction' and 'cyclic mobility' respectively.

In this note, the broader sense of the term 'liquefaction' is adopted. However, the discussions made are specific to level ground subjected to cyclic loading due to earthquake and other dynamic loads. The subject of liquefaction of soil slopes is not covered.

2.2 The Phenomenon of Liquefaction

When a sand mass in a loose condition is subjected to a strong cyclic loading, its volume will decrease. If the sand is saturated and if drainage is unable to occur fast enough, there will be a progressive increase in the pore water pressure. If the pore water pressure builds up to a level equal to the soil confining pressure, the effective stress in the soil will become zero. The sand, having lost its strength, is said to have liquefied.

Upon liquefaction, water will be squeezed out of the soil, resulting in settlement of the ground. Sometimes, sand particles may be squeezed out as well, through cracks in the overlying soil layers, and sand 'volcanos' or 'boiling' can be observed. If a building is supported on footings or piles founded within the liquefied soil mass, foundation failure will occur. For buildings supported on piles which have been taken down to a founding material below the liquefied soil, negative skin friction will be exerted on the piles. Liquefaction of soil behind a seawall causes an increase in earth pressure on the seawall due to the temporary loss in soil shear strength. This may bring about collapse of the seawall.

The prediction of ground settlement caused by liquefaction is a very difficult and complex problem that is still far from being resolved (Seed, 1987). Much research work is being done on this subject. Tokimatsu & Seed (1987) proposed a simple chart for predicting settlement due to liquefaction from SPT values. However, the chart is based on limited data and should only be considered as tentative. Rosidi & Wigginton (1991) used the Tokimatsu & Seed chart to estimate settlement caused by the 1989 Loma Prieta earthquake. They found that the observed settlement in the sand fill was larger than that calculated.

The evaluation of negative skin friction on piles and of increased earth pressure on seawalls resulting from liquefaction are also fraught with difficulties. Therefore, the current practice is to carry out precautionary works (e.g. deep compaction) to remove the problem when an assessment of liquefaction potential indicates the likelihood of liquefaction.

2.3 Factors Influencing Liquefaction Potential

2.3.1 General

The following factors can influence the liquefaction potential of a soil mass :

(a) source of vibration,
(b) soil type,
(c) relative density or void ratio,
(d) confining pressure, and
(e) position of water table.

These factors are briefly discussed below.

2.3.2 Source of Vibration

In theory, vibratory loading induced by any means can trigger liquefaction of a soil mass under unfavourable conditions. However, for liquefaction to occur, there needs to be a sufficient amount of energy to bring about the collapse of the soil structure. The energy involved in the operation of vibratory construction equipment is far too small to be able to induce extensive liquefaction of a soil mass. Vibrations resulting from pile driving is also of relatively small magnitudes. Increase in pore water pressure during pile driving may render the soil around the pile to lose resistance temporarily. However, the bearing capacity of the pile will increase again soon after piling stops. Any soil densification which may occur during pile driving will be localised. Local liquefaction may also be induced by cyclic loadings from initiators such as sea waves (on offshore structures), transient traffic and machinery.

Blasting and earthquakes are both capable of triggering liquefaction if there is sufficient energy. However the area affected by blasting is much smaller than that affected by earthquakes. For a given soil mass, the vulnerability to liquefaction during an earthquake or a blasting operation depends on the intensity of ground shaking, the duration of the shaking, and the frequency of the seismic waves generated. It can be expected that the stronger the ground shaking intensity and the longer the shaking duration, the more likely it is for liquefaction to occur. It is known that low frequency (less than about 50 Hz) seismic waves are more critical than high frequency waves. For the purpose of assessing liquefaction potential, the degree of ground shaking is usually expressed in terms of the ground acceleration.

2.3.3 Soil Type

The particle size distribution of the soil has a pronounced influence on its liquefaction potential. Uniformly-graded materials are more susceptible to liquefaction than well-graded materials and fine clean sands tend to liquefy more easily than coarse sands, gravelly soils, silts or clays. In some of the methods used for assessing liquefaction potential, correction factors are applied to account for the effects of the presence of fines (e.g. Ambraseys, 1988; Chinese code, 1989; Seed, 1987; Seed et al, 1983). Tokimatsu & Yoshimi (1983) went as far as suggesting that soils containing more than 20 percent by weight of particles finer than 0.005 mm could hardly liquefy unless their plasticity indices are low. They also found that sands containing more than 10 percent by weight of particles finer than 0.074 mm have a much greater resistance to liquefaction than clean sands having the same SPT N-values.

There are very few reported cases of liquefaction in gravelly soils. For the few cases where liquefaction has occurred, the gravelly soils involved are very loose and contain a substantial amount (over 40%) of particles of sand and smaller sizes (Valera & Kaneshiro, 1991). Wong et al (1975) postulated that the primary reason for the much lower susceptibility of gravelly soils to liquefaction is their ability to dissipate pore water pressure induced by ground shaking.

2.3.4 Relative Density or Void Ratio

Experience indicates that the susceptibility of a soil to liquefaction is determined to a high degree by the soil's relative density or void ratio. For example, during the Niigata earthquake in Japan in 1964, extensive liquefaction occurred in areas of sands with a relative density of about 50%. However, there was no signs of liquefaction in sand areas where the relative density exceeded about 70% (Seed & Idriss, 1967). SPT N-values, CPT values and the 'state parameter' are used by various investigators to characterise the density state of a soil mass in liquefaction potential assessments.

2.3.5 Confining Pressure

There is considerable laboratory and field evidence which show that the liquefaction potential of a soil is reduced with increase in confining pressure. Liquefaction at depths greater than 15 metres is extremely rare.

2.3.6 Position of Water Table

By definition, dry soils will not liquefy although loose dry soils will compact when subjected to vibration.

The effective stress of a soil element depends on its position below the water table. The lower the effective stress, the more likely it is for the soil to liquefy (see Section 2.3.5). Therefore, ground with a high water table is more vulnerable to liquefaction than one with a low water table.

2.4 Methods for Assessing Liquefaction Potential

2.4.1 Types of Methods

As the more serious liquefaction incidents are mostly associated with earthquakes, the methods available for assessing liquefaction potential are mostly developed for earthquake-induced vibrations.

Basically, the methods can be divided into two categories :

(a) Methods based on the theoretical approach, in which the cyclic stresses or strains induced in a soil mass due to an earthquake are evaluated by theoretical analysis and compared with the resistance to liquefaction obtained from laboratory tests, or in which the induced pore water pressures in the soil mass computed from analysis are compared with the initial effective stresses in soil.

(b) Methods based on the empirical approach, which have been developed on the basis of results of characterisation of sites at which liquefaction had occurred, as well as sites which did not show signs of liquefaction during past earthquakes.

Examples of methods based on the theoretical approach are those of Seed & Idriss (1971), Liam Finn et al (1976) and Sladen et al (1985). Examples of methods based on the empirical approach are those of Seed et al (1983), Tokimatsu & Yoshimi (1983), Crooks et al (1985), Robertson & Campanella (1985), Seed et al (1985), Ambraseys (1988) and Reyna & Chameau (1991). Brief descriptions of the above methods are given in the next Section.

2.4.2 Brief Descriptions of Methods Available

(1) Seed & Idriss (1971). The average shear stress τ_{av} induced in a soil element due to an earthquake is calculated using the following equation :

$$\tau_{av} = 0.65\ \gamma h\ \frac{a_{max}}{g}\ r_d \qquad \ldots\ldots\ldots\ldots \quad (1)$$

where γ = unit weight of the soil
h = depth of soil element below ground
g = acceleration due to gravity
a_{max} = maximum acceleration at the ground surface induced by the earthquake
r_d = stress reduction factor to allow for the flexibility of soil

The coefficient 0.65 in the above equation is the ratio between the average and maximum ground accelerations experienced during the earthquake. A curve is given by Seed & Idriss for determining r_d, which depends on h : r_d equals 1.0 for h = 0, reducing to 0.8 for h = 15 m. In this method, the average shear stress calculated is compared with the shear resistance to liquefaction, which is obtained from cyclic triaxial compression tests.

This method was applied by Pyke et al (1978) to assess the liquefaction potential of hydraulically-placed fills.

(2) Liam Finn et al (1976). This is a theoretical method in which an effective stress soil model is used in an analysis that computes the changes in pore water pressures in a soil mass subjected to an earthquake motion. Liquefaction is assumed to occur at points where the calculated pore water pressure exceeds the initial effective stress.

(3) Seed et al (1983). The authors back-calculated the cyclic stress ratios (τ_{av}/σ_o') for a number of sites with known SPT profiles, using the following equation :

$$\frac{\tau_{av}}{\sigma_o'} = 0.65 \frac{a_{max}}{g} \frac{\sigma_o}{\sigma_o'} r_d \quad \ldots\ldots\ldots\ldots \quad (2)$$

where σ_o = total vertical stress at the depth of the soil element
σ_o' = effective vertical stress at the depth of the soil element

τ_{av}, a_{max} and g have the same meaning as those defined in equation (1) and r_d have the same values as proposed by Seed & Idriss (1971).

The authors then plotted the cyclic stress ratios against N_1 (the SPT values corrected to an overburden pressure of 1 ton/ft^2), and derived an empirical curve which separates the data points corresponding to sites where liquefaction had occurred from those where liquefaction had not occurred during past earthquakes. For a given cyclic stress ratio, the curve gives the critical N_1 value below which liquefaction will occur during an earthquake. This approach forms the basis of subsequent SPT-based methods for assessing liquefaction potential.

(4) Tokimatsu & Yoshimi (1983). In this method, the equivalent dynamic shear stress (ratio) is compared with the soil liquefaction resistance. The equivalent dynamic shear stress is essentially calculated using the same equation as that for cyclic stress ratio adopted by Seed et al (1983). The exception is that a correction factor of 0.1(M-1), where M is the earthquake magnitude, is used in equation (1) instead of the constant coefficient 0.65. Also, r_d = (1-0.015h) is adopted for the calculation in the same equation.

An empirical expression in term of SPT value has been developed for calculating the liquefaction resistance ratio (τ_1/σ_o'), where τ_1 is the shear resistance to liquefaction.

(5) Crooks et al (1985). A state parameter is defined as the void ratio difference between the current state and the 'steady state' of the soil at current stress level. The 'steady state' line is the locus of states at which a soil flows at constant void ratio, constant mean effective stress, constant shear stress and constant rate of shear strain. The void ratios at steady state are the same as those at critical state (Castro & Poulos, 1977; Poulos et al, 1985). A positive state parameter indicates that the soil is contractive and hence liquefaction is possible.

The state parameters of a soil mass are assessed by using CPT results. Therefore, in this approach, liquefaction potential is assessed using CPT values.

(6) Robertson & Campanella (1985). Using data from Seed et al (1983) and Vald et al (1981), and converting SPT and relative density values into CPT values by means of empirical formulae, the authors developed an empirical relationship between CPT and cyclic stress ratio. This relationship separates cases in which liquefaction had occurred from those in which liquefaction had not occurred during past earthquakes.

(7) Seed et al (1985). This is a modification of the SPT-based method given by Seed et al (1983). A standardised SPT value, $(N_1)_{60}$, was introduced to cater for the inconsistencies in SPT test procedures adopted in different countries. $(N_1)_{60}$ is the SPT N-value corrected to an overburden pressure of 1 ton/ft^2 and an energy ratio (defined as the ratio between the actual energy delivered to the drill rods and the theoretical free-fall energy of the falling hammer) of 60%. An empirical correlation between the cyclic stress ratio which can cause liquefaction and $(N_1)_{60}$ has been derived for a design earthquake magnitude of 7.5. Scaling factors are available for using the empirical correlation for other design earthquake magnitudes. The scaling factors are derived based on the number of seismic cycles representative of different magnitudes.

Seed et al had compared their method with the requirement for determining liquefiable sites given in the 1978 version of the Chinese Code and found reasonable agreement. The latter requirement has now been amended in the latest version of the Chinese Code (1989). However, as the basis of derivation of the code requirement cannot be ascertained, no further discussion on the Chinese Code will be given in this note.

(8) Sladen et al (1985). A 'collapse surface', which is the surface of peak shear strengths (obtained from undrained triaxial compression tests), has been defined in the three-dimensional void ratio-shear stress-normal stress space. A necessary condition for liquefaction to occur is that the soil state should reach the collapse surface during the period of ground shaking. This method requires a knowledge of the insitu soil state and the anticipated stress path under seismic loading.

(9) Ambraseys (1988). Ambraseys considered the use of scaling factors based on the number of equivalent uniform cycles representative of different earthquake magnitudes, as proposed by Seed et al (1985), to be inappropriate. He felt that it was unreasonable to allow the cyclic stress ratio for an earthquake of a given magnitude to vary in a deposit only with a fixed number of equivalent cycles. Hence, he carried out similar correlations as Seed et al to derive empirical curves for liquefaction potential assessment, except that different curves are derived for different earthquake magnitudes instead of a single 'master' curve. No scaling factor is thus required to assess the liquefaction potential of a site for a given earthquake magnitude. In this method, the $(N_1)_{60}$ values are corrected to an overburden pressure of 100 kPa.

(10) Reyna & Chameau (1991). An empirical relationship between DMT (Dilatometer values) and the cyclic stress ratio which can cause liquefaction has been developed. However, the database is limited.

2.4.3 Discussion

At present, laboratories in Hong Kong are not equipped with cyclic loading test equipment. Therefore, methods relying on such tests cannot be used in Hong Kong without arranging for specialist testing overseas or equipping local laboratories.

Liam Finn et al (1976)'s method is very theoretical. It requires sophisticated testing to determine the relevant parameters for the soil model. Equipment for carrying out such testing is not available in Hong Kong.

Sladen et al (1985)'s method requires many well-planned triaxial compression tests to define the 'collapse surface'. A lot of time and resources are required for carrying out these tests.

The insitu state parameters of a soil mass are generally evaluated from CPT results. Hence, it is more direct to use the CPT values for liquefaction potential assessment.

Empirical methods appear to be relatively simple to use. Amongst these, methods which are based on SPT N-values are considered more reliable than those based on other field tests, because of the more extensive database available. For SPT-based methods, those which use standardised N-values are more rational than others. The methods proposed by Seed et al (1985) and Ambraseys (1988) fall into this category. However, these methods require a comparison to be made between the SPT procedures currently adopted in Hong Kong and the 'standardised' procedures. This is to establish the correction factor required to be applied to convert Hong Kong SPT values to $(N_1)_{60}$ values.

3. PROPERTIES OF MARINE SAND FILLS

The liquefaction potential of a sand fill depends very much on its insitu density. The insitu density of a sand fill, in turn, is greatly affected by the placement method such as bottom dumping, pipeline discharge from the sea surface or pipeline discharge near the fill surface.

Sladen & Hewitt (1989) found that materials placed by bottom dumping were significantly denser than pipeline-placed materials, other factors being equal. They also mentioned that dense sand could not be obtained by hydraulic placement. At best, an average relative density of up to 60% can be achieved, but more generally, it will be less than 50%. Also, the range of relative density about the mean can be very large: the relative density within a given fill placed hydraulically can vary from about 10 to 70%.

Crooks (1985) characterised the insitu state of sand fills using the state parameter approach. He considered hydraulically-placed sand fill to be always dilatant (Chan, 1990). This is different from the experience of Sladen & Hewitt (1989).

It is understood that some local data on the properties of hydraulically-placed sand fill are available from the reclamation projects for Container Terminals 6 and 7. Unfortunately, it has not been able to obtain such data at the time of writing this report.

Some data on the properties of marine sand fills have also been acquired at two sites at the Kai Tak Airport (Leung, 1988; Leung & Yuen, 1988). These sites are located at the cargo and maintenance areas, which are believed to have been formed during the period between 1904 and 1924. The placement method of the marine sand fills at these sites is not certain.

Based on Leung (1988) and Leung & Yuen (1988), the marine sand fill at the Kai Tak Airport sites can be described as loose to very loose, grey to yellowish grey, fine to coarse sands with shell fragments. The amount of fines (particles < 0.063 mm) in the fills lies between nil and 10%. Above water, the SPT values of the sand fills vary from 3 to 10. Below water, the SPT values obtained are generally larger than 6, with localised areas having SPT values of 3 and 4.

4. LIQUEFACTION POTENTIAL ASSESSMENT

4.1 General

As discussed in Section 2.3.3, the liquefaction potential of a soil mass depends very much on the soil type. Experience indicates that reclamation fills comprising essentially uniform gravels, silts or clays are very unlikely to liquefy. CDG fills, which generally contain more than 20% by weight of fines (particles < 0.063 mm), are also not susceptible to liquefaction. Marine sand fills which comprise of relatively clean sand and gravelly soils with approximately over 40% sand particles deserve further consideration.

4.2 Preliminary Assessment of the Liquefaction Potential of Two Sites at the Kai Tak Airport

Based on the SPT values of the marine sand fills at two sites at the apron areas of the Kai Tak Airport, a preliminary assessment of the liquefaction potential of the sites has been made. The methods proposed by Seed et al (1985) and Ambraseys (1988) have been used for the assessment. Details of the assessment are given in Appendix A.

The assessment in Appendix A suggests that extensive liquefaction of the marine sand fills at the Kai Tak sites is very unlikely. However, liquefaction at local weak spots is possible. It should be noted that the ground acceleration at a site due to an earthquake is very sensitive to the natural period of the site, which in turn depends on the thickness of the superficial deposits. A higher ground acceleration will have to be adopted in the calculations if the thickness of the superficial deposits is less than that assumed. In that case, there will be a higher liquefaction potential. If the fill layer is thicker, the critical SPT values will also increase, indicating that there is a higher probability of liquefaction. A check should also be made to see whether the SPT values need to be corrected for the purpose of liquefaction potential assessment.

Despite the preliminary nature of the assessment in Appendix A, it is considered that the outcome would largely be the same even if input parameters are varied slightly. It should be noted that the assumption of 5% critical damping in the soil in the event of an earthquake is fairly conservative.

5. CONCLUSIONS

A review of existing literature indicates that the liquefaction potential of a soil mass depends on a number of factors, viz., the source of vibration, soil type, relative density, confining pressure and the initial pore water pressures in the soil. Experience show that reclamation fills comprising essentially of uniform gravels, silts or clays are unlikely to liquefy. Likewise, liquefaction of CDG fill is also very unlikely. Settlement induced by pile driving vibrations is generally not of concern. However, for marine reclamations formed from relatively clean sand fills, its liquefaction potential due to an earthquake warrants further study.

The prediction of settlement of a liquefied sand fill is a very difficult and complex problem, although the order of settlement may be estimated from a simple chart developed by Tokimatsu & Seed (1982). If the liquefaction potential of a sand fill is found to be high, it is suggested that suitable precautionary works be carried out to densify the fill. Alternatively, a minimum amount (say > 5%) of fines (particles < 0.063 mm) may be specified for the fills. However, this needs to be considered together with the method of placement of the fill, as the amount of fines may not be easily controlled. Also, the effect of the increase in consolidation settlement due to the presence of fines will need to be considered in design.

Many methods are available for assessing the liquefaction potential of sand fills due to earthquakes. Amongst these, two methods which are based on SPT $(N_1)_{60}$ values (viz. Seed et al, 1985 and Ambraseys, 1988) appear to be relatively simple to use and practical. However, a study should be conducted to assess the need to apply correction factors to convert Hong Kong SPT values to $(N_1)_{60}$ values. As to the CPT-based method, further investigation is required to establish a representative relationship between CPT values, liquefaction potential and ground acceleration.

Ground investigations carried out indicate that the marine sand fills at two sites at the cargo and maintenance apron areas of the Kai Tak Airport have low SPT values in localised areas. Nevertheless, most of the SPT values are generally higher than the critical $(N_1)_{60}$ values obtained from a preliminary assessment of liquefaction potential based on the methods of Seed et al (1985) and Ambraseys (1988). This indicates that although the fill materials at the Kai Tak Airport sites are vulnerable to liquefaction when subjected to strong ground motions, extensive occurrence is very unlikely. However, there is still a potential for localised liquefaction at weak spots.

Finally, further work that has been identified as worthwhile is summarized below :

(a) to collect data on the properties of marine sand fills in Hong Kong, including particle size distributions, insitu densities ,SPT values, CPT values and the construction history,

(b) to check whether there is a need to apply correction factors to convert Hong Kong SPT values to $(N_1)_{60}$ values, and

(c) to assess the liquefaction potential of sand fills at other sites.

6. BIBLIOGRAPHY

6.1 On the Phenomenon of Liquefaction

Been, K. & Jefferies, M.G. (1985). A state parameter for sands. Géotechnique, vol. 35, no. 2, pp 99-112.

Been, K., Conlin, B.H., Crooks, J.H.A., Fitzpatrick, S.W., Jefferies, M.G., Rogers, B.T. & Shinde, S. (1987). Back analysis of the Nerlerk berm liquefaction slides : discussion. Canadian Geotechnical Journal, vol. 24, pp 170-179.

Berrill, J.B. & Davis, R.O. (1985). Energy dissipation and seismic liquefaction of sands : revised model. Soils and Foundations, vol. 25, no. 2, pp 106-118.

Castro, G. (1975). Liquefaction and cyclic mobility of saturated sands. Journal of Geotechnical Engineering, American Society of Civil Engineers, vol. 101, no. GT6, pp 551-569.

Castro, G. & Poulos, S.J. (1977). Factors affecting liquefaction and cyclic mobility. Journal of Geotechnical Engineering, American Society of Civil Engineers, vol. 103, no. GT6, pp 501-516.

Green, P.A. & Ferguson, P.A.S. (1971). On liquefaction phenomena, by Professor A. Casagrande : report of lecture. Géotechnique, vol. 21, no. 3, pp 197-202.

Ishihara, K. & Takatsu, H. (1979). Effects of overconsolidation and Ko conditions on the liquefaction characteristics of sands. Soils and Foundations, vol. 19, no. 4, pp 59-68.

Martin, G.R., Liam Finn, W.D. & Seed, H.B. (1975). Fundamentals of liquefaction under cyclic loading. Journal of Geotechnical Engineering, American Society of Civil Engineers, vol. 101, no. GT5, pp 423-438.

Seed, H.B. & Idriss, I.M. (1967). Analysis of soil liquefaction : Niigata Earthquake. Journal of the Soil Mechanics and Foundation, American Society of Civil Engineers, vol. 93, no. SM3, pp 83-108.

6.2 On Methods for Assessing Liquefaction Potential

Ambraseys, N.N. (1988). Engineering seismology. Earthquake Engineering and Structural Dynamics, vol. 17, no. 1, pp 1-105.

Cao, Y.L. & Law, K.T. (1991). Energy approach for liquefaction of sandy and clayey silts. Proceedings of the Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, USA, vol. 1, pp 491-497.

Chinese Code (1989). Earthquake Resistant Design Code GBJ 11-89 (in Chinese). Chinese Standards Institution, PRC, 64 p.

Lavania, B.V.L., Mukerjee, S. & Sharma, J.N. (1991). Evaluation of liquefaction potential for an earth dam site. Proceedings of the Second International Conference on Recent Advances in Geotechnical Earthquake

Engineering and Soil Dynamics, St. Louis, USA, vol. 1, pp 445-450.

Law, K.T., Cao, Y.L. & He, G.N. (1990). An energy approach for assessing seismic liquefaction potential. Canadian Geotechnical Journal, vol. 27, pp 320-329.

Liam Finn, W.D. (1991). Assessment of liquefaction potential and post liquefaction behaviour of earth structures; developments 1981-1991. Proceedings of the Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, USA, vol. 2, pp 1833-1850.

Liam Finn, W.D., Byrne, P.M. & Martin, G.R. (1976). Seismic response and liquefaction of sands. Journal of Geotechnical Engineering, American Society of Civil Engineers, vol. 102, no. GT8, pp 841-855.

Liam Finn, W.D., Lee, K.W. & Martin, G.R. (1977). An effective stress model for liquefaction. Journal of Geotechnical Engineering, American Society of Civil Engineers, vol. 103, no. GT6, pp 517-532.

Martin, G.R., Tsai, C.F. & Armulmoli, K. (1991). A practical assessment of site liquefaction effects and remediation needs. Proceedings of the Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, USA, vol. 1, pp 411-418.

Poulos, S.J., Castro, G. & France, J.W. (1985). Liquefaction evaluation procedure. Journal of Geotechnical Engineering, American Society of Civil Engineers, vol. 111, no. 6, pp 772-793.

Poulos, S.J., Robinsky, E.J. & Keller, T.O. (1985). Liquefaction resistance of thickened tailings. Journal of Geotechnical Engineering, American Society of Civil Engineers, vol. 111, no. 12, pp 1380-1394.

Pyke, R.M., Knuppel, L.A. & Lee, K.L. (1978). Liquefaction potential of hydraulic fills. Journal of Geotechnical Engineering, American Society of Civil Engineers, vol. 104, no. GT11, pp 1335-1354.

Reyna, F. & Chameau, J.L. (1991). Dilatometer based liquefaction potential of site in the Imperial Valley. Proceedings of the Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, USA, vol. 1, pp 385-392.

Robertson, P.K. & Campanella, R.G. (1985). Liquefaction potential of sand using the CPT. Journal of Geotechnical Engineering, American Society of Civil Engineers, vol. 111, no. 3, pp 384-403.

Rosidi, D. & Wigginton, W.B. (1991). Liquefaction and surface settlement in the Marina District. Proceedings of the Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, USA, vol. 2, pp 1673-1677.

Saeed, I. (1991). Liquefaction potential of sand layers in foundations of an embankment dam. Proceedings of the Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, USA, vol. 1, pp 379-384.

Seed, H.B. (1987). Design problems in soil liquefaction. Journal of Geotechnical Engineering, American Society of Civil Engineers, vol. 113, no. 8, pp 827-845.

Seed, H.B. & Idriss, I.M. (1971). Simplified procedure for evaluating soil liquefaction potential. Journal of Soil Mechanics and Foundation, American Society of Civil Engineers, vol. 97, no. SM9, pp 1249-1273.

Seed, H.B., Idriss, I.M. & Arango, I. (1983). Evaluation of liquefaction potential using field performance data. Journal of Geotechnical Engineering, American Society of Civil Engineers, vol. 109, no. 3, pp 458-482. (Discussion, vol. 111, no. 11, pp 1343-1346).

Seed, H.B., Mori, K. & Chan, C.K. (1977). Influence of seismic history on liquefaction of sands. Journal of Geotechnical Engineering, American Society of Civil Engineers, vol. 103, no. GT4, pp 257-270.

Seed, H.B., Tokimatsu, K., Harder, L.F. & Chung, R.M. (1985). Influence of SPT procedures in soil liquefaction resistance evaluations. Journal of Geotechnical Engineering, American Society of Civil Engineers, vol. 111, no. 12, pp 1425-1445.

Seed, R.B., Dickenson, S.E. & Riemer, M.F. (1991). Liquefaction of soils in the 1989 Loma Prieta earthquake. Proceedings of the Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, USA, vol. 2, pp 1575-1586.

Sladen, J.A., D'Hollander, R.D. & Krahn, J. (1985). The liquefaction of sands, a collapse surface approach. Canadian Geotechnical Journal, vol. 22, pp 564-578.

Sy, A. & Campanella, R.G. (1991). An alternative method of measuring SPT energy. Proceedings of the Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, USA, vol. 1, pp 499-505.

Tokimatsu, K. & Seed, H.B. (1987). Evaluation of settlements in sands due to earthquake shaking. Journal of Geotechnical Engineering, American Society of Civil Engineers, vol. 113, no. 8, pp 861-878.

Tokimatsu, K. & Yoshimi, Y. (1983). Empirical correlation of soil liquefaction based on SPT N-value and fines content. Soil and Foundations, vol. 23, no. 4, pp 56-74.

Tokimatsu, K., Kuwayama, S. & Tamura, S. (1991). Liquefaction potential evaluation based on Rayleigh wave investigation and its comparison with field behaviour. Proceedings of the Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, USA, vol. 1, pp 357-364.

Vaid, Y.P., Byrne, P.M. & Hughes, J.M.O. (1981). Dilation angle and liquefaction potential. Journal of Geotechnical Engineering, American Society of Civil Engineers, vol. 107, no. GT7, pp 1003-1008.

Valera, J.E. & Donovan, N.C. (1977). Soil liquefaction procedures - a review. Journal of Geotechnical Engineering, American Society of Civil Engineers, vol. 103, no. GT6, pp 607-625.

Valera, J.E. & Kaneshiro, J.Y. (1991). Liquefaction analysis for rubber dam and review of case histories of liquefaction of gravels. Proceedings of the Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, USA, vol. 1, pp 347-356.

Wong, R.T., Seed, H.B. & Chan, C.K. (1975). Cyclic loading liquefaction of gravelly soils. Journal of Geotechnical Engineering, American Society of Civil Engineers, vol. 101, no. GT6, pp 571-583.

Wu, Y.J. (1991). Liquefaction criterion for sand deposits during earthquake. Proceedings of the Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, USA, vol. 1, pp 605-608.

Yasuhara, K., Hyodo, M., Konami, T. & Hirao, K. (1991). Earthquake - induced settlement in soft grounds. Proceedings of the Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, USA, vol. 1, pp 365-370.

6.3 On Marine Sand Fills

Papers in the Proceedings of the Seminar on Marine Sources of Sand, Geological Society of Hong Kong, 1987.

Been, K., Crooks, J.H.A., Conlin, B.H. & Horsfield, D. (1988). Liquefaction of hydraulically placed sand fills. Proceedings of the ASCE Specialty Conference "Hydraulic Fill Structures' 88", Ft. Collins, Colorado, pp 1-18.

Chan, Y.C. (1990). Design and construction of hydraulic sand fill. (Report on a lecture by J.H.A. Crooks). Hong Kong Engineer, vol. 18, no. 4, pp 8-10.

Crooks, J.H.A., Shinde, S.B. & Been, K. (1985). In situ state of underwater hydraulic sand fills. Proceedings of ASCE Conference "Arctic '85 - Civil Engineering in the Arctic Offshore", San Francisco, pp 1-9.

Leung, B.N. (1988). Settlement of Proposed Extension of Cargo/Long Term Apron at Hong Kong Airport. GCO Advisory Report ADR 20/88, Geotechnical Control Office, Hong Kong, 36 p. (Unpublished).

Leung, B.N. & Yuen, K.S. (1988). Settlement of Proposed Extension of Maintenance Apron at Hong Kong Airport. GCO Advisory Report ADR 19/88, Geotechnical Control Office, Hong Kong, 31 p. (Unpublished).

Sladen, J.A. & Hewitt, K.J. (1989). Influence of placement method on the insitu density of hydraulic sand fills. Canadian Geotechnical Journal, vol. 26, pp 453-466.

APPENDIX A

PRELIMINARY ASSESSMENT OF THE LIQUEFACTION POTENTIAL OF TWO SITES AT THE KAI TAK AIRPORT

A.1 Evaluation of Ground Acceleration

The steps taken in assessing the liquefaction potential of the marine sand fills at the two sites at Kai Tak Airport are briefly described below.

First of all, the peak ground acceleration at bedrock level (i.e. the 'free field' acceleration) is estimated using Figure A1, which is based on Pun (1990a). This Figure shows the predicted peak ground accelerations of different return periods.

The acceleration of a soil layer depends on its seismic response, which is governed by its natural period T. A rough estimate of T can be obtained using the following equation :

$$T = 4H/S \qquad \text{(A1)}$$

where H = thickness of soil layer
S = average shear wave velocity of the soil layer

The results of ground investigations indicate that the Kai Tak sites are underlain mainly by marine sand and clay fills, marine and alluvial deposits, and CDG. The total thickness of the superficial deposits varies from 20 m to 30 m (see Figure A2). For the purpose of analysis, an average thickness of 25 m is assumed.

As the value of S for CDG is very high (over 500 m/s), its natural period is very short. Therefore, the free field acceleration can be assumed to apply there without any magnification. For loose sandy marine fills, marine and alluvial deposits, S generally lies between 60 m/s and 100 m/s. A value of 80 m/s is assumed in the calculations. With the assumed parameters, the natural period of the superficial deposits is found to be 1.25 seconds.

In order to determine the seismic response, an acceleration response spectrum has to be used. A number of response spectra are given in Figure A3. These spectra are for 5% critical damping, which is a conservative assumption for the superficial deposits. Responses in between the 'Standard' spectrum given by Housner (1959) and the spectrum given in draft Eurocode 8 are considered to be appropriate for Hong Kong (see Pun (1990b) for discussion on the choice of spectrum). From Figure A3, the normalised acceleration response for a natural period of 1.25 seconds is 0.7 and 0.8 from the 'Standard' and Eurocode spectra respectively. Therefore, an average value of 0.75 is assumed here.

Using the above assumptions, the horizontal acceleration in CDG and the acceleration of the superficial deposits at ground level can now be calculated. The results of the calculations are summarised below :

	Horizontal Acceleration	
Acceleration Return Period T_R	In CDG	At Ground Level (a_{max})
100 years	0.058 g	0.044 g
500 years	0.080 g	0.060 g
1000 years	0.092 g	0.069 g

where g is acceleration due to gravity.

Amongst the superficial deposits, only the loose marine sand fills below water has a risk of liquefaction. The marine and alluvial deposits have much higher fines contents and hence do not pose any risk.

With the calculated accelerations, the critical SPT N-values, i.e. values below which liquefaction may occur, have been evaluated for the marine sand fills using two empirical methods, as outlined in the next Section. The earthquake magnitude responsible for liquefaction is assumed to be 6.0. This is a conservative assumption because previous earthquakes within 200 km of Hong Kong had magnitudes less than 6.0 (Pun, 1990a). Larger magnitude earthquakes have occurred outside the 200 km region. However, due to attenuation, much smaller accelerations will be experienced in the event of such ground motions.

A.2 Assessment of Liquefaction Potential

A.2.1 Seed et al (1985)'s Method

The cyclic stress ratios (τ_{av}/σ_o') for different accelerations have been calculated using equation (2). For the calculations, the maximum depth of the sand fills is taken as 5 m and the depth of water table is taken as 3 m below ground, based on ground investigation results at the sites. Liquefaction potential is evaluated at the depth of 5 m. At this depth, the corresponding r_d and (σ_o/σ_o') values are 0.95 and 1.3 respectively. The results of the calculations are given in the following Table. The probability, p, of exceeding a_{max} for a design life of 100 years have also been computed and tabulated below.

T_R(years)	p for Design Life of 100 years	a_{max}/g	τ_{av}/σ_o'	Critical $(N_1)_{60}$ Values for M = 6.0
100	63%	0.044	0.035	2
500	20%	0.060	0.048	3
1000	10%	0.069	0.055	4

It should be noted that the above assessment is for sands with a fines content less than or equal to 5%. This is considered to be appropriate for the two sites at Kai Tak Airport. If the fines content of the fill is greater than 5%, the critical $(N_1)_{60}$ values for the return periods in question will reduce to almost zero, indicating that liquefaction is most unlikely.

A.2.2 Ambraseys (1988)'s Method

With the same assumptions as before, the critical $(N_1)_{60}$ values for sands with less than 5% fines have been calculated using Ambrasey's method :

T_R(years)	p for Design Life of 100 years	a_{max}/g	τ_{av}/σ_o'	Critical $(N_1)_{60}$ Values for M = 6.0
100	63%	0.044	0.035	3
500	20%	0.060	0.048	3
1000	10%	0.069	0.055	4

For sands with more than 5% fines, the critical $(N_1)_{60}$ values should be subtracted by 4, with a lower limit of zero.

A.3 Discussion

It can be seen that the results obtained from the above two methods are very similar. The critical $(N_1)_{60}$ values can be compared with the actual SPT values of the marine sand fills to evaluate the liquefaction potential of the sites.

The SPT N-values of the Kai Tak fills below water are generally larger than 6, which is higher than the critical $(N_1)_{60}$ values which correspond to ground accelerations even of long return periods. Moreover, conservative assumptions have been made on earthquake magnitude and damping characteristics of the soil. Hence extensive liquefaction is very unlikely. However, some of the N-values of the fill are as low as 3, indicating that there is still a potential for localised liquefaction at weak spots. A check should be made to see whether the Hong Kong SPT values need to be corrected for the purpose of liquefaction potential assessment.

A.4 References

Ambraseys, N.N. (1977). Long-period effects in the Romanian earthquake of March 1977. Nature, vol. 268, no. 5618, pp 324-325.

Commission of the European Communities (1989). Draft Eurocode 8 for Structures in Seismic Regions, Parts 1.1 and 1.2. Commission of the European Communities, 77 p.

Housner, G.W. (1959). Behaviour of structures during earthquakes. Proceedings of the American Society of Civil Engineer, vol. 85, no. EM4, pp 109-129.

Pun, W.K. (1990a). Seismicity of Hong Kong. MSc dissertation, Department of Civil Engineering, Imperial College of Science, Technology & Medicine, University of London, 277 p. (Unpublished).

Pun, W.K. (1990b). A Note on the Comparison of Seismic Load and Wind Load (Draft). Geotechnical Control Office, Hong Kong, 22 p. (Unpublished).

LIST OF FIGURES

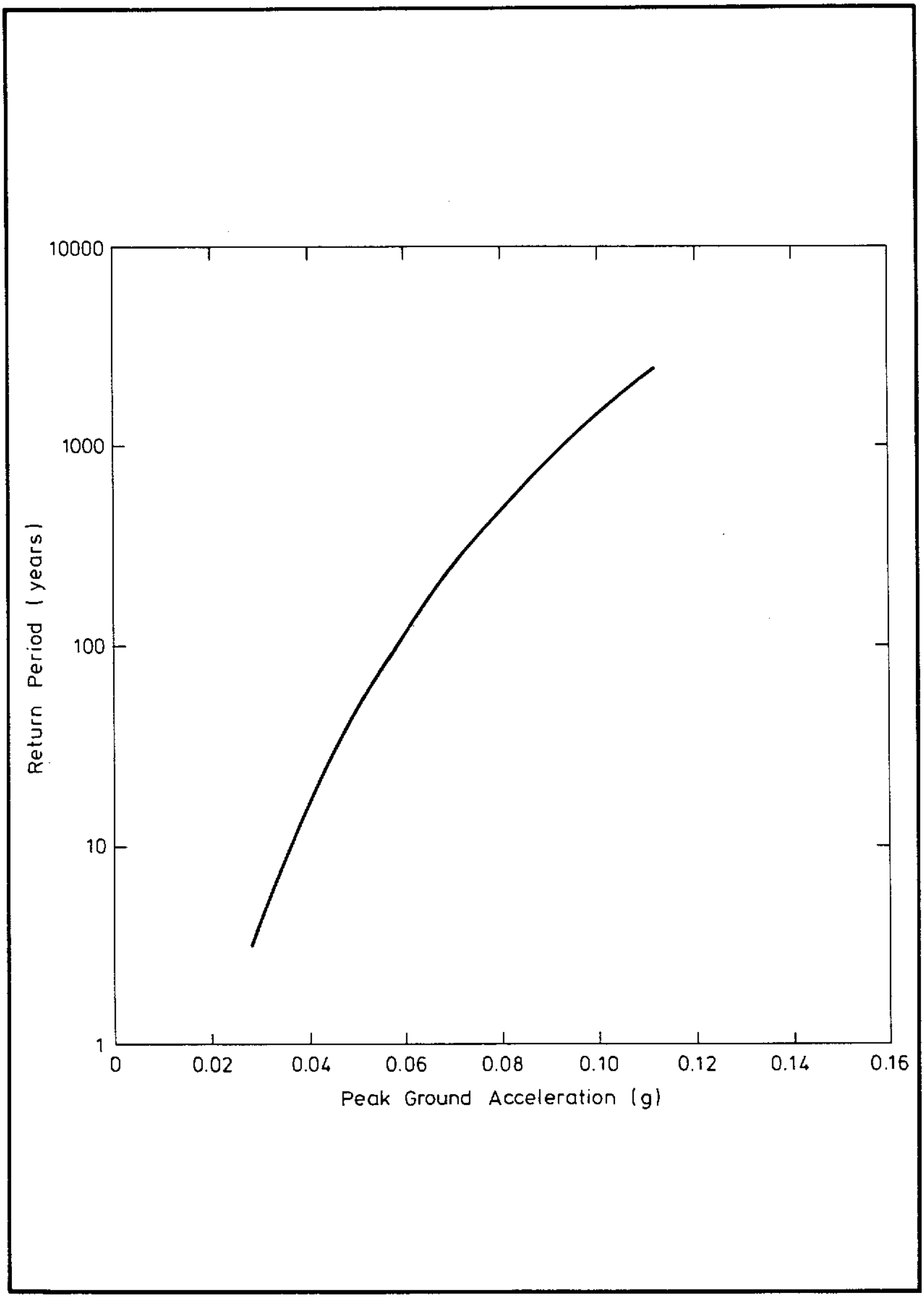


Figure A1 - Peak Ground Acceleration Versus Return Period Relationship for Hong Kong (after Pun, 1990a)

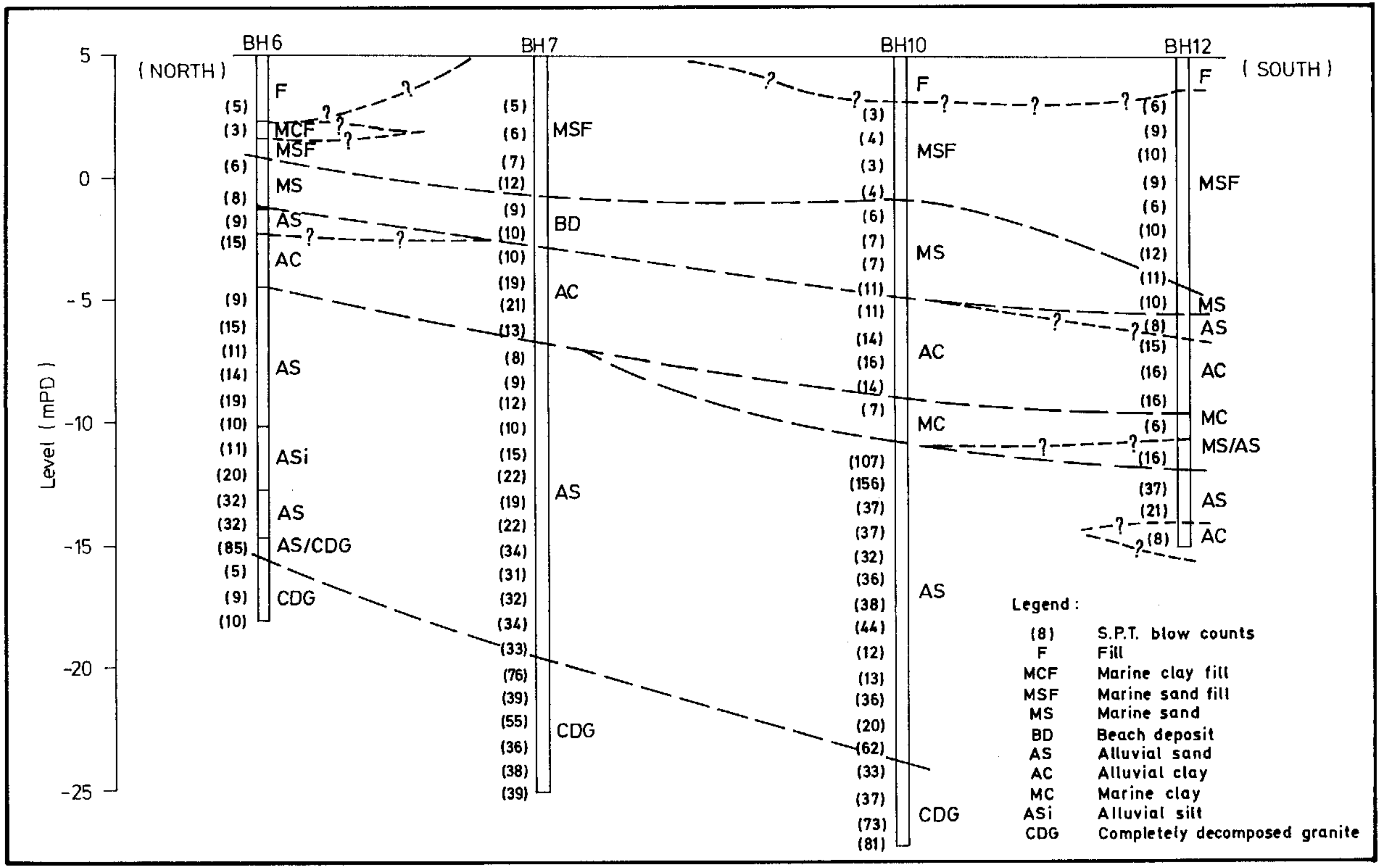
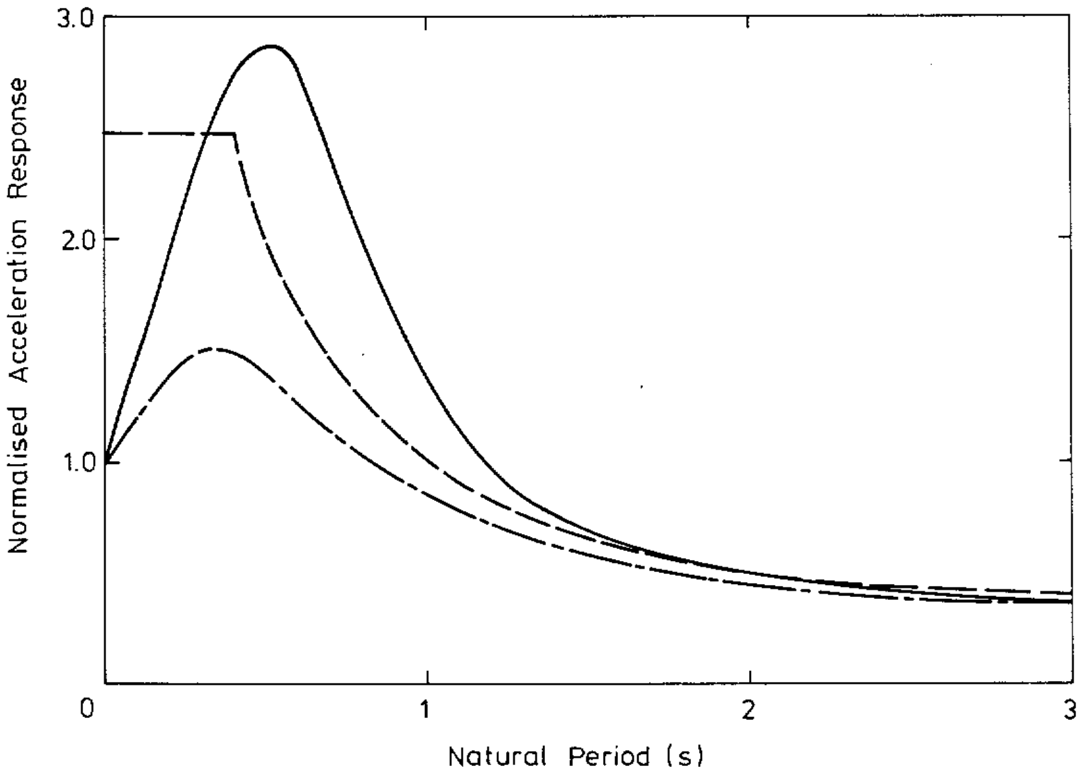


Figure A2 - Typical Subsoil Profile at the Kai Tak Sites (after Leung, 1988)



Legend :

—·— 'Standard' Spectrum given by Housner (1959)
––– Spectrum from draft Eurocode 8
—— El Centro Spectrum (Ambraseys, 1977)

Figure A3 - Normalised Acceleration Response Spectra (after Pun, 1990b)

SECTION 2 : A NOTE ON THE EARTHQUAKE RESISTANCE OF BUILDINGS IN HONG KONG

W.K. Pun

This report was originally produced as GCO Technical Note No. TN 10/91

FOREWORD

In 1990, Mr W.K. PUN carried out a short study on the relative magnitude of wind load and seismic load on buildings in Hong Kong under the supervision of Professor Ambraseys at the Imperial College of Science, Technology and Medicine, London University. The study was a follow up to his MSc dissertation work on earthquake data review for the Hong Kong Region.

This note was prepared by Mr W.K. PUN under the supervision of Dr P.L.R. PANG to document the results of the study. The note also discussed some of the factors which can influence the earthquake resistance of buildings. Messrs S.H. MAK and Y.C. LEUNG reviewed a draft version of the note and provided useful comments on structural aspects of the subject.

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CONTENTS

1. INTRODUCTION

In Hong Kong, all new buildings are routinely designed against wind load but not seismic load. As a follow up to the work of earthquake data review for the Hong Kong region (PUN, 1990; GCO, 1991), a short study was carried out to compare the relative magnitude of wind load and seismic load on buildings in Hong Kong. This note documents the results of the study and discusses some of the factors which can influence the earthquake resistance of buildings.

2. BACKGROUND AND LITERATURE REVIEW

Since 1959, buildings in Hong Kong are required by the Buildings Ordinance to be designed against wind load.

In 1972, the Royal Observatory carried out a study of the seismicity of Hong Kong, the results of which were documented by Lau (1972). The study concluded that "structures built to withstand seismic accelerations of 0.07 g in Hong Kong will probably have survived all historical earthquakes since 288 A.D. in Guangdong". As a result of this work, an acceleration of 0.07 g has been adopted by some designers for aseismic design of selected structures in Hong Kong. Highway bridges are now routinely designed against such a horizontal acceleration (EDD, 1983). In the calculations, the total vertical load is multiplied by a coefficient of 0.07 (corresponding to 0.05 g acceleration times a load factor of 1.4) to obtain the design horizontal forces. The calculated horizontal forces are then used in design without further factoring.

In 1982, some work on the relative magnitude of seismic load and wind load on buildings in Hong Kong was carried out by Chan (1982), under the supervision of a structural engineer, Mr K. L. Lo, in the BOO. Chan found that, by taking proper account of dynamic structural response, the bending moments and shear forces in the beams and columns of a four storey building due to a 0.07 g ground acceleration were 1.5 to 4.0 times those due to wind. For a 14 storey building, the moments and shears induced by the earthquake were half to one time those induced by wind.

Choi (1984) compared the building displacements, storey shears, and beam and column forces induced by wind with those induced by seismic load. For the evaluation of earthquake effects, the UBC (Uniform Building Code, USA) spectrum and the 1/4 El Centro Spectrum were used. For wind, the loading was based on the Hong Kong Wind Code (BOO, 1983). It was found that the magnitude of member forces induced by seismic load evaluated from the 1/4 El Centro Spectrum was always the greatest. The relative importance between wind load and the 'UBC seismic load' was found to be building-type dependent. Two 'wall-frame' buildings were studied. Wind load was found to be more critical for the building with a longer fundamental period while seismic load was more critical for the one with a shorter fundamental period. The study concluded that "in general, buildings in Hong Kong would be damaged by an earthquake of the 1/4 El Centro intensity. As for the UBC level of loading, certain buildings seem to withstand it very well while others would suffer minor damages". With regard to the 1/4 El Centro Spectrum, the peak ground acceleration used was 0.083 g. However, UBC recommends different levels of seismic load according to a seismic zonation plan and there were many revisions for the Code. It is not clear which revision was used and what loading level was adopted.

While in design, both wind load and seismic load are taken as a lateral force, their characteristics are quite different and deserve some discussion. The dominant periods of wind pressure are in the range of 50 sec. and longer while the corresponding values for earthquake ground motions are in the range of 0.2-1 sec. (Gould & Abu-Sitta, 1980). The fluctuation of wind pressure magnitude is relatively small, despite the occurrence of occasional strong gust. Hence, wind pressure can conveniently be measured relative to a static mean pressure, whereas earthquake ground motions do not appear to have such a bias. Because of these differences, the response of structures to the two types of lateral loadings can be very different. The presence of a significant non-fluctuating component in the total wind forces results in a largely static response for many structures subject to wind loading. The smallest prevalent period of wind pressure is much larger than the fundamental period of many structures except for the very flexible ones. This means that the response of most structures to wind load can safely be assumed static and the effect of resonance is not important. However, the dominant periods of earthquake ground motions are in the fundamental period range of many structures and hence the resonance effect cannot be neglected.

3. COMPARISON OF WIND LOAD AND SEISMIC LOAD

3.1 Scope and Methodology of Comparison

For the present study, buildings were modelled as a rectangular block of length L, breath B and height H (Figure 1). Simplified calculation models were used and only the base shear force was evaluated. Wind load was taken as a static pressure acting over the frontal area of the block and seismic action was represented by a horizontal acceleration acting at the centre of gravity of the block with due account taken of the resonance effect.

3.2 Assessment of Wind Load

The wind load on buildings was taken from the Hong Kong Wind Code (BOO, 1983). The design wind pressure adopted, which corresponds to a 50-year return period wind, is shown in Figure 2. No correction was made for the effect of terrain and shielding. Based on the pressure diagram, the total shear force at the base of a building was calculated for different building dimensions and heights. The total base shear V was expressed as a ratio of the total weight, W, of the building. This ratio is known as the 'base shear coefficient' C_b, where

$$V = C_b W \qquad \qquad (1)$$

In the calculation, the weight of building was taken as 8 kPa per storey and storey height was taken as 2.8 m. The results are tabulated in Table 1 and depicted in Figure 3.

3.3 Assessment of Seismic Load

3.3.1 General

Two methods of seismic analysis are commonly used, viz. the equivalent static force method and the multi-modal spectral analysis. The former takes into account only the fundamental period of a structure and is simpler to use. For tall buildings with a long fundamental period, the higher modes of vibration are also significant and the multi-modal spectral analysis should be used. Most national codes allow the use of both methods. Some exceptions are the Russian Code, the Yugoslavian Code, the Bulgarian Code and the former West Germany Code, which mention the multi-modal spectral analysis only. In the present study, the equivalent static force method was used. This method is considered to be adequate for the purpose of comparison of base shears.

3.3.2 Equivalent Static Force Method

In the equivalent static force method, a static lateral force is applied to a building. The force is proportional to the weight W of the building. The total shear force V at the base of the building due to seismic load is given by :

$$V = C_b W \qquad \text{(2)}$$

where C_b = seismic base shear coefficient

The following equation, which has been adopted from the draft Eurocode 8 (Commission of the European Communities, 1989), may be used to evaluate C_b :

$$C_b = ADS\eta/q \qquad \text{(3)}$$

where A = acceleration factor
D = dynamic amplification factor
S = soil factor
η = damping correction factor
q = behaviour factor

The acceleration factor A is applied to reflect the seismicity of a country or a region. For a given region, the value of A can be taken as the horizontal peak ground acceleration (PGA) in %g with a selected recurrence period.

The dynamic amplification factor D allows for the effect of dynamic response of a structure (i.e. resonance effect). It is normally given as a normalised elastic response spectrum for the bedrock condition. The ordinate of the spectrum gives the value of D as a function of the fundamental period of the structure.

Response spectra given in design codes are usually for structures with 5% critical damping. For structures not founded on bedrock or not having 5% critical damping, the given '5%-rock' response spectrum will need to be corrected. The soil factor S and the damping correction factor η cater for variations in the foundation condition and damping characteristics respectively.

For most structures, some yielding or plastic deformation produced by seismic loads are acceptable provided that the 'ductility limits' of the structure are not exceeded. The ability of a structure to dissipate energy by ductile deformation depends primarily on the structural form, the material used and the degree of redundancy of the structure. The behaviour factor q allows for the differences in the ductility or acceptable plastic deformation of the structures. In the draft Eurocode 8, q is defined as the average ratio between the intensity of seismic excitation inducing an ultimate limit state in the structure, taking into account its plastic behaviour, and the intensity of seismic excitation used in design, assuming linear elastic behaviour.

3.3.3 Calculation of Seismic Base Shear Coefficient

The seismic coefficient of a building was calculated using equation (3) for different building dimensions and heights.

An appropriate acceleration factor may be selected based on the work of Pun (1990). For a return period of 50 years, the PGA at Hong Kong can range from 0.05 g to 0.09 g (Figure 4). One major uncertainty in the prediction is in the assumed attenuation law (viz. the Joyner & Boore (1981) attenuation law) used in deriving the PGA. This is due to the lack of strong motion records in South China. In the present study, acceleration factors of 0.05, 0.07 and 0.09 were used for a sensitivity study.

To evaluate the dynamic amplification factor, an acceleration response spectrum is required. Again, due to the lack of strong motion records in South China, no local acceleration spectrum is available. In the present study, the El Centro spectrum (Ambraseys, 1977), the 'standard' spectrum (Housner, 1959), and the spectrum recommended by the draft Eurocode 8 (Commission of the European Communities, 1989) were used. These spectra are shown in Figure 5. The range of response of these spectra is sufficiently wide to cover the response in spectra recommended in most aseismic design codes. The fundamental period of the building was calculated using the rule of thumb of 0.1 N where N is the number of storeys of the building (Newmark & Hall, 1982).

Since a behaviour factor q of 1.0 is implicitly assumed in the evaluation of wind forces, q is also taken as 1.0 in the evaluation of seismic forces. For simplicity, the values of S and η are also taken as 1.0. These assumptions imply that the structure is founded on bedrock and exhibits 5% critical damping.

3.4 Discussion of Results

A simple way of comparing the total shear force at the base of a building due to wind load and that due to seismic load is by comparing the values of C_b obtained. The values of C_b, when multiplied by the total weight of the building, give the total base shear force.

Figure 6 shows the C_b values for seismic load calculated in accordance with Section 3.3.3 for an acceleration factor of 0.05, which corresponds to the mean value in Joyner & Boore's attenuation law. The C_b values shown in Figure 2 for wind load have also been reproduced in Figure 6 for the ease of comparison.

As can be seen from Figure 2, the base shear coefficient corresponding to wind load is not too sensitive to the length of a building but varies significantly with its breath. This is expected because if only the breath is increased, the total lateral force due to wind load remains virtually the same even though the weight of the building is greatly increased. This results in a large decrease in the base shear coefficient. The C_b value also increases steadily with height.

One of the assumptions involved in the calculation of C_b values due to wind load is on the magnitude of the dead and live load. A total vertical load intensity of 8 kPa per storey was assumed in the calculation. This is likely to be on the low side. Should a larger intensity, say 10 kPa, be used, the C_b values due to wind load will be smaller.

Unlike that due to wind load, the seismic base shear coefficient varies with the height of a building. It reaches a maximum value for buildings of around three to five storeys and then drops off for taller buildings.

As can be seen from Figure 6 the C_b values due to wind load are generally greater than those due to seismic load for high-rise buildings, but smaller for low-rise buildings. This observation also holds for PGA's of 0.07 g and 0.09 g. Despite the wide range of acceleration response spectra used, the results show that wind load in Hong Kong is generally more critical than seismic load for high-rise buildings while seismic load is more critical for low-rise buildings, in terms of the total base shear force. It is of interest to note that the results of Chan (1982) and Choi (1984), who carried out detailed analysis on real buildings, showed similar findings.

The exact point at which wind response prevail over seismic response depends on many factors, viz. the form and dimensions of a building, the magnitudes of dead load and live load, the PGA, and the structural response of a building. While Figure 6 shows that wind load is generally more critical than seismic load in terms of the total base shear force for buildings taller than about 20 storeys, and seismic load is more critical for buildings lower than about 7 storeys, these figures are only valid for the simple cases studied.

High-rise buildings in Hong Kong are usually founded on piles. For them, the acceleration response spectra given in Figure 5 may still be used if an appropriate fundamental period is adopted. Generally, the fundamental period of a building on piles is larger than that of a similar building on rock. However, Ward (1991) reported that the fundamental periods of R.C. buildings in Hong Kong are smaller than 0.1N because of the presence of rigid core walls. The two effects are therefore counter-acting to some extent. In order to fully evaluate the soil-structure interaction effect due to a pile foundation, numerical analysis using techniques such as the finite element method is required. However, in view of the contrast between C_b values associated with wind load and seismic load for low-rise and high-rise buildings, the conclusions drawn from Figure 6 are believed to be applicable to buildings on piles. Reference may be made to Dowrick (1987) for more detailed discussion on the effect of pile foundation.

4. DISCUSSION ON EARTHQUAKE RESISTANCE OF BUILDINGS

The assessment of earthquake resistance of buildings in Hong Kong is a complex matter. Many factors, some of which inter-related, are involved. Some of these factors are examined below.

The relevant factors include :

(a) the design loading,

(b) the site geology and soil conditions,

(c) the shape and structural form of the building,

(d) the nature of the materials used,

(e) the provision of ductility, and

(f) the standard of workmanship achieved in construction.

The design loading is a function of PGA, dynamic amplification, site condition, etc., as discussed in Section 3.3.

In the previous Sections of this note, the results of a study of the relative magnitude of wind load and seismic load have been presented. In the study, a PGA corresponding to a 50-year return period earthquake only was considered. However, in aseismic design, PGA's of different return periods are often used, depending on the type of structure involved.

For example, the US Applied Technology Council (1978) of the National Bureau of Standards recommends the use of a PGA with a probability of exceedance of 10% for the design of buildings. For a building with a design life of 50 years, this level of probability corresponds to a PGA induced by a 500-year return period earthquake. Based on the work of Pun (1990), the 'best estimate' PGA at Hong Kong due to such an earthquake event is about 0.08 g. This higher PGA will result in higher inertia forces in buildings than those calculated previously for the 50-year return period PGA (0.05 g). As such, the level of damage may be significant for certain types of buildings designed to resist wind load only if a PGA of 0.08 g occurs at Hong Kong.

It is worth noting here that the design level of PGA set by code drafters is often influenced by economic considerations, in particular, the level of additional cost that can be afforded by a country to incorporate aseismic design requirements, as well as the loss of life and property it is prepared to accept in the event of a strong ground motion occurring. Even without codified requirements, a prudent designer may need to assess the dynamic response of a building subjected to earthquakes of selected recurrence interval to satisfy himself that the movements induced will not cause undue alarm to residents.

A brief discussion of the choice of design response spectrum here is also considered useful, as this influences the assessment of design seismic loads for buildings in Hong Kong. As no record of local strong ground motion exists, there is some uncertainty in the assessment of structural response to earthquakes. In the study described earlier, three response spectra were used. These were specifically chosen to cover worldwide upper and lower bound values of dynamic response of structures founded on rock due to earthquakes. However, for 'tuned' structures, i.e. structures founded on top of a layer of soil deposit having a fundamental period close to that of the structure or the incoming seismic wave, a much higher dynamic magnification of the free field PGA could occur. The level of damage could be high for such structures even when the incoming PGA is not large.

The shape and structural form of a building can significantly affect its performance in the event of an earthquake. Buildings with the following features have been found to be particularly prone to earthquake damage (Tiedemann, 1990) :

(a) non-symmmetrical or irregular plans,

(b) a large difference in the architectural plans and stiffnesses at different storeys,

(c) a 'single direction' of strength (e.g. load-bearing walls all in the same direction, or frame buildings with a uni-directional structure),

(d) excessive wall openings in a storey (i.e. a 'soft storey'),

(e) heavy roof forms and canopies,

(f) design 'imperfections', e.g. vertical load-bearing elements not aligned from one floor to the next.

Depending on the material used, some forms of construction are more vulnerable to earthquake damage than others. Observations support the view that welded structural steelwork is more ductile than reinforced concrete, which is in turn less vulnerable than masonry.

Adequate structural framing and careful detailing are also very important. For wind loads, a large static component is usually present in the wind pressure. Therefore, the framing (e.g. provision of ties) for stability and the detailing requirements to cater for dead and live loads given in building codes are generally adequate to deal with the effects of wind. However, seismic loads, because of their inertia-driven cyclic nature, impose a much higher ductility demand on the structural elements and their connections than wind loads. Therefore, even if a structure is designed and detailed to cater for a level of wind load equivalent to the design seismic load, there is no certainty that structural damage will not occur in the event of the seismic load occurring. The situation could be worse if plastic behaviour of the structure has been allowed for in the design against wind forces.

Standard of workmanship, while an important factor, often cannot be quantified. Vulnerability functions which take this factor into account have been proposed for the assessment of seismic risk. However, such functions have not been established for the types of construction commonly found in Hong Kong.

Based on the above discussion, it can be seen that it is not a simple task to quantify the earthquake resistance of buildings. However, from a careful consideration of the relevant factors, it is not difficult to recognise that the inherent earthquake resistance of some buildings in Hong Kong, which even though designed against typhoons, may be relatively low. Examples of susceptible structures include low-rise buildings, medium height buildings with a 'soft storey' and buildings founded on top of a layer of soil deposit with a very similar fundamental period.

Some supplementary comments are made here regarding highway structures in Hong Kong. These are currently designed against a horizontal ground acceleration of 0.07 g. No account is taken of the possible effect of structural resonance. Based on the work of Pun (1990) and the magnification factors given by response spectra found in various design codes, it is considered that the probability of earthquake damage of 'rigid' highway structures may not be negligible.

5. CONCLUSIONS

Dowrick (1976) pointed out that "in countries with low seismicity, seismic load is usually ignored in design on the tacit assumption that wind load will be more severe, but this may not always be the case. For example, for some structures the earthquake response may be worse than the wind response for the same probability of occurrence. This is partly due to the fact that wind loads tend to increase with increasing (building) height while seismic loads tend to increase with decreasing height."

The findings of Dowrick are supported by the work carried out by local investigators (Chan, 1982 & Choi, 1984). In the present study, a comparison of the relative effect of wind load and seismic load has been made, using peak ground accelerations derived from a recent seismic hazard analysis carried out by Pun (1990). The trend of the findings of previous investigators who have carried out more detailed analysis is confirmed.

Apart from loading, a number of other factors can affect the earthquake resistance of buildings. A review of these factors is given in Section 4 of this Note. Amongst the factors, site amplification due to geology and soil conditions, the shape and structural form of the building and the provision of ductility (e.g. by careful detailing) are particularly important. Since no consideration is given to these aspects in existing Hong Kong design codes, some types of buildings even though designed to resist wind loads may still be damaged in the event of a moderately strong earthquake.

6. REFERENCES

Ambraseys, N.N. (1977). Long-period effects in the Romanian earthquake of March 1977. Nature, vol. 268, no. 5618, pp 324-325.

Applied Technology Council (1978). Tentative Provisions for the Development of Seismic Regulations for Buildings (ATC Publication No. 3-06). National Bureau of Standards, USA, 505 p.

BOO (1983). Code of Practice on Wind Effects, Hong Kong. Hong Kong Government Publication, 14 p.

Chan, C.H. (1982). Earthquake Resistance of Buildings in Hong Kong. Summer Student Training Report, Buildings Ordinance Office, 74 p. (Hong Kong Government Civil Engineering Library Accession No. E64-50990). (Unpublished).

Choi, E.C.C. (1984). Seismic resistance of buildings designed against typhoon. Proceedings of the Third International Conference on Tall Buildings, Hong Kong and Guangzhou, pp 343-349.

Commission of the European Communities (1989). Draft Eurocode 8 for Structures in Seismic Regions, Parts 1.1 and 1.2. Commission of the European Communities, 77 p.

Dowrick, D.J. (1976). Overall stability of structures. Structural Engineer, vol. 54, no. 10, pp 399-409.

Dowrick, D.J. (1987). Earthquake Resistant Design. (Second edition). John Wiley & Sons Ltd, 519 p.

EDD (1983). Civil Engineering Manual, Vol. V, Chapter 4 - Design of Highway Structures and Railway Bridges. Engineering Development Department, 170 p.

GCO (1991). Review of Earthquake Data for the Hong Kong Region (GCO Publication No. 1/91). Geotechnical Control Office, 115 p.

Gould, P.L. & Abu-Sitta, S.H. (1980). Dynamic Response of Structures to Wind and Earthquake Loading. Pentech Press Ltd., Plymouth, 175 p.

Housner, G.W. (1959). Behaviour of structures during earthquakes. Proceedings of the American Society of Civil Engineers, vol. 85, no. EM4, pp 109-129.

Joyner, W.B. & Boore, D.M. (1981). Peak horizontal acceleration and velocity from strong-motion records including records from the 1979 Imperial Valley, California earthquake. Bulletin of the Seismological Society of America, vol. 71, no. 6, pp 2011-2038.

Lau, R. (1972). Seismicity of Hong Kong. Royal Observatory, Hong Kong, Technical Note no. 33, 30 p. (Reprinted with revisions, 1977).

Newmark, N.M. & Hall, W.J. (1982). Earthquake Spectra and Design. E.E.R.I. Monographs Series, 103 p.

Pun, W.K. (1990). Seismicity of Hong Kong. MSc Dissertation, Department of Civil Engineering, Imperial College of Science, Technology & Medicine, University of London, 277 p. (Unpublished).

Tiedemann, H. (1990). Newcastle : The Writing on the Wall. Swiss Reinsurance Company, Zurich, Switzerland, 85 p.

Ward, H.S. (1991). Earthquake resistant design and construction. Proceedings of the International Conference on Seismicity in Eastern Asia, Geological Society of Hong Kong. (In preparation).

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Table 1 - Base Shear Coefficients Corresponding to the Design Wind Pressure

No. of Storeys N	B = 10 m			B = 20 m		B = 30 m
	L = 10 m	L = 20 m	L = 30 m	L = 20 m	L = 30 m	L = 30 m
2	0.040	0.044	0.052	0.020	0.021	0.013
5	0.051	0.054	0.064	0.025	0.026	0.016
10	0.066	0.069	0.077	0.031	0.031	0.020
15	0.076	0.077	0.086	0.035	0.035	0.022
20	0.084	0.083	0.095	0.038	0.038	0.024
25	0.092	0.090	0.102	0.041	0.041	0.026
30	0.097	0.095	0.107	0.043	0.043	0.027
40	0.110	0.104	0.116	0.047	0.047	0.030

Legend :

B Breath of building
L Length of building

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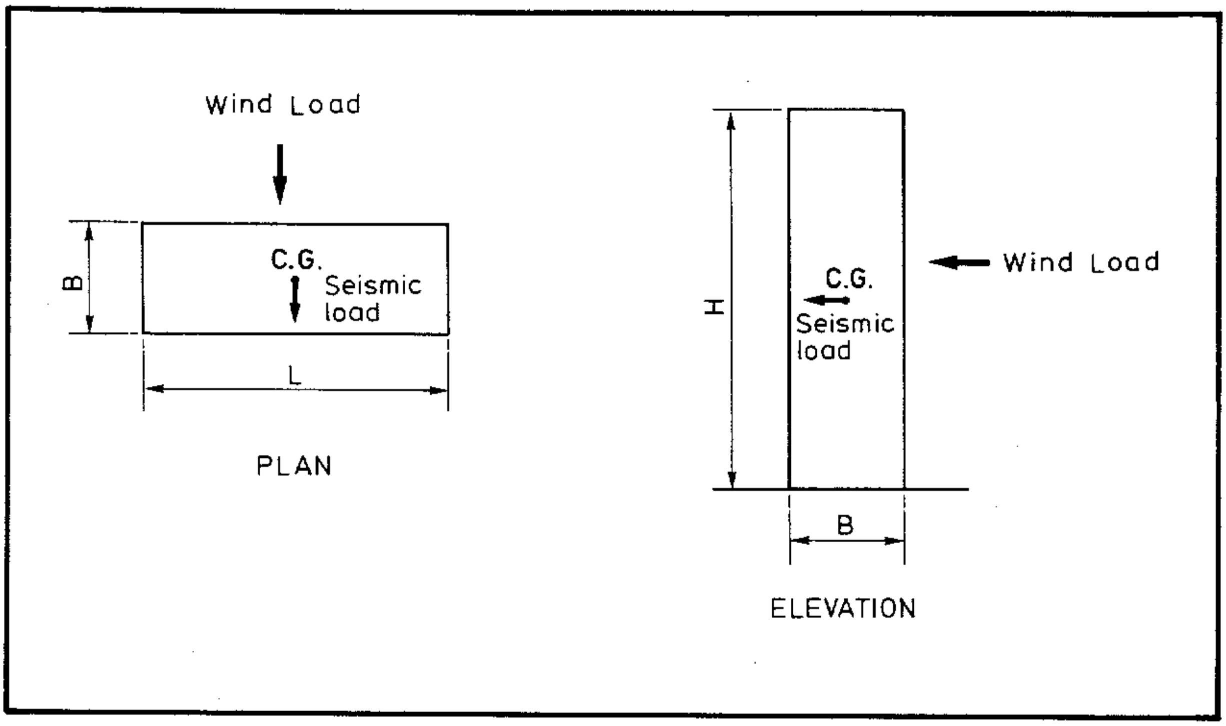


Figure 1 - Schematic Model of Buildings

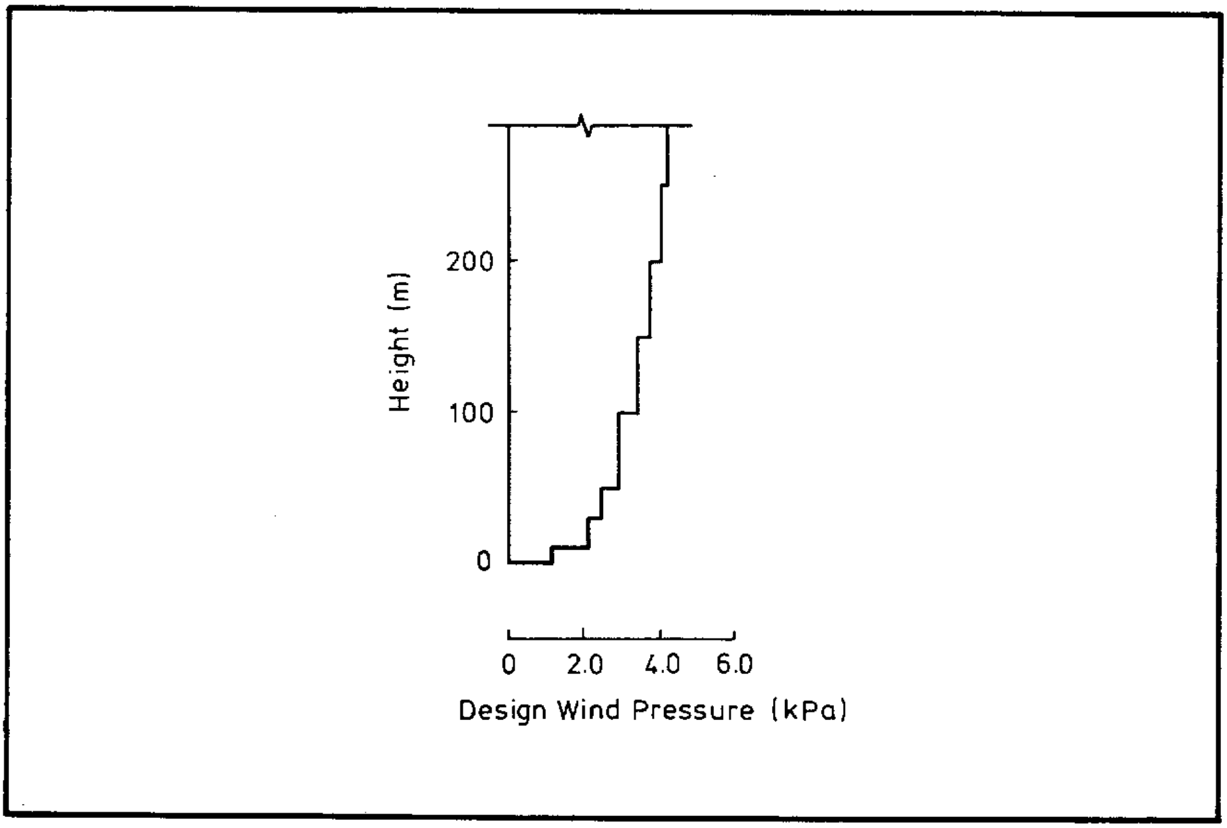
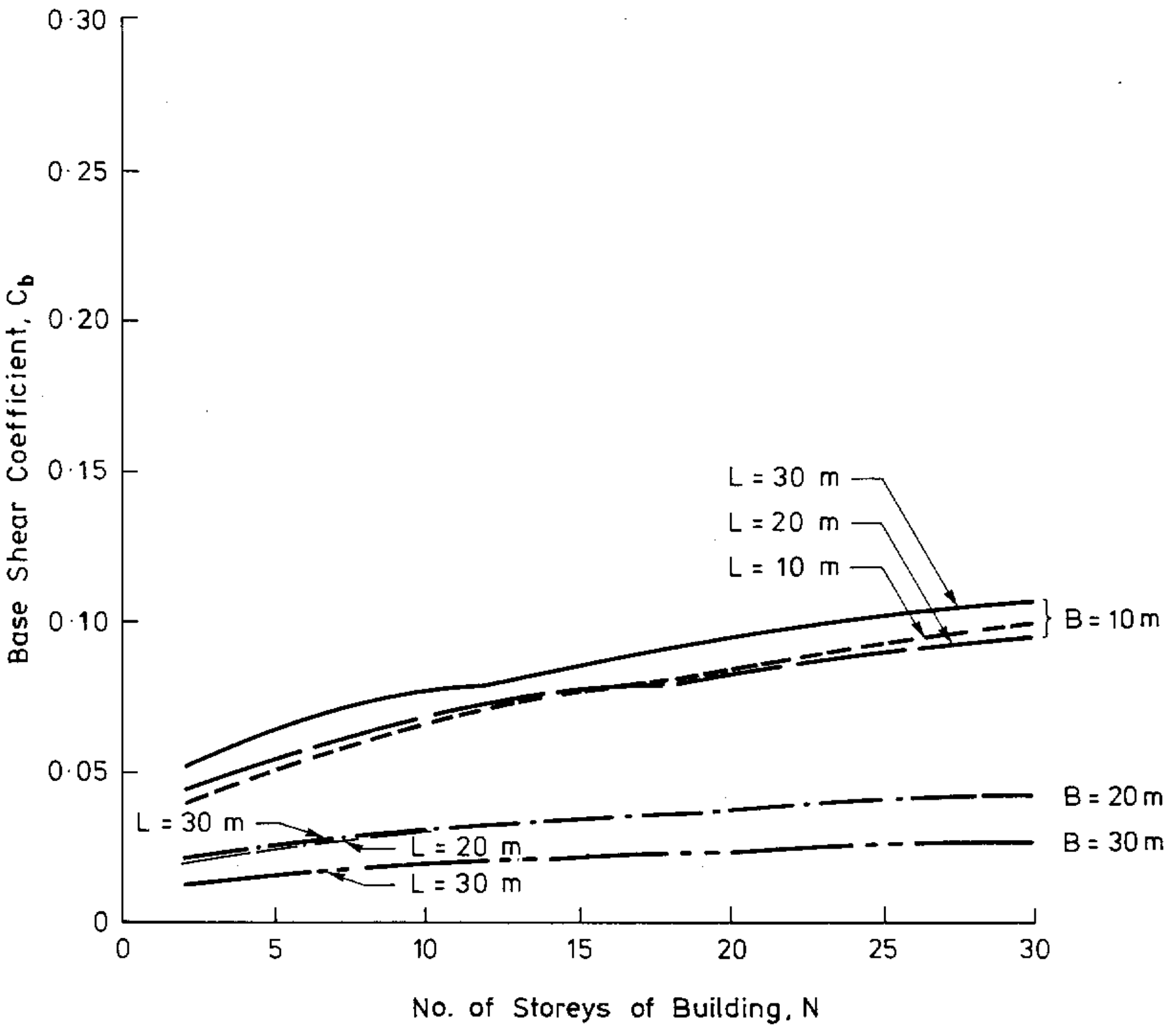


Figure 2 - Design Wind Pressure Used in the Study



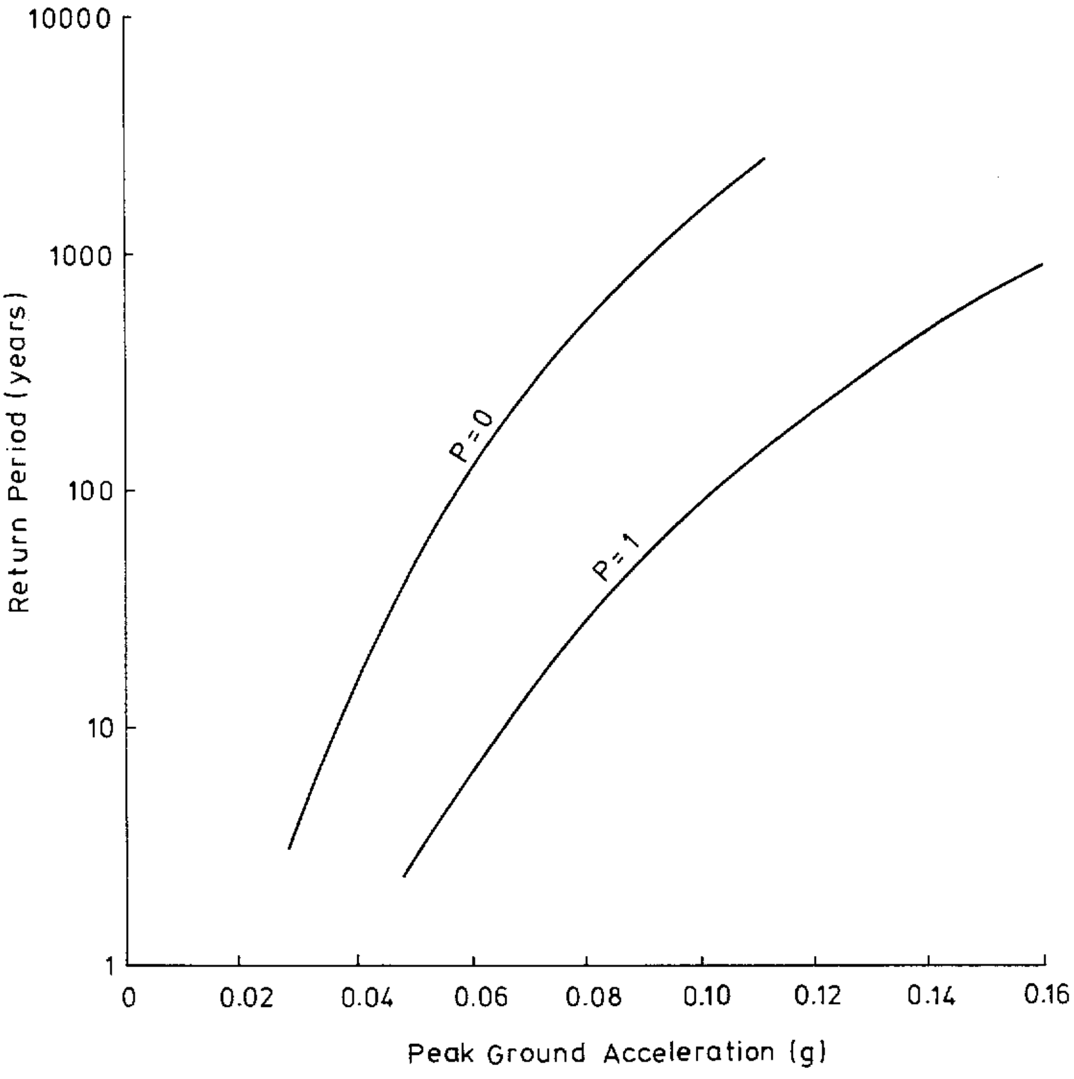
Notes : In the calculation of C_b, the following have been assumed :

(a) Return period of wind load = 50 years.

(b) Storey height = 2.8m.

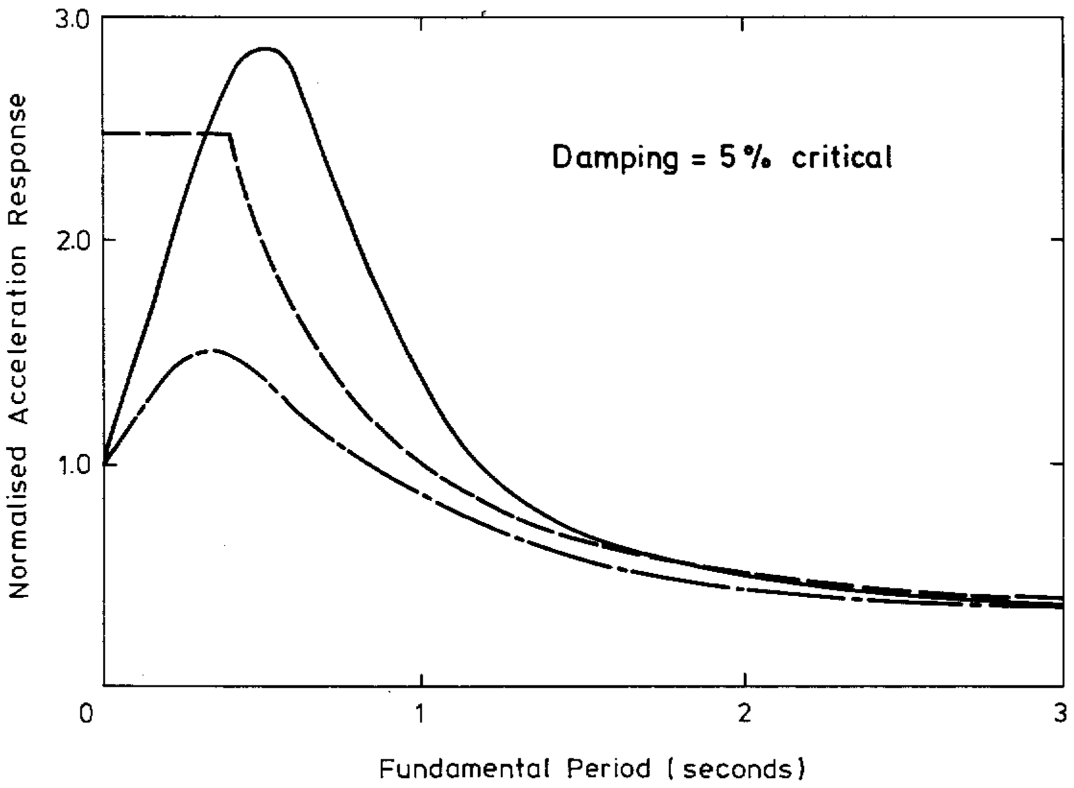
(c) Dead and live load = 8kPa per storey.

Figure 3 - Base Shear Coefficients Corresponding to the Design Wind Pressure



Note: P is a variable given in Joyner & Boore (1981)'s attenuation law. P=0 refers to mean value and P=1 refers to the upper 84 percentile value of the peak ground acceleration.

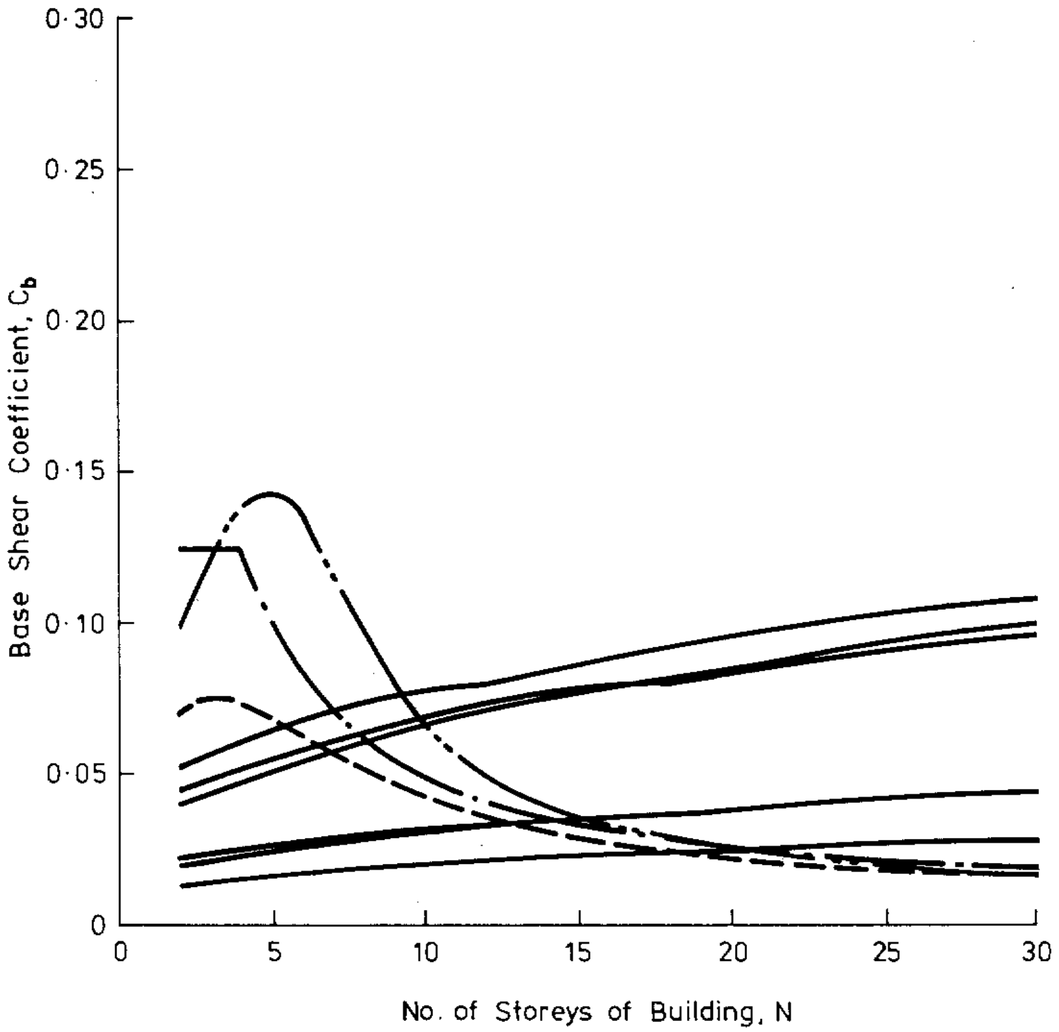
Figure 4 - Peak Ground Acceleration Versus Return Period Relationship for Hong Kong (Pun, 1990)



Legend:

—·— 'Standard' Spectrum given by Housner (1959)
— — Spectrum from draft Eurocode 8 (Commission of the European Communities, 1989)
—— El Centro Spectrum (Ambraseys, 1977)

Figure 5 - Normalised Acceleration Response Spectra Used in the Study



Legend :

- C_b values due to design wind pressure
- C_b values due to seismic load with 'Standard' spectrum
- C_b values due to seismic load with Eurocode spectrum
- C_b values due to seismic load with El Centro spectrum

Note : The various curves of C_b values for wind load correspond to different length and breath of buildings as shown in Figure 3.

Figure 6 - Comparison of Base Shear Coefficients due to Wind Load and Seismic Load (PGA = 0.05 g)