QRA OF COLLAPSES AND EXCESSIVE DISPLACEMENTS OF DEEP EXCAVATIONS

GEO REPORT No. 124

Ove Arup & Partners Hong Kong Ltd

GEOTECHNICAL ENGINEERING OFFICE
CIVIL ENGINEERING DEPARTMENT
THE GOVERNMENT OF THE HONG KONG
SPECIAL ADMINISTRATIVE REGION

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PREFACE

In keeping with our policy of releasing information which may be of general interest to the geotechnical profession and the public, we make available selected internal reports in a series of publications termed the GEO Report series. A charge is made to cover the cost of printing.

The Geotechnical Engineering Office also publishes guidance documents as GEO Publications. These publications and the GEO Reports may be obtained from the Government's Information Services Department. Information on how to purchase these documents is given on the last page of this report.

R.K.S. Chan
Head, Geotechnical Engineering Office
February 2002

FOREWORD

This GEO report presents a study on the quantitative risk assessment (QRA) of collapses and excessive displacements due to deep excavations associated with private building developments in Hong Kong. The QRA study was carried out by Ove Arup & Partners Hong Kong Ltd. for the Geotechnical Engineering Office. Mr Y S Au-Yeung of Island Division administered the consultancy and reviewed the study report.

The main objectives of this study are (i) to estimate the risk level of private deep excavations and compare it with that of other geotechnical hazards, (ii) to identify the principal causes of past failures and (iii) to review and improve the effectiveness of GEO control efforts on private deep excavations. This study reviewed cases of past collapses and significant displacements in the last two decades. It categorized the cases based on frequency and consequence, and assessed the overall risks due to these failures.

The report concludes that the overall risk from deep excavations in Hong Kong is many times less than that from man-made slopes, but the risk from an individual excavation however is in the same order as the annual risk from a slope feature. Workmanship in either wall installation or strutting was found to be the main cause of failures. Based on the findings, the report also gives recommendations on measures to further reduce the risk of deep excavations in Hong Kong.

S H Tse
Chief Geotechnical Engineer/Island

EXECUTIVE SUMMARY

Ove Arup & Partners Hong Kong Limited (ARUP) have been appointed by GEO to carry out the Quantitative Risk Assessment (QRA) of collapses and excessive displacements due to deep excavations associated with private developments in August 1998. ARUP are supported in this study by ERM Hong Kong Ltd. The risk assessment is to quantify the risk to life from deep excavations. The study was carried out based on the past records of collapses and excessive displacements as catalogued in GEO Report No. AR 2/92 "Review of collapses and excessive deformation of excavations".

The approach to the study includes a review of previous incidents in Hong Kong and abroad. Subsequently hazard identification studies, frequency assessment, consequence assessment and risk estimation were carried out based only on data from private developments in Hong Kong. As stated above only the risk to life was estimated.

The QRA study presented in this report leads to the following conclusions:

a) The risk to life is calculated to have a PLL of about between 0.015 and 0.03 per year. These values include workers who account for about a third of the risk.

b) The higher of the above range of results comes from the average rate of failures observed since 1980. Government control has improved since 1990 and if trends since then are used for predicting the effects of future excavations the lower figure is more realistic.

c) The contribution to the risk is significantly higher for sheet pile walls than for other types of walls. The case histories show this is mainly due to inadequate penetration due to obstructions or inadequate strutting.

d) Poor site control is a dominant cause of the observed problems. Occasionally poor planning leads to reports of excessive displacement.

e) The public are most at risk from buildings on pad foundations adjacent to excavations collapsing. Risk to pedestrians is the next main contribution.

The following measures are recommended to reduce the risk:

a) improved site control, principally by more thorough supervision and random site visits.

b) improved planning to prepare better for cases where significant displacements are expected at the design stage.

c) routine monitoring to maximise the chance of warning of a collapse.

The estimated risk has been compared to that estimated previously from pre GCO man made slopes and retaining walls. The overall risk from deep excavations in the Hong Kong Special Administrative Region is many times less than that from slopes. The risk from an individual excavation however is the same order as the annual risk from a slope feature. For example the worst combination for an individual excavation, namely a deep sheet pile supported excavation adjacent to a building on pad foundations, is comparable to the annual risk calculated for some sites with a history of failure. It must be noted however that the risk from an excavation is transient whereas for a slope it is effectively permanent over many years.

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1. INTRODUCTION

In August 1998 GEO appointed Ove Arup and Partners Hong Kong Ltd (ARUP) to carry out a Quantitative Risk Assessment (QRA) of collapses and excessive displacements of deep excavations associated with private building sites. ARUP are supported in this study by ERM Hong Kong Ltd. The risk assessment is to quantify the risk to life from deep excavations. For the purposes of this study deep excavation means an excavation deeper than 5m that uses a retaining wall system. Collapse and excessive displacement are defined in Section 4 and includes the formation of a void or displacement adjacent to the excavation sufficient to cause potential risk to the public.

In the past twenty years there have been several examples of excessive displacement or collapse caused by deep excavations in the Hong Kong Special Administrative Region. Many of these are catalogued in a GEO report “Review of collapses and excessive deformation of excavations” produced in 1992. These cases have lead to the GEO increasing their requirements for design submissions and for site supervision together with additional spot checks on site by their staff. For the purposes of planning GEO require a QRA of deep excavations associated with private building sites to be able to rank the importance of this area of their work with that of other areas. It should also allow them to target on what aspects of excavations they should concentrate their efforts.

This report is the final report produced for the GEO. It presents the considerations for the QRA study and the calculations and results. It also presents conclusions and recommendations of the study and makes suggestions for further studies and improved procedures in the future.

2. APPROACH TO THE STUDY

This section outlines the broad approach to the study.

The main stages of QRA are as follows:

- Hazard identification;
- Frequency estimation;
- Consequence assessment and
- Risk summation.

2.1 Hazard Identification

The objective of this first stage of assessment is to identify all hazards and their failure modes. Hazard identification is based on detailed analysis of historical incidents to identify the failure modes. Several of the case histories that have occurred in Hong Kong have also been re-analysed using the computer program *Oasys* FREW. The purpose of this analysis was to study whether the FREW analysis could assist in identification of the probable causes of the failure.

2.2 Frequency Estimation

Failure frequencies may be estimated from historical failure data or from a detailed examination of the causes of failure due to a range of mechanisms.

In the present study, the frequencies of failure are estimated based on historical data. The data sample is not very large given that it includes data only from Hong Kong. The Consultants are not aware if any observed failure rates for excavations have been estimated elsewhere and even if it were, it may not be applicable since geotechnical control, construction practices, ground conditions etc could be very different.

However, information on incidents elsewhere is still useful to understand the causes of failure, the potential for large failures and the potential for failures to cause multiple fatalities which may not all be reflected in the limited data set for Hong Kong. A brief description on some of the major failures reported elsewhere are also therefore included.

The overall frequency of failure is further broken down by wall type to account for the effect of wall type on failure rate. To analyse the historical data four typical wall types have been considered:

- Sheet pile,
- Caisson wall,
- Diaphragm wall and
- Soldier pile/pipe pile.

Recently hand-dug caissons have stopped being used in Hong Kong except for very difficult site conditions, and therefore caisson walls are not likely to be used in the future. Therefore for future excavations the wall types considered have been modified to include large diameter bored pile walls.

2.3 Consequence Assessment

The consequence assessment for collapses and excessive displacements involves estimation of the extent of failure (ie, the area affected by collapse), the potential for failures to affect buildings, roads, utilities and the excavation sites and the potential for fatalities.

There is not sufficient data to undertake a rigorous analysis of the extent of failure in terms of effect distances and the impact of such failures on buildings and roads. The assessment is therefore largely based on expert judgement. The extent of failure caused by a collapse is represented by the following 3 types; each with an assumed affected area on plan and volume of failed debris.

- Small collapse
- Medium collapse
- Large collapse

Furthermore, it is assumed that the probability of occurrence of each of the above type is related to depth of excavation. Two typical depth ranges <10m and >10m are considered and the associated probabilities for type of collapse corresponding to depth are based on historical data, modified suitably to account for the potential for large failures.

Event tree techniques have been adopted to model the various outcomes following an initial failure. The event tree branch probabilities are based on judgement. In the derivation on some of the branch probabilities however, available data have been used which are explained in the relevant sections of the report.

The analysis should be treated with caution due to the uncertainties associated with the estimation of escalation probabilities (ie, escalation of the primary failure to an outcome resulting in multiple fatalities), and the probabilities associated with different levels of fatalities.

2.4 Risk Summation

It is common to estimate and express risk in terms of 'Individual Risk' and 'Societal Risk'.

Individual risk

Individual risk is, as the name suggests, the risk to specific individuals (for example, various categories of workers, the general public, road users, etc.). Individual risk is in fact a frequency with which individuals within the specified category are expected to suffer the harm (eg, to be fatally injured, or receive major injuries).

The Hong Kong Government has established both individual and societal risk guidelines for planning applications around Potentially Hazardous Installations (PHIs). Chapter 11 of the Hong Kong Planning Standards and Guidelines discusses these guidelines. The Individual Risk criterion specifies that the risk of fatality to an off-site individual should not exceed 10^{-5} per year. A recent study by GEO has recommended that the same criterion should also be used as interim risk guidelines for landslides and boulder fall from natural terrain. This criterion could therefore be regarded as applicable for any hazardous activity that could affect an individual not connected with it.

However, in the case of collapses caused by an individual excavation, it is to be noted that the risk posed to an individual at a specific location is transient (unlike hazards posed by a PHI or a segment of natural terrain) since any specific excavation at a particular location would only last up to a year or for a couple of years. Even during this relatively short period different stages of activities may pose different degrees of hazard. However, the apportioning of the total risk to different stages of activities is out the scope of this study and an average risk over the whole excavation period is assumed.

Societal Risk

Societal risk is a measure of the overall risk associated with a situation or system. It accounts for the likely impact of all accidental events, not just on a particular type of individual, as in the case of individual risk, but on all individuals who may be exposed to the risk, and it reflects the number of people exposed.

The main form in which societal risk is presented is as a relation between incidents which cause some number N or more fatalities and the frequency F of such incidents. This is represented as an F-N curve. The basic F-N data may be integrated to obtain a value for the equivalent annual fatalities, which is termed as Rate of Death or Potential Loss of Life (PLL).

In this study of collapses due to deep excavations, societal risk results have been generated for the whole of Hong Kong and there is no comparable criteria to judge if the risks derived are tolerable. One possible measure for comparison is the interim societal risk criteria developed for transport of dangerous goods (which traverses through different parts of Hong Kong) that is applied on a Hong Kong wide basis, but this would require further assessment on whether the approach adopted to derive Hong Kong wide criteria for transport is applicable for excavation work.

Societal Risk Guidelines have been developed for PHIs and more recently for natural terrain landslide and boulder full hazards. The societal risk criteria is applied for each PHI or for a specific segment of natural terrain. For the present study, comparisons have been made between the calculated risk for an individual excavation and those estimated previously for individual slope features. These comparisons are made for societal risk results in terms of F-N curves and PLL. It is important to note however that the risk for an excavation is a singular occurrence whereas those calculated for slope features are annual risk values. However, the average number of future deep excavations per year in Hong Kong has been estimated based on past records and therefore the average annual risk from excavations estimated. This has enabled the overall annual risk in Hong Kong, for both types of geotechnical features (i.e. excavation and man-made slope), to be compared.

3. <u>REVIEW OF PREVIOUS INCIDENTS</u>

The aim of this part of the study is to understand types of failures and their causes and to assess risk to life from these events. The data sources are as follows.

3.1 <u>Information from GEO</u>

In 1992 GEO published their report AR 2/92 ‘Review of collapses and excessive deformation of excavations’. The report lists 31 cases of collapse or excessive displacements arising from deep excavations in the years from the late 1970’s up until 1992. In addition to this GEO have provided one additional case that occurred in 1993. It should be noted that these cases are not an exhaustive list of all events, but only those cases made known to GEO.

Table 3.1 lists the cases supplied by the GEO and listed in their report AR 2/92. For most of these cases GEO have supplied additional information from their files as part of this study. This information was used in compiling the original 1992 report. This involved GEO retrieving many of the original files and was done to get a better understanding of the ground conditions at the site and of the design including details of the wall section used etc. Table 3.1 summarises the data and gives the following data for each of the 31 cases.

Total planned depth of the excavation.

The type of wall (ShP = sheet pile, DW = diaphragm wall).

The number of planned strutting levels.

Details of the ground conditions.

The observed depth to the water table.

Depth at time of failure - it should be noted that, in a few cases excessive displacement occurred due to activities to install the retaining wall prior to any excavation occurring

The year in which the incident occurred.

Classification of the failure - whether excessive displacement or collapse. The scale of the collapse is also given and is defined later in Section 4.

Description of failure giving details of the damage caused.

Cause of failure - as stated in the GEO report AR 2/92.

Effects on water pipes - whether it is known whether a water pipe was present and any effects on it. The damage descriptions match those in Report AR 2/92.

Comments by ARUP - other observations and may briefly discuss the design. Government projects are also identified but it must be noted these are excluded from the risk assessment discussed later.

Whether an *Oasys* FREW calculation has been carried out as part of this project.

The final case from the GEO that occurred in 1993 is listed as case number 32 at the beginning of Table 3.2. Table 3.2 contains very similar information to that shown in Table 3.1 and described above. It also states our understanding of the cause of failure and gives the source of the data.

3.2 Data from Other Sources

3.2.1 Ove Arup and Partners for Failures in Hong Kong

As part of this project senior staff in ARUP Geotechnics were interviewed to obtain details of other failures in Hong Kong. Two cases were identified and these are listed in Table 3.2 as Case Numbers 33 and 34. Both of these events involved diaphragm walls.

3.2.2 Other Consultants in Hong Kong

As part of this project ARUP wrote to other consultants in Hong Kong asking for case history data (see enclosed sample letter in Appendix A). This letter has been endorsed by GEO as shown in the appendix. Twenty one letters were sent out and two responses gave details of case histories from failures. Two of these were already listed in the GEO report and two others were the two cases where ARUP were involved and therefore already had data as described above.

3.2.3 Ove Arup and Partners for Major Failures Worldwide

ARUP worldwide was canvassed for examples of severe collapses. Several case histories were revealed by this. These are as follows

- An old example has been cited in the UK. This was a sheet piled excavation with an orthogonal set of steel propping. At the intersection of each line of props the propping system was supported by small driven piles and the propping system was effectively pin jointed at each intersection point. At an advanced stage of the excavation one of the driven piles pulled out of the ground leading to instability of the entire propping system which behaved like a mechanism. Extensive failure resulted but it is not known if there were any deaths.

- In 1989 there was a major collapse of a diaphragm wall in Seoul in South Korea. Details of this failure have been published in Davies (1990) and it is believed to be typical of some previous major failures in Korea. The excavation was about 20m deep to bedrock in ground comprising 7m of fill over 13m of loose alluvial sands, silty sand and sandy gravel. The ground water table was about 8m deep. There were generally 4 levels of propping and an additional level in one corner near to a deeper sump (see Figures 3.1 and 3.2). The failure occurred after heavy rain when the excavation was at its deepest. It is believed that the strutting system was inadequate, especially the details for the diagonal struts near the corners, and that the wall may have been somewhat under-reinforced. Extensive lengths of the wall collapsed completely. No deaths resulted but it is noted that some warning of the collapse was given by noises from the propping system.

- In 1997 there was a major collapse of a circular 50m diameter diaphragm wall cofferdam in Bangkok. A photo of the collapse is shown in Plate 3.1. This excavation was about 22m deep and was formed in typical Bangkok ground conditions comprising about 2m of fill over 15m of soft clay over a succession of stiff clays and medium dense sands. The ground water table is near ground level at the surface but water in the underlying sand layers has been drawn down. The circular cofferdam was unbraced and relied on the ring compression of the diaphragm wall for support. However one corner was open due to an additional adjoining excavation with conventional propping. To complete the ring compression there was a set of propping. The failure was caused by this propping failing leading to the circular cofferdam collapsing. Again no deaths resulted and there was warning due to noises from the propping system.

- In the early 80's there was a failure in Singapore due to base heave. The ground conditions comprised fill over soft clay over alluvial sands. As the excavation progressed the water pressure acting up under the clay exceeded the weight of the remaining clay and a quick condition resulted. This caused loss of excavation equipment but no loss of life.

3.2.4 Other Published Cases

In the 1996 HKIE Geotechnical Seminar entitled "Geotechnical Problems of Rapidly Developing Areas in China" there is a paper by Lu, Lai and Li about two excavation failures in Guangdong Province. No mention of resulting loss of life is made in either event.

In the first case in 1994 a wall made up of cantilever bored piles 1.2m diameter spaced at 1.5m was used to support an 8m excavation in ground comprising about 3m of fill over 3m of sandy clay over alluvial sands. The ground water was about 5m deep. The piles failed in bending leading to collapse and tilting of a 5 storey building about 12m from the excavation.

The second case in 1995 was a 13m excavation supported by a cantilever caisson wall in ground comprising 5m of fill over 7m of silty clay over strongly weathered siltstone. The ground water table is believed to have been about 4m deep. The wall collapsed suddenly leading to a one storey house 4.5m behind the wall collapsing into the excavation. Plate 3.2 shows the site after the collapse. The failure is ascribed to structural failure of the wall.

3.3 Analytical Study of Selected Case Histories

Several of the above case histories that have occurred in Hong Kong have been re-analysed. The purpose of this effort was to further investigate whether there was a design error and/or whether the analysis can sensibly replicate the observed failure mechanism. It must be emphasised that this re-analysis exercise cannot provide conclusive evidence for the behaviour of the retaining system. The exact details of the failure are rarely given, reference can be made to excessive displacement affecting roads and footpaths but rarely is the magnitude of the movement quoted. Details of the state of the strutting etc. at the time of the failure are also usually very approximate.

The computer program *Oasys* FREW has been used to re-analyse the failure and the data assumptions and the results are attached in Appendix B. Eight cases (numbers 2, 4, 8, 9, 11, 17, 27 and 29) were identified as being suitable for re-analysis.

In two of the six cases of excessive displacement, Cases 4 and 8, the back analysis shows that the design displacement could possibly have been sufficient to give rise to the observed effects. In Case 17 the analysis suggests the cause of the observed effects had very little to do with the wall or excavation. In other cases of excessive displacement construction problems associated with the strutting are cited as the cause and, while the analysis will show this effect for certain assumptions of poor strutting, generally there is not sufficient details of the non compliance of the strutting to be confident the analysis has identified the correct cause.

For cases of collapse the back analysis has also proved useful. In Case 2 modelling the reported omission of struts shows the wall is likely to suffer distress. For Case 27 (see Plates 3.3 and 3.4) the analysis shows collapse in much the same way as that reported.

To summarise, the above back analysis has proved useful mainly because it helps clarify what the expected effects are if the wall is constructed to the original design. It also confirmed that, for the two cases studied, the stated cause of failure is likely and that no other effect was necessary.

4. HAZARD IDENTIFICATION

The hazard is defined as an event that arises due to a deep excavation that could potentially lead to loss of life. Generally it takes the form of excessive displacement which is sufficient to lead to some secondary cause of a fatality or a collapse. A collapse is loss of ground causing a hole to appear at the ground surface adjoining the excavation. This ground loss could then lead to damage or destruction of adjacent existing buildings, falling of pedestrians or the fall of vehicles. Another possible consequence is casualties of workers within the excavation.

This section initially classifies the level or degree of hazard. It then discusses and classifies the observed cases of failure and assigns a level of hazard and the causes of failure. Finally it summarises the cause of failures and what mitigating measures could be taken.

4.1 Level of Hazard

The hazard has been divided into categories as follows.

(a) **excessive displacement** - situations where the movements are sufficient to cause unexpected disturbance to adjacent property, roads or services.

(b) **a small collapse**, represented by a plan area of 1 to 10m^2. This corresponds to a typical surface area of 2 by 3m, an area of 5m^2 or a volume of about 15m^3.

(c) **a medium collapse**, represented by a plan area of 10 to 100m^2. This corresponds to a typical surface area of 5 by 10m, an area of 50m^2 or a volume of about 150m^3.

(d) **a large collapse**, represented by a plan area in excess of 100m^2. This corresponds to a typical surface area of 10 by 20m, an area of 200m^2 or a volume of about 1,000m^3.

4.2 Classification of the Case Histories

The case histories have been reviewed with the objective of classifying the failure in terms of the hazard categories described in Section 4.1 above and to determine the cause of failure. Table 4.1 shows the results associated with private developments and government developments. It is shown that all the collapses are associated with sheet pile walls except for Case 27 which is a soldier pile wall and Case 33, a diaphragm wall. The only large collapse to occur was Case 5 on Queen's Road Central in 1981. Plate 4.1 shows this quite dramatic collapse.

Of the 12 sheet pile cases involving collapse 6 were caused by inadequate penetration and 5 by missing strutting. Two cases, namely Case 5 and Case 21, had both inadequate penetration and inadequate struts and in the other case the wall was probably not structurally adequate.

In the case histories quite a few involved damage to water mains leading to the presence of water. It is difficult to judge whether the failure was caused by the presence of water because in many cases a high water table was already present.

It is interesting to note that there are two cases of failures with open excavations, Cases 18 and 19, one being effectively a large trench and the other a base heave problem due to bearing capacity failure of the underlying soft clay. These cases are not considered further in this study.

4.3 Causes of Hazards and Mitigating Measures

Table 4.2 lists various causes that have been revealed by the study of the case histories.

In each case the case history numbers that suffered from the cause is given. It can be seen that poor workmanship of shoring and inadequate penetration of sheet piles are the dominant causes.

Mitigating measures are discussed for each of the causes. These are methods which, if followed, should help to reduce the rate at which the cause may lead to a failure in future.

5. FREQUENCY ESTIMATION

This section describes the approach adopted to estimate the frequency of failure from excavations for private projects. To determine the expected frequency of failures it is necessary to determine both the probability or rate at which failures are likely to occur and the total number of excavations that are likely in the future. To determine the probability at which failure are likely to occur we have studied the probability of failures that have occurred in the past. To do this both the total number of failures and the total number of excavations that were carried over the same time period are required.

This section begins by examining the total number of private excavation projects that have been undertaken in the past two decades. It then examines the total number of failures in private projects in the same period and derives the observed probability of failure. This is followed by a study of the future rate of excavations to then derive the expected frequency of failure in the future, assuming a similar trend as in the past.

5.1 Total Number of Private Excavation Projects in the Past Two Decades

The GEO have supplied a listing of all private projects that were referred to them, that involve support systems including sheet piling, diaphragm walls, pipe or soldier piles walls and caisson walls. Table 5.1 summarises this data up to and including 1995. Data since this time has been ignored as it not complete and will distort the yearly averages.

The data is extracted from the GEO's "Computerised Monitoring of GEO File Movements using Microcomputers" (COMMFI) database. The original objective of this system was to monitor the movements of District checking files. While it was therefore not originally designed for the purposes used in this study it is probably the best data source available. These figures only represent the number of excavation submissions referred from the Buildings Department to GEO for checking. It can reasonably be assumed however, that all major excavation works will be referred to GEO.

It must be noted that the year in Table 5.1 means the year when the first submission for the private development was reviewed by the BD and it is unknown when the excavation work actually started or was completed. For example the first submission could have been made in 1986, but the excavation might be finally designed in 1988 and the works commenced in 1989 and completed in 1990.

As the COMMFI was set up in 1984/85, the information on submissions before 1985 is far from complete. To compensate for this all the numbers between 1981 and 1984 have been doubled. While it is appreciated this is a rather arbitrary adjustment it can be seen that the resulting frequency of excavations in this period compare reasonably to the following years. There is very little data before 1981 and this has been ignored.

Having adjusted the frequency numbers as discussed above there are a total of 1787 projects listed. A plot of the number of projects and the types of retaining walls involved are given in Figure 5.1.

5.2 Observed Number and Distribution of Failures

The GEO report AR 2/92 lists 27 cases of collapse or excessive displacement associated with deep excavation work in private sites which involve collapses and excessive displacements over the last 20 years. In addition three more case histories have been identified. Table 4.1 gives a break down of damage classes of these incidents and a summary of the number involving private projects is shown in Table 5.2. It can be seen that there have been a total of 9 collapses and 16 cases of excessive displacement occurred between the year of 1981 to 1995 for private jobs.

The observed number of collapses or excessive displacements are plotted against year in Figure 5.2. As can be seen the rate of failures is quite variable with several cases occurring in the early 80's and again several cases between 1987 and 1991. The rate of failures against time will be expected to vary for a number of reasons. One reason may be variations in the economy which has a significant effect on the total number of projects under construction at any one time. Figure 5.1 shows how the total number of projects has varied since 1980 and this may reflect the economic activity at the time.

There are other factors however which include the change in legislation and geotechnical standards with regard to building control and checking activities. Special statutory and administrative measures were introduced in early 1990's to tighten geotechnical control of deep excavations in Hong Kong, including:-

(a) Buildings Ordinance (Chapter 123) was amended in 1990, namely (Administration) (Amendment) Regulations 1990, to require the submission of excavation and lateral support plans for building developments and this regulation was added to the BO under B(A)R8(1)(bc). With this regulation, requirements for qualified supervision can be imposed to enhance site supervision of deep excavations by qualified personnel.

(b) Practice Note for AP's and RSE's was issued in 1991 to specify the requirements for excavation and lateral support planes prescribed above.

(c) Administrative measures in GEO were introduced in 1992 by means of GEO Circular to tighten geotechnical control of deep excavations for private developments.

By comparing Figures 5.1 and 5.2 it can be seen that the number of failures since 1991 have decreased markedly despite the large increase in excavation projects which were first registered since 1990. This may imply the measures introduced by GEO have had a significant effect on the quality of building works especially with regard to construction control.

The observed failures have been correlated with other effects. Depth of excavation correlates reasonably with hazard level and this is discussed later in Section 5.4. There is no clear correlation between size of collapse and soil type however.

The overall distribution of failures by severity has been assessed and Table 5.2 shows a summary of the breakdown of collapses as a function of severity. As can be seen the overall distribution of collapses is about 45% small, 45% medium and 10% large.

5.3 Observed Probability of Failure

To determine the probability of failure the data in Table 5.2 need to be divided by those in Table 5.1. The results are shown in Table 5.3. Given that the data represents about 15 years of construction activity this leads to an annual average failure frequency of collapse or excessive displacement of around 1.7. This equates to a probability of failure of an excavation of about 0.0014 or a rate of 1.4 per thousand excavations. The division between excessive displacement and collapse is 0.009 and 0.005 respectively.

As discussed above it is likely that improvement in construction practices has brought about a reduction in failure probability over the years and therefore the failure probability derived from historical data may be conservative. It should also be noted, however, that many cases of excessive displacement are likely to not be reported to the authorities. Therefore the actual average annual frequency of excessive displacements is likely to be higher than the observed from the case histories and therefore the probability would also rise.

5.3.1 Breakdown of Probability by Wall Type

The failure probabilities have been derived from the analysis of the historical data as a function of wall type. Table 5.3 shows the resulting probabilities of collapse and excessive displacements for the various wall types. As can be seen the observed probability of excessive displacement is greatest for diaphragm walls at 29 in 1,000 reducing to 16 in 1,000 for sheet piles and much lower at 2 in 1,000 for caisson walls and soldier pile walls. The probability of collapse however is significantly higher for sheet pile walls (at 13 in 1,000) than the other wall types which are small or zero.

5.3.2 Breakdown of Probability with Time

As discussed previously the rate of failure has not been uniform in the 15 year time covered by the data and that the rate of failure has reduced in the more recent years. Comparing Figures 5.1 and 5.2 shows this quite clearly. To explore this aspect the data have been divided into three 5 year periods. These are 1981 to 1985, 1986 to 1990 and 1991 to 1995. Table 5.4 shows how the failure rate, including both excessive displacement and collapse, has varied in these periods.

It can be seen from Table 5.4 that the likelihood of failure since 1990 is about half of the average in the period of 1981 to 1995. It is also interesting that the probability of collapse has reduced by a factor of four whereas the probability of excessive displacements has reduced by about a third. It must be emphasised however that the number of incidents is very low with only one reported collapse since 1990. It is therefore recommended that, when exploring how the improved situation since 1990 could affect the predicted risk from future excavations, both the average failure probability since 1980 and half that probability be used.

5.4 Breakdown of Size of Collapse by Depth

The data on all excavations is only available in terms of wall type in GEO COMMFI database, while failure cases do include information on depth. Therefore subdivision of the failure probability into effects such as depth of excavation requires expert judgement to be applied to the hazard scenarios. The observed distribution of collapse is shown against depth in Table 5.5. The quantity of data is considered to be too small to be statistically significant however and therefore expert judgement has been used to derive an assumed breakdown for future works. This assumed breakdown is also shown in Table 5.5.

5.5 Number and Distribution of Future Excavations

The number of excavations can be obtained by examining the past records. It can be seen from Table 5.1, for example, that the total number is about 100 per year.

The distribution of past excavations by wall type can be obtained by examining the GEO records as summarised in Table 5.1. This shows a distribution of 37% sheet pile walls, 28% pipe pile or soldier pile walls, 8% diaphragm walls and 27% caisson walls. This however can not reflect the future distribution as caisson walls are no longer being used due to a change in legislation. The legislation was introduced as a result of the continuing number of fatalities, about 1 to 2 each year, that were occurring as a result of caisson construction. To estimate the future distribution it could be assumed that the caisson walls will be evenly distributed between sheet pile walls, pipe pile or soldier pile walls, large bored pile walls and diaphragm walls. This leads to a future distribution of 44% sheet pile walls, 35% pipe pile or soldier pile walls, 15% diaphragm walls and 7% large bored pile walls.

In order to address this and other issues the checking engineers at GEO were canvassed to obtain a summary of their experience over the past 2 to 3 years. The one page questionnaire is included in Appendix C. The respondents also included the number of excavations they had assessed. This was used to give a weighting to their answers prior to averaging them. It can be seen however that several respondent had assessed more than a hundred excavations and it is not clear whether the number of excavations is the number of actual cases or the number of submissions which are associated with an excavation. There are usually many submissions associated with a single project. In order to give reasonable weighting to all respondents but still allow more weight to those with many submissions the actual weighting number was based on ten times the logarithm of the original number. If the respondent stated he has less than 10 cases then the original number was used. The results of the survey, expressed in terms of the number of cases corresponding to each answer derived using this formula are given in Appendix C. The GEO questionnaire implies that the recent distribution of wall types is 52% sheet pile, 32% soldier pile or pipe pile, 11% diaphragm wall and 5% large diameter bored pile. This distribution is reasonable in comparison with the recorded distribution over the past 20 years discussed above and is adopted for the QRA calculation.

In addition to the wall type it is desirable to understand the depth ranges that are likely to be used for each of the wall types. This will allow the effect of depth to be considered to take account of the data trends shown in Table 5.5. The results of the questionnaire for depth varied for each wall type and the results are shown in Table 5.6.

It is noteworthy that a third of all design work is for excavations less than 5m deep.

5.6 Frequency of Expected Failures in the Future

The overall frequency of failure is calculated by combining the probability of failure with the anticipated number of deep excavations. For the purposes of this study the calculation is carried out assuming there will be 100 deep excavations per year.

Table 5.7 shows the derivation and apportionment of the annual frequency of excessive displacement for the range of wall types. The probabilities of excessive displacement are not the same as those derived in Table 5.3. This is because of the limited number of data and statistical methods have been used to derive the best estimate of the probabilities. Appendix D shows how these statistical methods have been used to generate these probability rates. The probability of excessive displacement occurring for the large bored pile walls has been assumed to be the same as that for soldier pile walls.

Table 5.8 shows the derivation of frequency of collapse for a range of conditions. Again the probabilities are not the same as those derived in Table 5.3 and Appendix D shows the statistical methods that have been used to generate these probabilities of collapse. The probability of collapse occurring for the large bored pile walls has been assumed to be the same as that for diaphragm walls.

All the above discussion is incorporated into an event tree in Figure 5.3. This shows the expected frequency of excavation wall types, depth ranges and failures. The probabilities of failures correspond to the values given in Tables 5.7 and 5.8.

6. CONSEQUENCE ASSESSMENT

This section examines the consequences of failure - collapse and excessive displacement - by modelling the various outcome scenarios and the potential for fatalities. The number of fatalities are then estimated for each outcome scenario. As explained earlier, the analysis is largely based on expert judgement.

6.1 Consequence Scenarios

The various outcomes following a collapse or excessive displacement are modelled using an event tree approach.

The technique of event tree is used to describe and analyse how an initiating event may lead to a number of different outcomes. Probabilities are assigned to the intermediate stages in the development of the initiating event. The frequency of occurrence of a hazardous outcome is then a product of the frequency of occurrence of the initiating event and the probabilities that the event develops to that outcome.

Event trees are drawn for the following initiating events:

- collapse of an excavation adjoining a building;
- failure of an excavation adjoining a road;
- collapse of an excavation affecting workers.

The event trees for collapses model the outcomes depending on what is adjacent to the excavation. It is assumed that an excavation is adjacent to a road or a building with equal probability. Excavations are more likely to be adjoining both road and building. However, the possibility of a failure affecting both road and building simultaneously is considered to be low. However, if the potential for such an outcome is considered to be significant, it can be accounted for by factoring up the initiating event frequency by 10 to 20%. Excavation work may also be undertaken in areas where there are no facilities such as roads or buildings immediately adjoining (for example, in the Kowloon station area of airport railway). However, this is not considered (due to insufficient data and even if there is data, the proportion of such excavations may form less than 5%).

6.1.1 Collapse of an Excavation Adjoining a Building

This event tree models the potential for affecting buildings adjoining an excavation in the event of a collapse - small, medium or large. Figures 6.1 to 6.3 show the event trees.

The effect on adjoining buildings will depend on the type of building. Three types of building are considered as follows.

> *tower block* which represents modern multi-storey buildings greater than 10 storeys, whether residential or commercial. It is assumed that all such buildings will be on pile foundation. The probability of structure partial failure or non-structure failure is considered very low for such buildings. Different probabilities are assigned for small, medium and large collapses.
>
> *medium rise buildings* include buildings that have 5 to 10 storeys, whether residential or commercial. These could be constructed on pile foundation or pad foundation. Both cases

are considered separately. The probability of structure partial failure or non-structure failure is considered very low for buildings on piles, but could be higher for buildings on pad foundation. Buildings on pad foundation could also suffer total structure failure in the event of a large collapse.

low rise buildings include buildings with less than 5 storeys. These could be village type houses or low rise luxury type houses. These buildings are considered to be the most vulnerable to total structure failure, partial structure failure and non-structure failure.

The structure of the event tree is same for small, medium and large collapse but the branch probabilities are different.

6.1.2 Failure of an Excavation Adjoining a Road

Collapse of an excavation adjoining road can result in multiple effects. A collapse can affect

- footpath,
- road and
- utility services.

All of the above may occur simultaneously.

As mentioned earlier, the extent of failure for the 3 types of collapses - small, medium and large are considered as follows:

- Small collapse - volume of $15m^3$, ground surface area of $5m^2$, ie 3m by 2m;
- Medium collapse - volume of $150m^3$, ground surface area of $50m^2$, ie 10m by 5m;
- Large collapse - volume of $1,000m^3$, ground surface area of $200m^2$, ie 20m by 10m.

Based on these dimensions, all 3 types of collapse are assumed to affect a footpath which is generally 2m wide (assuming that all roads have a footpath and that the boundary of an excavation lies on the edge of the footpath). A small collapse is therefore, unlikely to affect the road. A medium collapse may affect up to one lane while a large collapse may affect up to 2 lanes.

It is also possible that a large collapse may affect a building on the other side of a road if the road width is <10m. There are some roads with width <10m (a very small proportion though) but the effect of a collapse on a building on the other side of the road is considered to be not significant.

Footpaths

Figure 6.4 shows an event tree constructed to model the outcome of a collapse affecting footpath. The event tree considers whether it results in pedestrian fall (which would depend on time of day and usage of footpath etc). The type of collapse (each with different dimensions) would determine the number of pedestrians at risk (a larger areas exposes more people to risk) and therefore each type of collapse is modelled separately.

Road Traffic

Figure 6.5 shows an event tree for collapse affecting road traffic. As small collapses are not expected to affect the road, event trees are drawn only for medium and large collapses to consider whether it could result in a road vehicle fall. The estimation of injury/fatality is considered later.

3 types of road are considered.

- Type A with 500 vehicles/lane/day (equal to AADT of 1,000)

- Type B with 2,500 vehicles/lane/day (equal to AADT to 5,000)

- Type C with 5,000 vehicles/lane/day (equal to AADT of 10,000).

Since collapses are not expected to affect more than two lanes of a road, the above cases would also represent 4 lane roads (4 lane roads generally have an AADT of 20,000 and above which is equal to about 5,000 vehicles/lane). In the case of major roads represented by AADT of 10,000 and more, a large collapse may affect 2 lanes whereas a medium collapse is likely to affect one lane.

Utility services

Utility services such as water mains and gas pipes are generally installed below the footpath or in the lane closest to the footpath, these could also be affected as follows.

If a water main fails (such failures could include cracks resulting in seepage, bursting of mains), it could have potential to aggravate the collapse from say, a small collapse to a large collapse. The process of infiltration and saturation following seepage or failure could have a delayed effect such that although symptoms of failure may be known, no emergency action such as isolating the section of road is taken. While the potential for escalation exists, it is assumed that the early symptoms of failure will be recognised and due emergency action taken to prevent fatalities due to escalation.

A collapse or excessive displacement may cause failure of the gas pipe. This is modelled separately to examine the potential for a gas release as shown in Figure 6.6. The possibility of a gas release to ignite and cause multiple fatalities is explored in Figure 6.7.

6.1.3 Collapse of an Excavation Affectng Workers

Figure 6.8 shows an event tree for collapse of an excavation affecting workers. This event tree models the potential for workers within the excavation to be affected in the event of collapse - small, medium or large. This will depend on whether workers are present in the vicinity of failure and are able to escape. Escape probabilities depend on the size of collapse in relation to the size of excavation since a large collapse in a small excavation has greater potential to affect workers than a large collapse in a large excavation.

The effects of collapse on workers include fall from heights , buried by debris, injury due to falling material such as wall or struts.

6.2 Estimation of Fatalities

The number of fatalities for each outcome scenario have been estimated based on judgement.

In order to account for uncertainty in the estimation of fatalities (particularly multiple fatalities), probabilities corresponding to various estimates on number of fatalities have been assigned.

The range of fatalities considered are:

- 1 (average 1)
- 2 to 3 (average 2)
- 4 to 10 (average 6)
- 11 to 30 (average 18)
- 31 to 100 (average 60)

The number of fatalities and their probability of occurring arising from a range of scenarios is discussed below.

6.2.1 Fatalities from Building Damage

Table 6.1 shows the number of fatalities and the associated probabilities for partial collapse of a tower block, a medium rise building and a low rise building.

The number of fatalities and the associated probabilities are also predicted for total collapse of a medium rise buildings and low rise buildings. These have been based on the observations from earthquakes. The Applied Technology Council report ATC-13 (1985) reviewed previous studies and concluded that total collapse of a structure causes an average of 20% fatalities, 40% serious injuries and 40% minor injuries. Based on an average maximum occupancy for a medium rise of 168 people (7 floors x 8 apartments x 3 people) and for a low rise of 36 people (3 floors x 4 apartments x 3 people) and that the building is 100% occupied for half the time and 20% occupied for the remaining time, leads to the distribution of fatalities shown in Table 6.1.

6.2.2 Fatalities from Vehicle Fall

A similar set of fatality probabilities is derived for vehicle fall. This is shown in Table 6.2. While deriving the potential for multiple fatalities, the average occupancy in vehicles is considered. The average number of people in car/taxi is 2 while the average number of people in bus is 20 (based on Traffic Census data). The average number of people in a mini bus is assumed as 4 to 10. The proportion of buses versus car/taxis on road is 1:4 (based on Traffic Census data). The proportion for buses is split into bus and mini bus.

Please note that the overall fatality rate per vehicle fall is equal to 0.117. This is similar to the observed rate of fatalities of 0.08 derived as a fraction of the total number of fatalities due to road accidents to the total number of fatal plus serious road accidents as given by the Transport Department.

6.2.3 Fatalities from Pedestrian Fall

It is recognised that frequency of usage of footpath is related to the location of road and footpath more than the type of road. The usage is generally higher in built-up areas like Kowloon and Hong Kong Island and near residential developments irrespective of whether it is a major road or a minor road but it is difficult to estimate the proportion of roads and associated footpaths which are more likely to be used than others.

Pedestrian density can be estimated based on The Transport Department Design Manual which provides guidance on design of footpaths. The pedestrian volume is given as 75 to 150 pedestrians per minute in urban areas. Assuming a value of 75 pedestrians/minute, expected number of persons in 10m section is estimated as

$$N = (L \times F)/V$$

L = length of footpath

V = walking speed = 4.5 km/hr = 4500 m/hr

F = pedestrian volume

$$N = 10 \times 75 \times 60 / 4500 = 10 \text{ pedestrian per 10m, i.e. 1 per 1 m}$$

However, the value given above corresponds to the design level of usage which may exist at most during peak hours and not otherwise. Therefore, a value corresponding to 10% of this value may be assumed as an average case. A 20m section will have 2, a 10m section will have 1 and a 3m section at most one person.

These values have been used in considering the maximum number of pedestrians exposed to the hazard. The overall probabilities of the number of fatalities due to a pedestrian fall incident for the various levels of collapse are shown in Table 6.3.

6.2.4 Fatalities Associated with a Gas Pipe Failure

The potential effects from a flash fire/jet fire will depend on the nature of release (ie, the pressure within the pipeline and the extent of damage to the pipeline etc), the time of day etc. Time of day will influence the number of persons exposed to the hazard. Accordingly, values for probability of fatality are derived for day and night conditions separately as given in Table 6.4. No detailed modelling of fire effects have been undertaken. Generally the impact from the failure of smaller gas pipes (operating at low pressures) is low.

6.2.5 Fatalities from Workers Exposed to the Hazard

The number of workers who could be present in the vicinity of failure is estimated based on the following:

Density of workers = 5 per 100m^2

Large collapse = 20m x 10m = 200m^2	Number workers = 10 people
Medium collapse = 10m x 5m = 50m^2	Number workers = 2.5 people
Small collapse = 3m x 2m = 6m^2	Number workers = 0.3 people

The above values are based on the assumption that the impact area due to debris is the same as the area of the collapse at the ground surface (see Section 4.1). It is recognised that the runout distances could be lager in some cases. The estimates of the likelihood of various numbers of fatalities of workers from collapse of a deep excavation are shown in Table 6.5.

6.3 Check on Past Failures

As a check on the reasonableness of the fatalities implied by the event trees an analysis has been made of the 9 cases of collapse that have been observed in private projects between 1981 and 1995. The predicted loss of life is 0.40 lives lost. Given that there has been no loss of life from these incidents this is a credible result. In fact any result between 0 and up to about 2 lives lost would be credible in that it would not be inconsistent with the data.

7. RISK SUMMATION

The overall risk result is a summation of the frequency of the hazardous outcome multiplied by the number of fatalities together with the associated probabilities of the various numbers of fatalities for each hazardous outcome. The results are presented initially in terms of the Potential Loss of Life (PLL) and then by F-N curves.

The results presented in this section all relate to the average probability rates derived form data from excavations in the 15 years from 1981 to 1996. If just the most recent 5 years of data were used from 1991 to 1996 then the failure rates would be about half of those reported here.

7.1 Potential Loss of Life (PLL)

7.1.1 Overall PLL

The overall Potential Loss of Life (PLL) per year is estimated as 0.021 per year (ie, 1 in 50 years) for the public and 0.010 (ie, 1 in 100 years) for workers as given in Table 7.1.

A breakdown of PLL by type of failure, by type of wall, by depth of excavation and by type of facility affected are given in Table 7.2 to Table 7.5.

From Table 7.2 to Table 7.5, it can be seen that:

* large collapse contributes the maximum to the PLL for the public (64%);

* sheet pile walls contribute the maximum to the PLL (76%) of the various wall types. This is expected since the sheet pile wall failure probability (0.015) is 5 times higher than other wall types (0.003). Also, excavations using sheet pile constitute about 52% of the total number of deep excavations.

* with regard to the type of facility affected, low rise building failures contribute the maximum to the PLL (61%) followed by pedestrian fall (17%). Failures involving medium rise buildings on pads constitute 11% of the overall PLL. The contribution to PLL from road traffic is about 9%.

7.1.2 Range of Results for Individual Excavations

The risk resulting from an individual excavation has also been explored. The variation of PLL for the four wall types for the two depth ranges are shown in Table 7.6. These values have been derived for the full range of scenarios of development adjecent to the walls each with their appropriate probability of occurrence.

To further explore the range of results various specific combinations of wall type, excavation depth and surrounding building types have been investigated. The results are shown in Table 7.7. It can be seen that the range of results varies from 0.15×10^{-4} to 15×10^{-4}. This represents a one hundred times variation in risk.

7.2 F-N Curves

The overall F-N curves for future excavations based on 100 excavations per year are shown in Figure 7.1. This shows that there is about a 1 in 100 chance per year of an incident causing 1 or more fatalities. There is about a 1 in 800 chance per year of causing an incident leading to 5 or more fatalities and a 1 in 10,000 chance per year of causing an incident leading to more than 20 fatalities.

The F-N curves, in terms of risk to the public have also been derived for individual excavations for different wall types and these are shown in Figure 7.2 for depths of excavation less than 10m and Figure 7.3 for depths of excavation greater than 10m. Please note that these curves are for each excavation and are not the total number expected in any one year. The corresponding PLL values to these curves are listed in Table 7.6. As expected the curves for sheet pile walls are noticeably higher than those for the other wall types.

Individual specific cases have also been investigated and these results are shown in Figure 7.4. These are the same cases as those listed in Table 7.7 and the corresponding PLL values are listed in that table. Again it is assumed that half of the excavation is bounded by a road Type B. As discussed above these cases show the range of results that can be expected for an individual excavation.

7.3 Comparison with Risk from Man Made Slopes and Retaining Walls

The GEO have published the results of an extensive study of the PLL of pre GCO man made slopes and retaining walls (GEO 1996). Three types of features, namely cut slopes, fill slopes and retaining walls were considered with facilities at their toe and/or crest. It is considered most sensible to compare the risk calculated from this study with those for buildings and roads adjoining the crest of a slope or retaining wall feature. For the purposes of comparison it is considered that a Group 1 building in that GEO report most closely matches the definition of a building used in this study on deep excavations. With regard to roads used in this study, it compares well to those roads in Group numbers 1, 2 and 3 in the GEO report.

7.3.1 Overall PLL to the Public

The GEO report calculates an average annual PLL for a building at the crest of a man made slope or retaining wall of 0.90 fatalities per year and for a road, 1.84 fatalities per year. These compare with annual fatality rates predicted here for deep excavations for a building of 0.015 and for a road adjoining an excavation of 0.006. Therefore the overall risk to the public from buidings and roads is over 100 times greater from pre GCO man made slopes and retaining walls than it is from deep excavations. If buildings and roads at the toe of a man made slopes or retaining walls are included, the risk is calculated to increase to over 500 times that from deep excavations.

These values provide an overall comparison of the derived PLL values for excavations and pre GCO man made slopes. It should be noted however that the number of features considered in each case are very different. There are several thousand significant pre GCO man made features while the number of excavation features is assumed to be only 100 per year. The following sections therefore provide a comparison of the risk to the public on a feature by feature basis.

7.3.2 PLL for Individual Features

The overall annual comparison shown above is not necessarily a meaningful comparison as, in any one year, the proportion of the population affected by deep excavations is much less than that affected by man made slopes and retaining walls. Therefore the calculated risk arising from any individual deep excavation has also been compared with the average predicted for individual man made slopes and retaining walls.

The GEO report calculates an average annual PLL for a building features at the crest of a man made slope or retaining wall of 1.87×10^{-4} fatalities per year and for roads, 1.77×10^{-4} fatalities per year. These compare with values predicted here for deep excavations for building features of 1.5×10^{-4} and for roads adjoining an excavation of 0.6×10^{-4}.

The order of risk, in terms of PLL, therefore appears to be similar for an individual slope feature and a single deep excavation. The risk values are expressed in terms of equivalent fatalities. There is an important difference however between the two types of feature. In an excavation the risk is transient in that it occurs once at a critical stage in the construction. When this stage is passed the risk is greatly reduced probably to near zero. For slope or retaining wall features, the risk values are permanent and therefore an annual risk that will exist throughout the life of the feature.

7.3.3 F-N Curves for Individual Features

Figure 7.5 compares the average F-N curve for a deep excavation with that derived for some sites with a history of failure in Hong Kong. It can be seen that the average F-N curve for a deep excavation is generally lower than that for these slope features. The worst case of a sheet pile wall for an excavation > 10m deep adjoining a low rise building (see Figure 7.4) is less than that for the Sau Mau Ping and Kwun Lung Lau sites. Again it must be emphasised that the landslip F-N curves are for permanent annual risk whereas the F-N curve for a deep excavation is for one transient situation.

7.4 Uncertainty and Sensitivity Tests

One approach to consider uncertainties is to consider for each of the factors that affect the outcome, an average value, best case value and a worst case value and then derive an average, best case and worst case result. While this approach gives a measure of the maximum uncertainty in the analysis, it also gives a very wide spread of results which are not very useful but instead affect the credibility of the output.

A more rigorous approach to modelling the uncertainty is to consider a distribution of probabilities corresponding to the best, average and the worst case value for each factor. For example, the probability of total structure failure due to collapse may be represented as a distribution instead of a point value as adopted in the conventional analysis. This necessitates the use of a numerical solution to model a large number of combinations of the decision probabilities using a sampling method such as Monte Carlo to calculate a probability distribution function of the desired results. The probability distribution function can then be analysed to calculate the mean and variance of the distributions of the results of interest.

A sensitivity analysis has been performed using Crystal Ball, Version 4, developed by Decisioneering Inc. Crystall Ball is a forecasting and risk analysis program requiring inputs in spreadsheet format. Crystal Ball forecasts the entire range of results possible for a given situation through Monte Carlo simulation, wherein the Monte Carlo method is used to generate random numbers (in this case selected from a table to conform to a probability distribution).

The simulation generates a sensitivity chart which ranks the assumptions (ie, various parameters in the event tree) according to their importance in forecast (ie in risk result). Crystal Ball calculates sensitivity by computing rank correlation coefficients between every assumption and forecast. The larger the absolute value of the correlation coefficient, the stronger the relationship. The simulation accuracy can be increased by increasing the sample size. A sample size of 10,000 has been used.

The probability distribution assumed is 0.6 for the 'most likely value' and 0.2 each for the 'lower bound' and 'upper bound' estimates. The 'most likely' value, the 'lower bound' and 'upper bound' estimates for all the input parameters in the event tree model are listed in Table 7.8.

The simulation results are given only for risks to the public. The PLL results from the simulation are summarised below:

Median value for PLL (public) : 0.032 per year

Mean value for PLL (public) : 0.037 per year

It can be said with 90% confidence that the PLL value lies in the range 0.013 to 0.075. Figure 7.6 illustrates the variation in the predicted PLL values. The 90% confidence bounds have also been estimated for the F-N curve and these are shown together with the median and mean values in Figure 7.7.

A sensitivity chart is attached in Figure 7.8. The five parameters whose uncertainty has the most influence on the risk results are :

- number of excavations per year;
- probability of collapse given an excavation with a sheet pile wall;
- probability of fatality due to partial collapse of a low rise building;

- probability of probability of collapse given an excavation with a soldier pile/pipe pile wall;

- probability of fatality due to total collapse of a low rise building.

The ranking of importance is useful in identifying which of the parameters' estimates require to be refined.

7.5 Significance of Results

One approach for interpreting the results is to use the PLL value derived in this study to determine the maximum justifiable spend on additional measures for risk reduction based on statistical value of life considerations. A 'statistical value of life' of HK$24 million is adopted based on a recent study by GEO. It is also common to adopt aversion factors up to 20 to represent society's strong aversion to multiple fatality events.

Maximum justifiable spend = Total estimated PLL x HK$24 million x aversion factor of 20

Considering only PLL to the public,

Maximum justifiable spend = 0.020 x HK$24M x 20
= HK$10M per year

If no aversion factor is applied, then,

Maximum justifiable spend = 0.020 x HK$24M
= HK$480,000 per year

The above would provide the limit of spend based on estimated PLL. It will be useful to carry out a more rigorous cost-benefit analysis. The costs could include existing and additional staffing costs within GEO for approval of excavation design work, inspection of site works, additional staffing costs for contractors to tighten supervision etc which can then be compared with benefits based on lives saved, injuries prevented, damage averted etc. This is beyond the scope of this study.

8. CONCLUSIONS AND RECOMMENDATIONS

The quantitative risk assessment presented in this report for fatalities arising from deep excavation in Hong Kong leads to the following conclusions and recommendations.

(1) The risk to life is calculated to give a PLL of about between 0.015 and 0.03 per year. This assumes about 100 deep excavations are carried out each year within the Special Administrative Region and includes the risk to workers. If workers are not considered, the risk to the public alone reduces to about two thirds of these values.

(2) The higher of the above range of results comes from the average rate of failures observed since 1980. Government control has significantly improved since 1990 and, if this is taken into account, the lower figure is more appropriate. It follows that for future excavations the lower figure is probably the most realistic.

(3) The study shows that the contribution to the risk is significantly higher for sheet pile walls than for other types of walls. Examination of case histories of failure shows this is mainly due to inadequate penetration due to obstructions and inadequate strutting.

(4) A dominant cause of observed problems is poor site control. Occasionally, for cases of excessive displacement, the cause is due to inadequate planning.

(5) The study also shows that the public are more at risk from buildings on pad foundations adjacent to excavations collapsing. Risk to pedestrian is the next main contribution.

(6) Measures should be instigated or investigated to help reduce the risk. These include:

Improved site control

(i) Lack of penetration of sheet piles is difficult to control effectively without a continuous presence on site. Measures such as pre-marking lengths on the piles will be effective but only with frequent inspection.

(ii) There are several cases where strutting has either been omitted or not installed with sufficient care ensure the struts are tight to the wall etc. Random site visits by GEO, or other parties, would help improve this situation.

(iii) There have been a couple of cases of failure due to lagging between soldier piles. Local excavation for lagging of soldier piles should be carefully controlled.

(iv) It is observed that large movements can occur due to the installation of diaphragm walls. To some extent this may be unavoidable but care must be taken to maintain the slurry head pressure etc.

Improved planning

(i) Lack of penetration of sheet piles is often caused by obstructions in the ground including boulders of less weathered residual material. If these conditions are suspected at the design stage then care should be taken to ensure that the construction contract has sufficient provision for predrilling, or other ground treatment, to effectively overcome these problems.

(ii) In some instances quite large deflection of wall and associated settlements would have been predicted by the design. Nevertheless when the excavation was carried out excessive displacement leading to an incident report was observed. It is suspected that, in some cases, if the expected movement had been planned for, and remedial measures in place, these cases would not have been reported as being a problem. It must be emphasised that, in many situations, it is almost impossible to reduce movement to a small value (e.g. <25mm). In these cases adequate preparation for movement is preferable to taking extreme and expensive measures to attempt to reduce the movement.

(iii) On a few occasions large movements have been observed during the pumping tests. This may only be able to be avoided by installing strutting prior to the test. Some form of staged testing regime may therefore be required. Depending on how the contract is setup this may require a change in construction sequence and could be considered as a design issue.

Routine monitoring

In the cases of a medium or large collapse the PLL will be dramatically reduced if there is some warning of the collapse. Therefore for deep excavation adjacent to buildings it is advisable that some monitoring be in place which is routinely checked. Line and level survey will generally be sufficient for this.

(7) The estimated risk has been compared to that estimated previously from pre GCO man made slopes and retaining walls. The overall risk from deep excavations in the Hong Kong Special Administrative Region is many times less than that from slopes. The risk from an individual excavation however is the same order as the annual risk from a slope feature. For example the worst combination for an individual excavation, namely a deep sheet pile supported excavation adjacent to a building on pad foundations, is comparable to the annual risk calculated for some sites with a history of failure. It must be noted however that the risk from an excavation is transient whereas for a slope it is effectively permanent over many years.

(8) To improve the estimation of risk in the future it is sensible that the reporting of failures be more systematic. The report should include, as a minimum, photographs of the collapse or distress, measurement of the affected area, recording of the exact date and time of the incident (if it is a collapse). For cases of inadequate propping, careful recording of the details of the popping together with photographs would be useful.

(9) For major collapse involving loss of life or serious disruption to adjoining works it is recommended that a more detail investigation be made in a similar manner to a forensic investigation of a fatal slope failure.

(10) There is also a problem that some failures are not reported to the GEO. The best way to reduce this is to make it a requirement in qualified supervision impose under BO17(1)6 that all collapses be reported to the GEO within a specified period of their occurrence.

(11) With the agreement of the GEO several types of excavations have been ignored in this study. These include deep excavation adjacent to steeply sloping sites, open excavations and trenches. The study could be extended to encompass these areas.

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LIST OF TABLES

Table 3.1 - Summary Table of GEO Report No. AR 2/92 'Review of Collapse and Excessive Deformation of Excavations'

Case No.	Total Designed Excavation Depth (m)	Wall Type	No. of Designed Levels of Struts	Ground Conditions	Designed Depth of Ground Water Table (m)	Depth at Time of Failure (m)	Year of Event	Classification of Failure Collapse (C) or Excessive Displacement (ED)	Failure Description	Stated Probable Cause	Water Pipe	Comments by ARUP	FREW Analysis by ARUP
1	7.8	?	?	7.5m Fill MD	1.5	0	1989	ED	50mm settlement and tilting	Groundwater drawdown	-	Dewatering due to caisson excavation	N
2	8.3	ShP	5	1.5m-3m Fill 3.5m-8m Colluvium 13m CDG/HDG	1.7	4	1990	C Medium	Road collapse	Strut omitted	100mm pipe burst	Inadequate struts – details of omitted struts given in report	Y
3	9.5	ShP	5	3.1m Fill 2m MD 10m CDG/MDG	2.5	7.5	1990	C Medium	Road collapse	Inadequate penetration of sheetpile wall	150mm main damaged	Excavation in progress – profile of sheet piles shown together with soil data	N
4	26.5	ShP+ DW	6	5m Fill 12.3m MD 18m CDG	3.0	?	1990	ED	Tilted building	Groundwater drawdown	Leakage	Predicted settlement 158.5mm	Y
5	11	ShP	2	5m Fill/ Alluvium/MD 0.5-4m CDG	2.5	10	1981	C Large	Road collapse	- Removal of strut; - Inadequate penetration of sheetpile	375mm main damaged	Struts near toe of SP wall removed 36 hours before collapse leading to toe collapse	N
6	6	ShP	4	7-12m Fill/Colluvium CDG	3 to 4.5	10	1987	ED	Excessive ground settlements and cracks along Stanley Street	Cofferdam work to remove obstructions	Leakage	Reference to existing wall. Sheet piles YSP III. Total excavation depth ~16m but 6m already existed. Row of sheet piles outside existing retaining wall	N
7	5	DW	Nil	4m Fill 8m Colluvium 3m CDG	4	0	1985	ED	Cracks on road surface	DW installation	450mm asbestos salt main broken	Minor damage	N
8	8.1	ShP	4	2m Fill 7m MD CDG	1.3	8.1	1988	ED	Road settlement	ShP deflection due to poor workmanship of strutting	-	Estimated settlement 83.6mm	Y
9	8.8	ShP	3	9m Fill 4.5m MD	2.5	8.8	1988	ED	Pavement damaged	Large spacing of walings (3rd level of strut)	Water main burst	Settlement prediction - 57.8mm 133mm measured – leading to damage on road	Y
10	8	ShP	2	8m Fill 16m MD	2.5	9	1988	ED	Ground settled and moved laterally	- Improper design of recharge wells – Soil weaker than expected	-	Result in 100mm settlement	N
11	10.5	ShP	5	6m Fill 6m MD Alluvium	3.8	7	1987	ED	Excess road settlement, building tilt	Poor workmanship in shoring installation	-	YSPII, crack widening on concrete road, estimated 98.1mm settlement due to omission of 3rd layer of strut – could have behaved as expected – appears propping not tight	Y

Table 3.1 - Summary Table of GEO Report No. AR 2/92 'Review of Collapse and Excessive Deformation of Excavations' (Cont.)

Case No.	Total Designed Excavation Depth (m)	Wall Type	No. of Designed Levels of Struts	Ground Conditions	Designed Depth of Ground Water Table (m)	Depth at Time of Failure (m)	Year of Event	Classification of Failure Collapse (C) or Excessive Displacement (ED)	Failure Description	Stated Probable Cause	Water Pipe	Comments by ARUP	FREW Analysis by ARUP
12	8.5	ShP	4	5-9m Fill 11-17m Alluvium	3.3	8.5	1987	C Small	Loss of ground	Inadequate penetration of sheetpile due to obstruction	-	Loss of ground through a gap in sheet piles leaving a void behind	N
13	6.2	ShP	2	8m Fill 6m MD Alluvium	2.6	6.2	1980	C Small	- ShP tilted & struts broken – Ground settlement	Inadequate penetration of sheetpile	Leakage burst	Loss of ground leading to collapse + burst of water main. ShP not driven to base of excavation in some places.	N
14	26.5	DW	6	11m Fill 6m MD 20m Alluvium 24m CDG	3	0	1990	ED	Excessive settlement and cracks on carriage way	Dewatering for caisson excavation	Small pipe broken	Large settlement (140mm) due to pumping test	N
15	2	ShP	?	No details	3	Slope	1987	C Small	Sheetpile collapse	?	-	No details (HyD site)	N
16	12.5	ShP	4	10m Fill CDG	?	12.5	1981	C Medium	Sheetpile collapse and rupture of water mains	Over excavation + strutting undersized	Rupture of 450mm water main	Over excavation and undersized struts are claimed but no details are given. A steep high (12m) berm was used. (Housing Dept. site)	N
17	7.3	ShP	3	0-2m Fill 2-8m Alluvium 8-20m CDG	3.0	0	1990	ED	Footbridge settled and tilt	Dewatering for caisson excavation	-	Horizontal and vertical displacement due to dewatering	Y
18	5	None	0	5-7m Fill 11-16m MClay 2-5m MSand 1-4m Alluvium 3-12 CDG/HDG	1.6	5	1990	ED	Lateral movement of H-piles up to 1.2m	Slope and bearing capacity failure	-	Sideways movement of an open cut excavation over 15m of soft clay due to bearing capacity failure (WSD site)	N
19	6.5	None	0	3m Fill 4-6m Seawall trench fill 2.8m MD	3.0	6.5	1987	C Small	Building damage with part of floor slab cracked & collapsed	Slope failure due to no support	-	Good sketch of damage area; collapse of open trench (Public site)	N
20	6.2	Soldier Piles	0	Fill	10-15	?	1982	ED	Cracks on the road behind the soldier piles	Poor slope surface protection + surcharge	-	Problem occurred during recompaction works	N
21	7.2	ShP	4	Fill MD CDG	1.5	6	1981	C Small	Excessive deflection of ShP	- Inadequate penetration of ShP, - Insufficient strut support	-	Single level of prop used instead of 4 levels shown in design drawings	N
22	8.5	ShP	4	5.5m Fill 3m MD 12m CDG	2.0	8.8	1982	C Medium	A hole (6x4x1.5) is formed cause road collapses	Inadequate penetration of ShP	Burst	Sheet pile toes exposed	N

Table 3.1 - Summary Table of GEO Report No. AR 2/92 ‘Review of Collapse and Excessive Deformation of Excavations’ (Cont.)

Case No.	Total Designed Excavation Depth (m)	Wall Type	No. of Designed Levels of Struts	Ground Conditions	Designed Depth of Ground Water Table (m)	Depth at Time of Failure (m)	Year of Event	Classification of Failure Collapse (C) or Excessive Displacement (ED)	Failure Description	Stated Probable Cause	Water Pipe	Comments by ARUP	FREW Analysis by ARUP
23	4.7	Channel Planking	3	4.6m Fill 2.4m Alluvium 2.5m CDG	1.0	4.5	1990	C Small	Service lane collapse	Strut ommitted	3 pipes damaged	Unsupported cutting	N
24	7	ShP	3	4-6m Fill 4m Alluvium CDG	1.0	7	1989	C Small	Shp collapse	Over excavation before installation of the lower struts	2 pipes burst	Appears footpath collapse only because of watermain burst	N
25	9	Soldier Piles	3	Compacted Fill/Alluvium/ Colluvium	Dry	9	1989	ED	Severe distress to village buildings, cracks on pavement	Temporary works not built as planned	-	There is dispute as to whether excessive displacement actually took place (TDD site)	N
26	4.7	ShP	1	Alluvium CDV	Dry	9	1989	ED	Steel struts deflected 100mm + cracks	Inadequate support system	-	Struts inadequate, could be design error (WSD site)	N
27	14	Soldier Piles	6	2-4m Fill 1.5m-4.5m MD 0-7m CDG	1.5	14	1991	C Medium	Collapse under road and building	Over excavation and poor grout curtain	4 pipes broken	Separate GEO report issued Excessive excavation for lagging	Y
28	9	ShP	2	Colluvium	N/A	Slope	1980	C Small	Collapse	- Poor workmanship of shoring, - Not constructed in accordance with design	-	Sturtting unstable because of steep raking strut and no tension capacity in wall	N
29	4.9	Planking	4	Fill	1.2	-	1991	ED	0.3m settlement	Strut omitted	Fresh watermains, salt and drainage pipes damaged	Estimated deflection is 42.3mm in original design	Y
30	4.5	Planking	3	6m Fill 3m MD 1-2m Alluvium	2.0	-	1991	ED	Deterioration of building, cracks in floor	Struts undersized	-		N
31	3.7	Planking	3	3m Fill CDG	3	3.7	1991	ED	Cracks in adjacent fence and open yard	- Poor workmanship of struts, - Not constructed in accordance with design	-		N

Table 3.2 - Summary Table of Other Cases of Collapse and Excessive Deformation of Excavations in Hong Kong

Case No.	Total Excavation Depth (m)	Wall Type	No. of Designed Levels of Struts	Ground Conditions	Designed Depth of Ground Water Table (m)	Depth at Time of Failure (m)	Year of Event	Classification of Failure Mode Collapse (C) or Excessive Displacement (ED)	Failure Description	Cause	Water Pipe	Comments by ARUP	Source	FREW Analysis by ARUP
32	7	ShP	4		-	7	1993	ED	600mm ground subsidence	Late installation of lateral support	Pipes leaking		GEO	N
33	60	DW	Circular excavation	18m Fill 18m MClay CDG	3.0	0	1995	C Medium	Collapse of ground crane settled into hole	Trench collapse	-	Trench left open for extended period (DSD site)	ARUP files	N
34	27	DW	4	5m Fill 5m MD 22m CDG 27.5m HDG	1.5m	27	1993	ED	300mm settlement of adjacent carpark	Defect/hole in diaphragm wall	-	Soil/bentonite inclusion in the diaphragm wall	ARUP files	N

Table 4.1 - Summary of Wall Types and Failures for Private and Government Projects in Hong Kong

Case History Number	Private Job	Excessive Displacement	Collapse			Year of Event	Depth at Time of Failure (m)	Probable Causes of Failure
			Small	Medium	Large			
1	Y	CW				1989	0	Drawdown
2	Y			ShP		1990	4	Struts omitted
3	Y			ShP		1990	7.5	Inadequate penetration of sheetpile
4	Y	DW				1990	?	Groundwater drawdown in pumping test
5	Y				ShP	1981	10	Inadequate penetration of sheetpile & strut omitted
6	Y	ShP				1987	10	Cofferdam to remove obstructions
7	Y	DW				1985	0	Installation movement
8	Y	ShP				1988	8.1	Poor workmanship of strutting
9	Y	ShP				1988	8.8	Poor workmanship of strutting
10	Y	ShP				1988	9	Unexpected ground condition
11	Y	ShP				1987	7	Poor workmanship of strutting
12	Y		ShP			1987	8.5	Inadequate penetration of sheetpile
13	Y		ShP			1980	6.2	Inadequate penetration of sheetpile
14	Y	DW				1990	0	Dewatering for caisson excavation
17	Y	ShP				1990	0	Dewatering for caisson excavation
20	Y	PS				1982	(6)	Recompaction, surcharge
21	Y		ShP			1981	6	Inadequate penetration of sheetpile, poor workmanship of strutting
22	Y			ShP		1982	8.5	Inadequate penetration of sheet pile

Table 4.1 - Summary of Wall Types and Failures for Private and Government Projects in Hong Kong (Cont.)

Case History Number	Private Job	Excessive Displacement	Collapse			Year of Event	Depth at Time of Failure (m)	Probable Causes of Failure
			Small	Medium	Large			
23	Y		ShP*			1990	4.5	Strut omitted
24	Y		ShP			1989	7	Strut omitted
27	Y			PS		1991	14	Poor workmanship of lagging wall construction, poor workmanship of grout curtain
28	Y		ShP			1980	Slope	Poor workmanship of strutting
29	Y	ShP*				1991	4	Struts omitted
30	Y	ShP*				1991	4	Poor workmanship of strutting
31	Y	ShP*				1991	4	Poor workmanship of strutting
32	Y	ShP*				1993	7	Struts omitted
34	Y	DW				1993	27	Poor workmanship of wall construction
15	*N*		*ShP*			*1987*	*Slope*	
16	*N*			*ShP*		*1981*	*12.5*	*Poor workmanship of strutting + high water table*
18	*N*	*Open*				*1990*	*5*	*Base heave*
19	*N*		*Open*			*1987*	*6.5*	*No design*
25	*N*	*PS*				*1989*	*9*	*Poor workmanship of lagging wall*
26	*N*	*ShP*				*1989*	*9*	*Poor workmanship of strutting*
33	*N*			*DW*		*1995*	*0*	*Panel trench collapse*

ShP = Sheet pile wall ShP* = channel planking CW = Caisson Wall DW = Diaphragm Wall PS = Pipe pile/Soldier pile wall

Note : Figures in italics refer to a government site

Table 4.2 - Causes of Hazards and Mitigating Measures

Causes	Examples	Case History Number				Mitigating Measures
		Excessive Displacement	Collapse			
			Small	Medium	Large	
Poor workmanship	Struts omitted or poorly detailed.	6 8 9 11 *26* 29 30 31 32	21 23 24 28	2 *16*	5	Random site checks by authorities and better site control.
	Inadequate penetration of sheet piles.		12 13 21	3 22	5	Effective full time site control and pre marking lengths on sheet piles. Identification of sites where penetration is likely to be a problem.
	Inclusions or trench collapses in construction of diaphragm walls	7 34		*33*		Careful monitoring of diaphragm wall construction to improve likelihood of defect being detected.
	Excessive surcharging outside excavation.	20				Improve site control and planning.

Table 4.2 - Causes of Hazards and Mitigating Measures (Cont.)

Causes	Examples	Case History Number				Mitigating Measures
		Excessive Displacement	Collapse			
			Small	Medium	Large	
Poor workmanship	Excessive groundwater draw down.	1 4 14 17				Good predictions and monitoring. In the case of pumping tests some form of strutting may be required to limit movements during the test (this could also be considered to be a design error)
	Recharge well poorly designed.	10				Good predictions and monitoring.
	Lagging for soldier piles inadequate.	*25*		27		Good site control to ensure adequate backfilling and correct size excavations for lagging.
Unexpected ground conditions	Small areas of severe mud waving etc.	10				Additional site investigation where such effects are possible. Careful observation of wall installation and subsequent excavation.
Ground water variation	Extreme rainfall or water pipe bursting leading to excessive water head outside excavation.		24	*16*		Monitoring to detect increasing water head. Installation of drains to prevent build up of pressure.
Inadequate planning	Significant displacement expected but not planned for.	4 8 10 11				Ensure measures are in place to deal with these by • repairs as they occur • additional support to services • warning to affected parties.

Note : Figures in italics refer to a government site

Table 5.1 - List of Private Projects Referred to GEO

Year	Sheet Piling	Diaphragm Wall	Caisson Wall	Pipe/Soldier Piling	Total Number
1981	*28*	*0*	*42*	*12*	*82*
1982	*12*	*0*	*18*	*14*	*44*
1983	*16*	*2*	*20*	*6*	*44*
1984	*18*	*6*	*24*	*8*	*56*
1985	24	7	28	15	74
1986	35	4	36	26	101
1987	44	10	30	29	113
1988	55	13	39	36	143
1989	58	15	46	38	157
1990	46	9	33	23	111
1991	51	5	32	37	125
1992	61	17	59	57	194
1993	74	19	51	70	214
1994	65	19	44	65	193
1995	52	10	28	46	136
Total	639	136	530	482	1787

Note : Figures in italics have been doubled in order to be approximately consistent with later years.

Table 5.2 - Summary of Number of Incidents for Private Projects between 1981 and 1995 Related to Wall Type

Wall type	Excessive Displacement	Collapse			Collapse Total
		Small	Medium	Large	
Caisson wall	1	0	0	0	0
Sheet pile	10	4	3	1	8
Diaphragm wall	4	0	0	0	0
Soldier pile / Pipe pile	1	0	1	0	1
Total for all walls	16	4	4	1	9

Table 5.3 - Observed Probability of Failure

	Sheet Piling	Caisson Wall	Diaphragm Wall	Soldier Piling	Total
Total number of excavations	639	530	136	482	1787
Excessive displacement					
Number of incidents	10	1	4	1	16
Probability	0.016	0.002	0.029	0.002	0.009
Collapse					
Number of small collapses	4	0	0	0	4
Number of medium collapses	3	0	0	1	4
Number of large collapses	1	0	0	0	1
Total number of collapses	8	0	0	1	9
Probability of collapse	0.013	0	0	0.002	0.005

Table 5.4 - Observed Probability of Failure with Time

	1981 - 1985	1986 - 1990	1991 - 1995	Total
Total number of excavations	300	625	862	1787
Number of excessive displacements	2	9	5	16
Number of collapses	3	5	1	9
Total number of failures	5	14	6	25
Probability of failure	0.017	0.022	0.007	0.014

Table 5.5 - Results from Incident Data for Collapse

Depth of Excavation at the Time of Failure	Observed Number of Collapses			
	Small	Medium	Large	Total
<10m	4 [65%]	3 [30%]	- [5%]	7
>10m	- [20%]	1 [50%]	1 [30%]	2
Overall	4	4	1	9

Table 5.6 - Results of GEO Survey for Depth Ranges

Type of Wall	Excavation Depth Range (%)				Total Number
	<5m	5 to 10m	10 to 15m	>15m	
Sheet pile	37	46	17	0	244
Soldier/pipe pile	44	38	16	2	148
Diaphragm wall	0	13	46	41	52
Large bored pile	0	50	35	15	26
Average	33	40	21	6	468

Table 5.7 - Apportionment of Frequency of Excessive Displacement

Wall Type	Annual Number of Excavations	Probability of Excessive Displacement	Annual Frequency
Sheet pile	52	0.015	0.78
Diaphragm wall	11	0.027	0.30
Large bored pile wall	6	0.003	0.02
Soldier pile / pipe pile	31	0.003	0.10
Total	100		1.20

Table 5.8 - Apportionment of Collapse Frequency

Wall Type	Depth Range	Annual Number of Excavations	Probability of Collapse	Annual Frequency
Sheet pile	<10m	43	0.012	0.516
	>10m	9	0.012	0.108
Diaphragm wall	<10m	1	0.003	0.003
	>10m	10	0.003	0.030
Large bored pile	<10m	3	0.003	0.009
	>10m	3	0.003	0.009
Soldier pile/ pipe pile	<10m	25	0.003	0.075
	>10m	6	0.003	0.018
Total	<10m	72	0.008	0.603
	>10m	28	0.006	0.165
	Overall	100	0.008	0.768

Table 6.1 - Likelihood of Number of Fatalities for Building Damage

No. of Fatalities (range)	1	2 to 3	4 to 10	11 to 30	31 to 90	Total Number
No. of Fatalities (average)	1	2	6	18	60	
Outcome Scenario	Probabilities Corresponding to Fatality Range					
Partial collapse of tower block	0.2	0.3	0.2	0.05	0.01	3.5
Partial collapse of medium rise	0.3	0.2	0.1	0.01	0	1.4
Total collapse of medium rise	0	0	0.5	0.3	0.2	20.4
Partial collapse of low rise	0.2	0.1	0.01	0	0	0.5
Total collapse of low rise	0.25	0.3	0.4	0.05	0	4.2

Table 6.2 - Likelihood of Fatalities from a Vehicle Fall

Vehicle Type	Proportion on Road	Number of Fatalities				
		1	2	6	18	Overall
Car	0.8	0.05	0.01	0	0	0.07
Minibus	0.15	0.05	0.05	0.01	0	0.21
Bus	0.05	0.01	0.05	0.05	0.01	0.59
Overall		0.048	0.018	0.004	0.0005	0.117

Table 6.3 - Likelihood of Fatalities for Pedestrian Fall

Size of Collapse	Number of Fatalities					Overall
	1	2	6	18	60	
Large	0.1	0.01	0	0	0	0.12
Medium	0.03	0	0	0	0	0.03
Small	0.01	0	0	0	0	0.01

Table 6.4 - Likelihood of Fatalities for a Gas Release Scenario

Time of Day	Number of Fatalities					Overall
	1	2	6	18	60	
Day	0.1	0.05	0.01	0	0	0.26
Night	0.01	0.01	0	0	0	0.03

Table 6.5 - Likelihood of Fatalities for Workers

Size of Collapse	Depth	Number of Fatalities					Overall
		1	2	6	18	60	
Large	> 10m < 10m	0.6 0.3	0.2 0.1	0.05 0.01	0 0	0 0	1.3 0.56
Medium	> 10m < 10m	0.3 0.1	0.05 0.01	0 0	0 0	0 0	0.4 0.12
Small	> 10m < 10m	0.05 0.01	0 0	0 0	0 0	0 0	0.05 0.01

Table 7.1 - Overall PLL

Group at Risk	PLL per Year	% of Total PLL
Public Workers	0.021 0.010	68% 32%
Total	0.030	100%

Table 7.2 - Breakdown of Overall PLL by Type of Failure

Type of Failure	PLL	
	Workers & Public	Public Only
Excessive displacement	0.0001 (<1%)	0.0001 (<1%)
Small collapse Medium collapse Large collapse	0.0008 (3%) 0.0099 (33%) 0.0196 (64%)	0.0005 (3%) 0.0068 (33%) 0.0133 (64%)

Table 7.3 - Breakdown of Overall PLL by Type of Wall

Type of Wall	PLL	
	Workers & Public	Public Only
Sheet pile wall	0.0230 (76%)	0.0157 (76%)
Diaphragm wall	0.0028 (9%)	0.0019 (9%)
Large bored pile wall	0.0009 (3%)	0.0006 (3%)
Soldier/pipe pile wall	0.0036 (12%)	0.0025 (12%)

Table 7.4 - Breakdown of Overall PLL by Depth of Excavation

Type of Wall	PLL	
	Workers & Public	Public Only
Less than 10m	0.0150 (49%)	0.0103 (50%)
10m or more	0.0154 (51%)	0.0104 (50%)

Table 7.5 - Breakdown of Overall PLL to the Public by Type of Facility at Risk

Type of Facility	PLL (Public Only)
Tower block	0.0000 (0%)
Medium rise building with pile foundation	0.0002 (1%)
Medium rise building with pad foundation	0.0023 (11%)
Low rise building	0.0126 (61%)
Footpath	0.0036 (17%)
Road	0.0019 (9%)
Gas Pipe	0.0003 (1%)

Table 7.6 - PLL Values (for the Public) for Individual Excavations as a Function of Wall Type

Type of Wall	Depth of Excavation	
	< 10m	> 10m
Sheet pile wall	2.06×10^{-4}	7.69×10^{-4}
Diaphragm wall	0.54×10^{-4}	1.94×10^{-4}
Large bored pile wall	0.51×10^{-4}	1.92×10^{-4}
Soldier/pipe pile wall	0.51×10^{-4}	1.92×10^{-4}

Table 7.7 - Range of Public PLL Values (for the Public) for Individual Excavations in Specific Conditions

Wall Type and Adjoining Building Type	Depth of Excavation	
	< 10m	> 10m
Sheet pile wall adjoining a low rise building	2.27×10^{-4}	9.22×10^{-4}
Sheet pile wall adjoining a medium rise building on pads	4.07×10^{-4}	15.3×10^{-4}
Diaphragm wall adjoining a tower block	0.15×10^{-4}	0.48×10^{-4}

Note : In all cases half of the excavation is assumed to be bounded by a road Type B.

Table 7.8 - Upper and Lower Bound Values Assigned to the Parameters of Interest in the Event Trees

Parameters to be Varied	Range or Value of			Reference
	Lower Bound	Most Likely	Upper Bound	
Number of excavations per year	40-80	80-120	120-160	Section 5.5
P(Sheet pile)	0.35-0.45	0.45-0.55	0.55-0.6	Fig 5.3
P(Diaphragm wall)	0.08-0.1	0.1-0.12	0.12-0.15	Fig 5.3
P(Large bored pile)	0.02-0.04	0.04-0.06	0.06-0.08	Fig 5.3
P(Depth<10m | Sheet pile)	0.65-0.75	0.75-0.85	0.85-0.95	Fig 5.3
P(Depth<10m | Diaphragm wall)	0.15-0.2	0.2-0.3	0.3-0.35	Fig 5.3
P(Depth<10m | Large bored pile)	0.2-0.4	0.4-0.6	0.6-0.8	Fig 5.3
P(Depth<10m | Soldier pile)	0.65-0.75	0.75-0.85	0.85-0.9	Fig 5.3
P(Collapse | Sheet pile wall)	0.005-0.01	0.01-0.015	0.015-0.02	Fig 5.3
P(Collapse | Diaphragm wall)	0.0005-0.001	0.001-0.005	0.005-0.01	Fig 5.3
P(Collapse | Large bored pile)	0.0005-0.001	0.001-0.005	0.005-0.01	Fig 5.3
P(Collapse | Soldier pile)	0.0005-0.001	0.001-0.005	0.005-0.01	Fig 5.3
P(Small collapse | Depth<10m)	0.55-0.6	0.6-0.7	0.7-0.75	Fig 5.3
P(Large collapse | Depth<10m)	0.01-0.03	0.03-0.06	0.06-0.09	Fig 5.3
P(Small collapse | Depth>10m)	0.1-0.15	0.15-0.25	0.25-0.3	Fig 5.3
P(Large collapse | Depth>10m)	0.15-0.25	0.25-0.35	0.35-0.45	Fig 5.3
P(Tower Block)	0.15-0.2	0.2-0.3	0.3-0.35	Fig 6.1-6.3
P(Medium rise building, pad)	0.05-0.1	0.1-0.15	0.15-0.2	Fig 6.1-6.3
P(Low rise building)	0.25-0.3	0.3-0.4	0.4-0.45	Fig 6.1-6.3
P(Tower block partial damage | SC)	0	0	0	Fig 6.1
P(Tower block nonstructure damage | SC)	0	0.001	0.005	Fig 6.1
P(Medium rise building, pile partial damage | SC)	0	0	0	Fig 6.1
P(Medium rise building, pile nonstructure damage | SC)	0.001	0.005	0.01	Fig 6.1
P(Medium rise building, pad total damage | SC)	0	0	0	Fig 6.1
P(Medium rise building, pad partial damage | SC)	0	0.001	0.01	Fig 6.1
P(Medium rise building, pad nonstructure damage | SC)	0.001	0.01	0.05	Fig 6.1
P(Low rise building total damage | SC)	0	0	0.001	Fig 6.1
P(Low rise building partial damage | SC)	0.002	0.01	0.05	Fig 6.1
P(Low rise building nonstructure damage | SC)	0.01	0.05	0.1	Fig 6.1
P(Tower block partial damage | MC)	0	0	0	Fig 6.2
P(Tower block nonstructure damage | MC)	0.01	0.03	0.05	Fig 6.2
P(Medium rise building, pile partial damage | MC)	0	0	0	Fig 6.2

Note: SC implies small collapse, MC implies medium collapse and LC implies large collapse

Table 7.8 - Upper and Lower Bound Values Assigned to the Parameters of Interest in the Event Trees (Cont.)

Parameters to be Varied	Range or Value of			Reference
	Lower Bound	Most Likely	Upper Bound	
P(Medium rise building, pile nonstructure damage | MC)	0.005	0.02	0.035	Fig 6.2
P(Medium rise building, pad total damage | MC)	0	0.001	0.01	Fig 6.2
P(Medium rise building, pad partial damage | MC)	0.001	0.01	0.02	Fig 6.2
P(Medium rise building, pad nonstructure damage | MC)	0.05	0.1	0.15	Fig 6.2
P(Low rise building total damage | MC)	0.005	0.01	0.02	Fig 6.2
P(Low rise building partial damage | MC)	0.05	0.1	0.15	Fig 6.2
P(Low rise building nonstructure damage | MC)	0.2	0.5	0.7	Fig 6.2
P(Tower block partial damage | LC)	0	0	0.001	Fig 6.3
P(Tower block nonstructure damage | LC)	0.001	0.005	0.01	Fig 6.3
P(Medium rise building, pile partial damage | LC)	0.005	0.01	0.02	Fig 6.3
P(Medium rise building, pile nonstructure damage | LC)	0.01	0.05	0.1	Fig 6.3
P(Medium rise building, pad total damage | LC)	0.005	0.01	0.02	Fig 6.3
P(Medium rise building, pad partial damage | LC)	0.05	0.1	0.15	Fig 6.3
P(Medium rise building, pad nonstructure damage | LC)	0.1	0.3	0.5	Fig 6.3
P(Low rise building total damage | LC)	0.05	0.1	0.15	Fig 6.3
P(Low rise building partial damage | LC)	0.2	0.4	0.5	Fig 6.3
P(Low rise building nonstructure damage | LC)	0.2	0.5	0.7	Fig 6.3
P(Pedestrian fall | LC, day)	0.01	1	1	Fig 6.4
P(Pedestrian fall | LC, night)	0	0.05	0.1	Fig 6.4
P(Pedestrian fall | MC, day)	0.1	0.5	1	Fig 6.4
P(Pedestrian fall | MC, night)	0	0	0.001	Fig 6.4
P(Pedestrian fall | SC, day)	0.05	0.1	0.15	Fig 6.4
P(Pedestrian fall | SC, night)	0	0	0.001	Fig 6.4
P(Type A road)	0.05	0.1	0.2	Fig 6.5
P(Type B road)	0.4	0.3	0.6	Fig 6.5
P(Vehicle fall | LC, type A road)	0.01	0.02	0.05	Fig 6.5
P(Vehicle fall | LC, type B road)	0.02	0.1	0.2	Fig 6.5
P(Vehicle fall | LC, type C road)	0.1	0.2	0.3	Fig 6.5
P(Vehicle fall | MC, type A road)	0.005	0.01	0.02	Fig 6.5
P(Vehicle fall | MC, type B road)	0.01	0.05	0.1	Fig 6.5
P(Vehicle fall | MC, type C road)	0.02	0.1	0.2	Fig 6.5
P(Car | Type A road)	0.7	0.8	0.9	Fig 6.5

Note: SC implies small collapse, MC implies medium collapse and LC implies large collapse

Table 7.8 - Upper and Lower Bound Values Assigned to the Parameters of Interest in the Event Trees (Cont.)

Parameters to be Varied		Range or Value of Lower Bound	Range or Value of Most Likely	Range or Value of Upper Bound	Reference
P(Bus | Type A road)		0.01	0.05	0.1	Fig 6.5
P(Car | Type B road)		0.7	0.8	0.9	Fig 6.5
P(Bus | Type B road)		0.01	0.05	0.1	Fig 6.5
P(Car | Type C road)		0.7	0.8	0.9	Fig 6.5
P(Bus | Type C road)		0.01	0.05	0.1	Fig 6.5
Probability given number of fatalities in partial collapse of tower block	1	0.1	0.2	0.3	Table 6.1
	2	0.1	0.3	0.3	
	6	0.05	0.2	0.2	
	18	0.005	0.05	0.1	
	60	0	0.01	0.05	
Probability given number of fatalities in partial collapse of medium rise building	1	0.2	0.3	0.2	Table 6.1
	2	0.1	0.2	0.3	
	6	0.05	0.1	0.2	
	18	0	0.01	0.1	
	60	0	0	0.01	
Probability given number of fatalities in total collapse of medium rise building	1	0.1	0	0	Table 6.1
	2	0.3	0	0	
	6	0.3	0.5	0.3	
	18	0.2	0.3	0.4	
	60	0.1	0.2	0.2	
Probability given number of fatalities in partial collapse of low rise building	1	0.1	0.2	0.3	Table 6.1
	2	0.05	0.1	0.2	
	6	0	0.01	0.1	
	18	0	0	0.01	
	60	0	0	0	
Probability given number of fatalities in total collapse of low rise building	1	0.2	0.25	0	Table 6.1
	2	0.2	0.3	0.39	
	6	0.1	0.4	0.5	
	18	0	0.05	0.1	
	60	0	0	0.01	
Probability given number of fatalities in car fall	1	0.05	0.05	0.1	Table 6.2
	2	0.005	0.01	0.05	

Note: SC implies small collapse, MC implies medium collapse and LC implies large collapse

Table 7.8 - Upper and Lower Bound Values Assigned to the Parameters of Interest in the Event Trees (Cont.)

Parameters to be Varied		Range or Value of			Reference
		Lower Bound	Most Likely	Upper Bound	
Probability given number of fatalities in mini-bus fall	1	0.05	0.05	0.1	Table 6.2
	2	0.01	0.05	0.1	
	6	0.005	0.01	0.05	
Probability given number of fatalities in bus fall	1	0.01	0.01	0.1	Table 6.2
	2	0.05	0.05	0.1	
	6	0.01	0.05	0.05	
	18	0.005	0.01	0.01	
Probability given number of fatalities in pedestrian fall due to large collapse	1	0.1	0.1	0.1	Table 6.3
	2	0	0.01	0.01	
Probability given number of fatalities in pedestrian fall due to medium collapse	1	0.005	0.03	0.05	Table 6.3
	2	0	0	0.01	
Probability given number of fatalities in pedestrian fall due to small collapse	1	0	0.01	0.05	Table 6.3
	2	0	0	0	

Note: SC implies small collapse, MC implies medium collapse and LC implies large collapse

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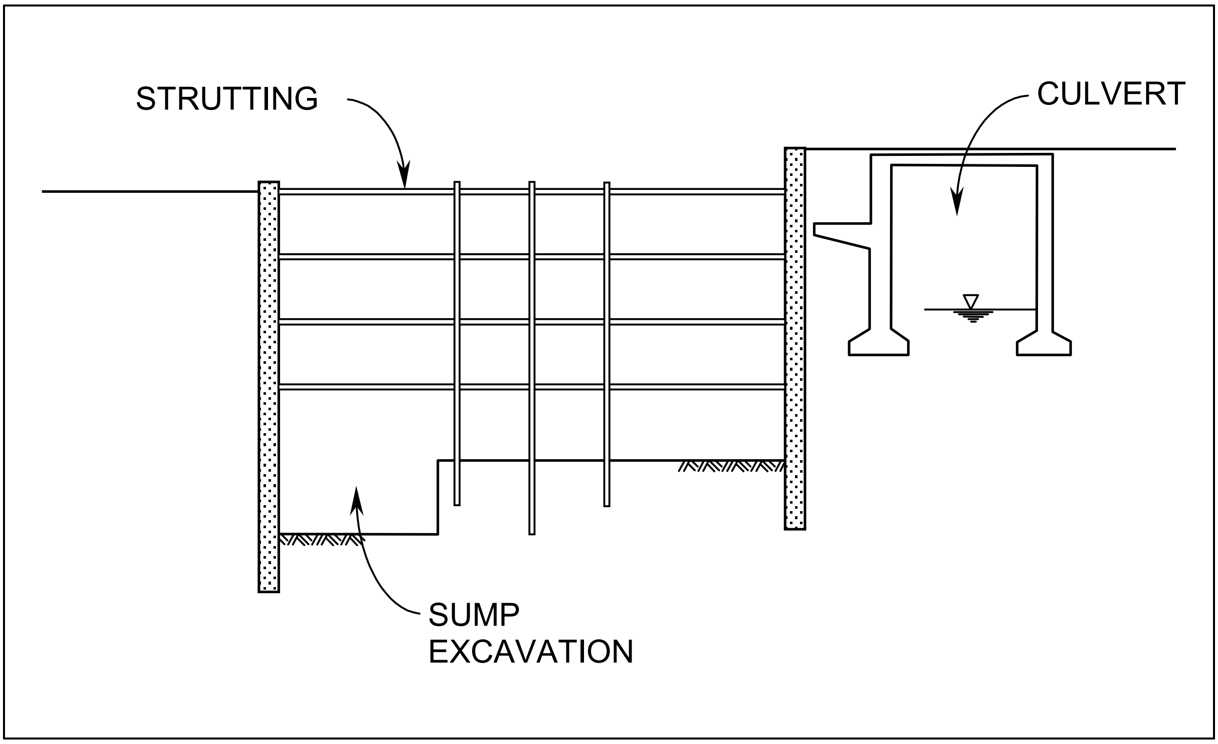

Figure 3.1 - Section Through Diaphragm Wall in Seoul, South Korea 1989

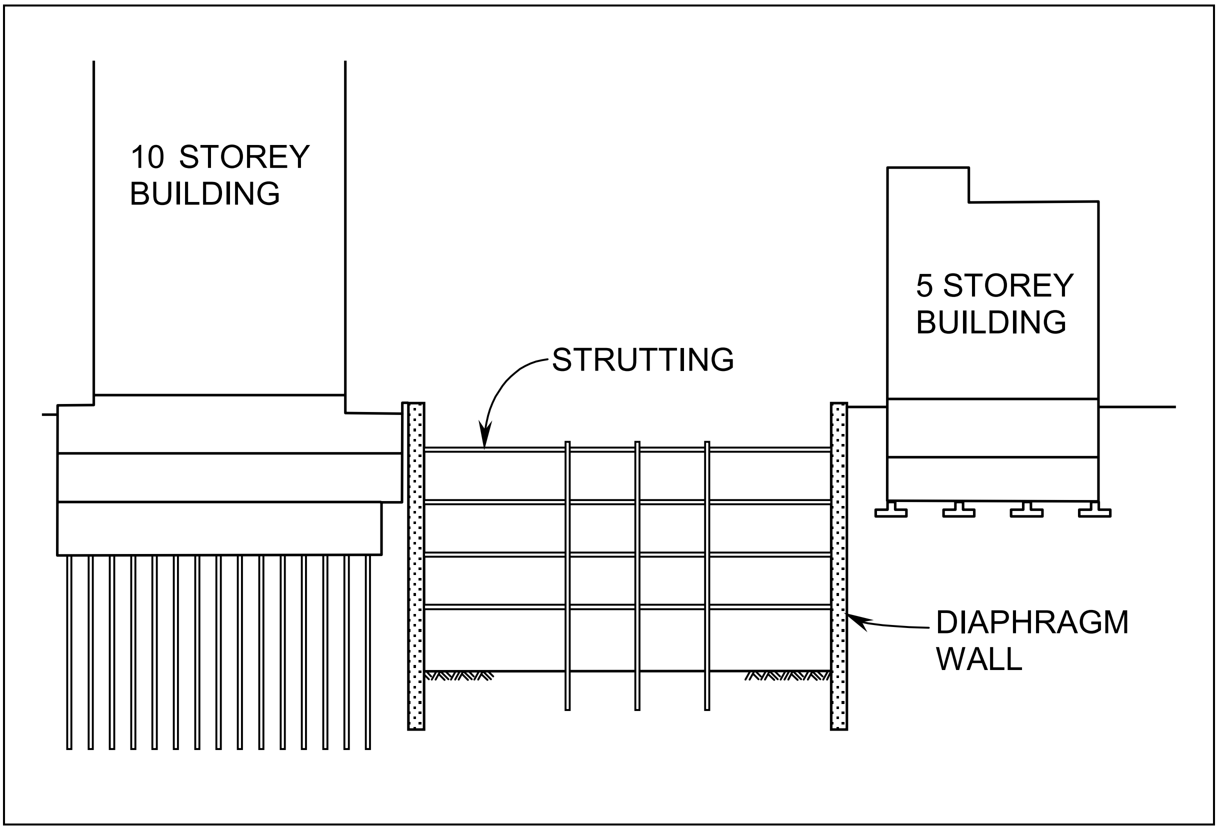

Figure 3.2 - Section Through Diaphragm Wall in Seoul, South Korea 1989

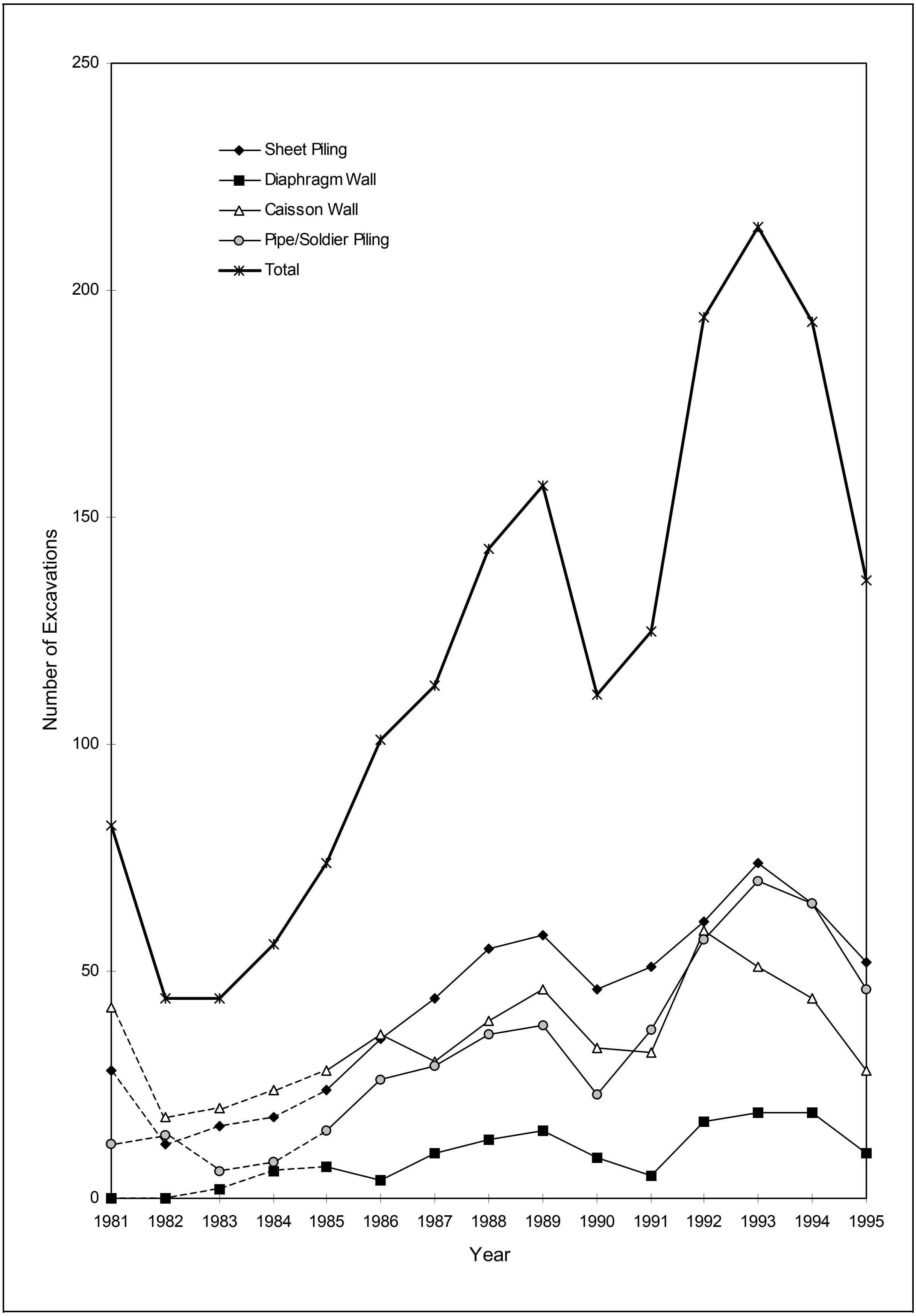

Figure 5.1 - Total Numbers of Excavation Referred to GEO

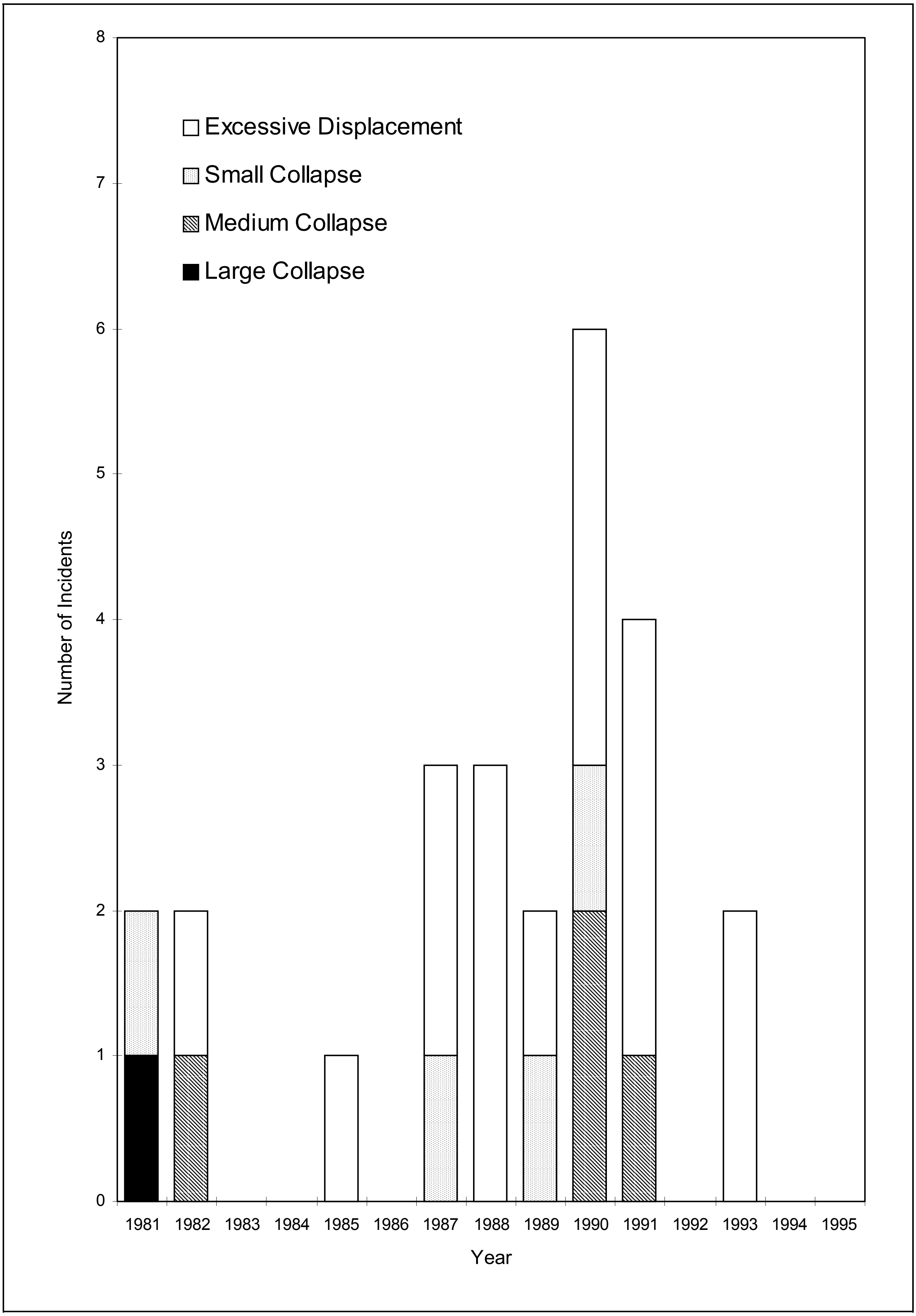

Figure 5.2 - Summary of Wall Types Failures for Private Projects in Hong Kong

(SH: sheet pile; DW: diaphragm wall; PS: pipe/soldier pile; BP: large bored pipe wall)

	Wall Type? [1]		Depth >=10m? [2]		Incident? [3]		Size of Collapse [4]		Outcome	Outcome Probability
1	SH	0.52	Y	0.17	ED	0.015			ED(SH>10m)	0.00132600
			N		CL	0.012	SC	0.2	SC(SH>10m)	0.00021216
							MC	0.5	MC(SH>10m)	0.00053040
							LC	0.3	LC(SH>10m)	0.00031824
					N	0.973			N(SH>10m)	0.08601320
				0.83	ED	0.015			ED(SH<10m)	0.00647400
					CL	0.012	SC	0.65	SC(SH<10m)	0.00336648
							MC	0.3	MC(SH<10m)	0.00155376
							LC	0.05	LC(SH<10m)	0.00025896
					N	0.973			N(SH<10m)	0.41994680
	DW	0.11	Y	0.87	ED	0.027			ED(DW>10m)	0.00258390
			N		CL	0.003	SC	0.2	SC(DW>10m)	0.00005742
							MC	0.5	MC(DW>10m)	0.00014355
							LC	0.3	LC(DW>10m)	0.00008613
					N	0.97			N(DW>10m)	0.09282900
				0.13	ED	0.027			ED(DW<10m)	0.00038610
					CL	0.003	SC	0.65	SC(DW<10m)	0.00002789
							MC	0.3	MC(DW<10m)	0.00001287
							LC	0.05	LC(DW<10m)	0.00000215
					N	0.97			N(DW<10m)	0.01387100
	PS	0.32	Y	0.18	ED	0.003			ED(PS>10m)	0.00017280
			N		CL	0.003	SC	0.2	SC(PS>10m)	0.00003456
							MC	0.5	MC(PS>10m)	0.00008640
							LC	0.3	LC(PS>10m)	0.00005184
					N	0.994			N(PS>10m)	0.05725440
				0.82	ED	0.003			ED(PS<10m)	0.00078720
					CL	0.003	SC	0.65	SC(PS<10m)	0.00051168
							MC	0.3	MC(PS<10m)	0.00023616
							LC	0.05	LC(PS<10m)	0.00003936
					N	0.994			N(PS<10m)	0.26082560
	BP	0.05	Y	0.5	ED	0.003			ED(BP>10m)	0.00007500
			N		CL	0.003	SC	0.2	SC(BP>10m)	0.00001500
							MC	0.5	MC(BP>10m)	0.00003750
							LC	0.3	LC(BP>10m)	0.00002250
					N	0.994			N(BP>10m)	0.02485000
				0.5	ED	0.003			ED(BP<10m)	0.00007500
					CL	0.003	SC	0.65	SC(BP<10m)	0.00004875
							MC	0.3	MC(BP<10m)	0.00002250
							LC	0.05	LC(BP<10m)	0.00000375
					N	0.994			N(BP<10m)	0.02485000
										1.00000000

Figure 5.3 - Event Tree for Deep Excavation Causing an Excessive Displacement or Collapse

Initiating event	Frequency	Y/N	Building type [1]	Damage type [2]	Outcome	Outcome probability
Small Collapse (SC)	0.00213697	Y / N	0.26 Tower block	0 Structure partial damage	Tower block structure partial damage	0.00000000
				0.001 Non-structure damage	Tower block non-structure damage	0.00000056
				0.999 None	No effect	0.00055506
			0.27 Medium rise building with pile foundation	0 Structure partial damage	Medium rise building with pile foundation structure partial damage	0.00000000
				0.005 Non-structure damage	Medium rise building with pile foundation non-structure damage	0.00000288
				0.995 None	No effect	0.00057410
			0.12 Medium rise building with pad foundation	0 Structure total damage	Medium rise building with pad foundation structure total damage	0.00000000
				0.001 Structure partial damage	Medium rise building with pad foundation structure partial damage	0.00000026
				0.01 Non-structure damage	Medium rise building with pad foundation non-structure damage	0.00000256
				0.989 None	No effect	0.00025362
			0.35 Low rise building	0 Structure total damage	Low rise building structure total damage	0.00000000
				0.01 Structure partial damage	Low rise building structure partial damage	0.00000748
				0.05 Non-structure damage	Low rise building non-structure damage	0.00003740
				0.94 None	No effect	0.00070306
						0.00213697

Figure 6.1 - Event Tree for Small Collapse Affecting Buildings

			Building type [1]	Damage type [2]	Outcome	Outcome probability
Medium Collapse (MC)	0.00131157	Y / N	0.26 Tower block	0 Structure partial damage	Tower block structure partial damage	0.00000000
				0.03 Non-structure damage	Tower block non-structure damage	0.00001023
				0.97 None	No effect	0.00033078
			0.27 Medium rise building with pile foundation	0 Structure partial damage	Medium rise building with pile foundation structure partial damage	0.00000000
				0.02 Non-structure damage	Medium rise building with pile foundation non-structure damage	0.00000708
				0.98 None	No effect	0.00034704
			0.12 Medium rise building with pad foundation	0.001 Structure total damage	Medium rise building with pad foundation structure total damage	0.00000016
				0.01 Structure partial damage	Medium rise building with pad foundation structure partial damage	0.00000157
				0.1 Non-structure damage	Medium rise building with pad foundation non-structure damage	0.00001574
				0.889 None	No effect	0.00013992
			0.35 Low rise building	0.01 Structure total damage	Low rise building structure total damage	0.00000459
				0.1 Structure partial damage	Low rise building structure partial damage	0.00004590
				0.5 Non-structure damage	Low rise building non-structure damage	0.00022952
				0.39 None	No effect	0.00017903
						0.00131157

Figure 6.2 - Event Tree for Medium Collapse Affecting Buildings

			Building type [1]	Damage type [2]	Outcome	Outcome probability
Large Collapse (LC)	0.00039146	Y / N	0.26 Tower block	0 Structure partial damage	Tower block structure partial damage	0.00000000
				0.005 Non-structure damage	Tower block non-structure damage	0.00000051
				0.995 None	No effect	0.00010127
			0.27 Medium rise building with pile foundation	0.01 Structure partial damage	Medium rise building with pile foundation structure partial damage	0.00000106
				0.05 Non-structure damage	Medium rise building with pile foundation non-structure damage	0.00000528
				0.94 None	No effect	0.00009935
			0.12 Medium rise building with pad foundation	0.01 Structure total damage	Medium rise building with pad foundation structure total damage	0.00000047
				0.1 Structure partial damage	Medium rise building with pad foundation structure partial damage	0.00000470
				0.3 Non-structure damage	Medium rise building with pad foundation non-structure damage	0.00001409
				0.59 None	No effect	0.00002772
			0.35 Low rise building	0.1 Structure total damage	Low rise building structure total damage	0.00001370
				0.4 Structure partial damage	Low rise building structure partial damage	0.00005480
				0.5 Non-structure damage	Low rise building non-structure damage	0.00006851
				0 None	No effect	0.00000000
						0.00039146

Figure 6.3 - Event Tree for Large Collapse Affecting Buildings

Initiating event	Frequency	Day? [1]	Pedestrian fall? [2]	Outcome	Outcome probability
Large Collapse (LC)	0.00039146	Y 0.5	Y 1	Pedestrian fall	0.00019573
			N 0	No effect	0.00000000
		N 0.5 Night	Y 0.05	Pedestrian fall	0.00000979
			N 0.95	No effect	0.00018594
					0.00039146

Initiating event	Frequency	Day?	Pedestrian fall?	Outcome	Outcome probability
Medium Collapse (MC)	0.00131157	Y 0.5	Y 0.5	Pedestrian fall	0.00032789
			N 0.5	No effect	0.00032789
		N 0.5 Night	Y 0	Pedestrian fall	0.00000000
			N 1	No effect	0.00065579
					0.00131157

Initiating event	Frequency	Day?	Pedestrian fall?	Outcome	Outcome probability
Small Collapse (SC)	0.00213697	Y 0.5	Y 0.1	Pedestrian fall	0.00010685
			N 0.9	No effect	0.00096164
		N 0.5 Night	Y 0	Pedestrian fall	0.00000000
			N 1	No effect	0.00106848
					0.00213697

Figure 6.4 - Event Tree for Collapse of Excavation Adjoining Road Affecting Footpath

		Type of road [1]	Vehicle fall? [2]	Vehicle type? [3]	Outcome	Outcome probability
Large Collapse (LC)	0.00039146	A 0.1	Y 0.02	Car 0.8	Car fall	0.00000063
				MB 0.15	Minibus fall	0.00000012
				Bus 0.05	Bus fall	0.00000004
			N 0.98		No effect	0.00003836
		B 0.3	Y 0.1	Car 0.8	Car fall	0.00000940
				MB 0.15	Minibus fall	0.00000176
				Bus 0.05	Bus fall	0.00000059
			N 0.9		No effect	0.00010569
		C 0.6	Y 0.2	Car 0.8	Car fall	0.00003758
				MB 0.15	Minibus fall	0.00000705
				Bus 0.05	Bus fall	0.00000235
			N 0.8		No effect	0.00018790
						0.00039146

		Type of road	Vehicle fall?	Vehicle type?	Outcome	Outcome probability
Medium Collapse (MC)	0.00131157	A 0.1	Y 0.01	Car 0.8	Car fall	0.00000105
				MB 0.15	Minibus fall	0.00000020
				Bus 0.05	Bus fall	0.00000007
			N 0.99		No effect	0.00012985
		B 0.3	Y 0.05	Car 0.8	Car fall	0.00001574
				MB 0.15	Minibus fall	0.00000295
				Bus 0.05	Bus fall	0.00000098
			N 0.95		No effect	0.00037380
		C 0.6	Y 0.1	Car 0.8	Car fall	0.00006296
				MB 0.15	Minibus fall	0.00001180
				Bus 0.05	Bus fall	0.00000393
			N 0.9		No effect	0.00070825
						0.00131157

Small collapse does not affect roads.

Figure 6.5 - Event Tree for Collapse of Excavation Affecting Road

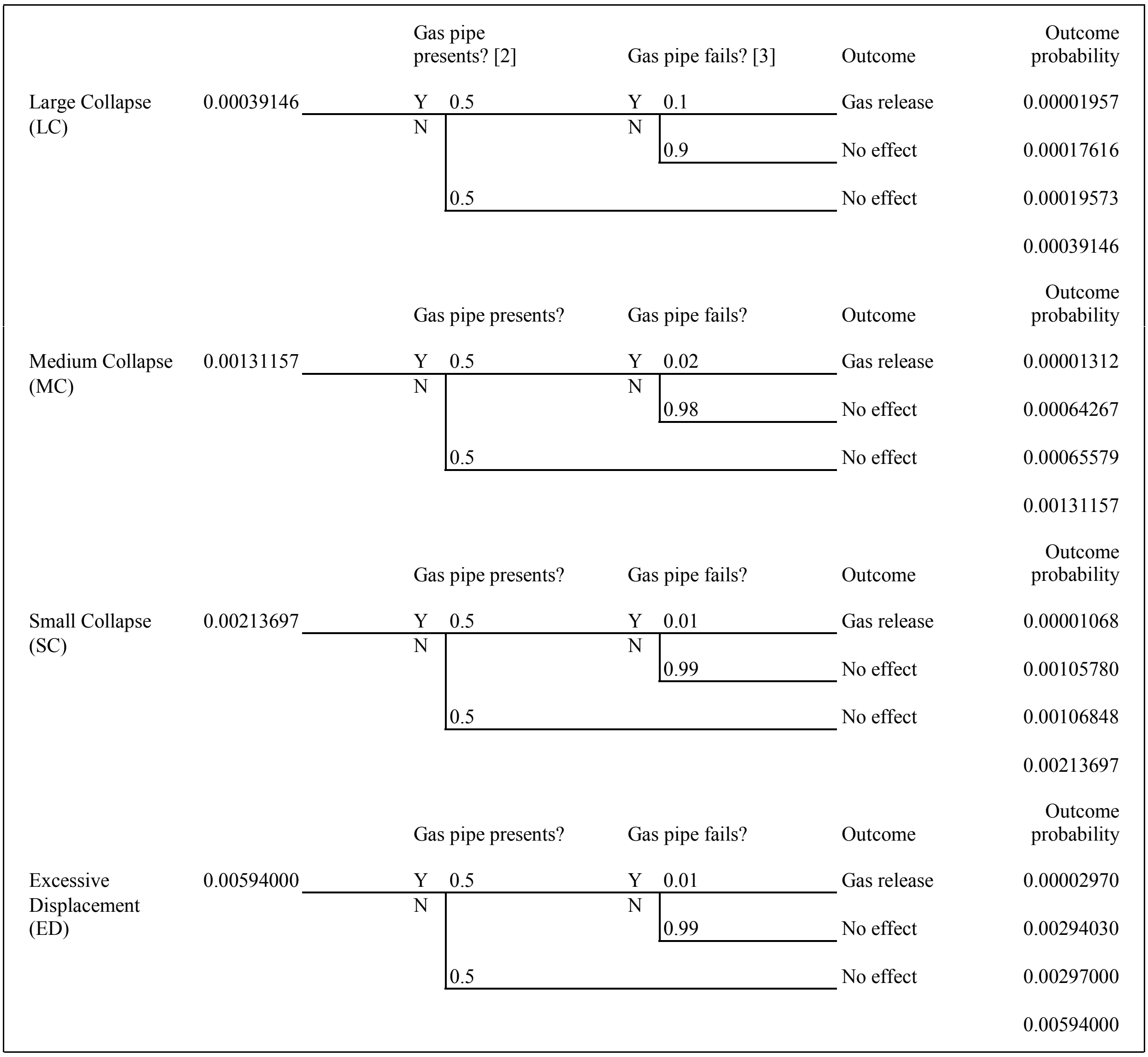

Figure 6.6 - Event Tree for Collapse of Excavation and Excessive Displacement Adjoining Road Affecting Gas Pipe

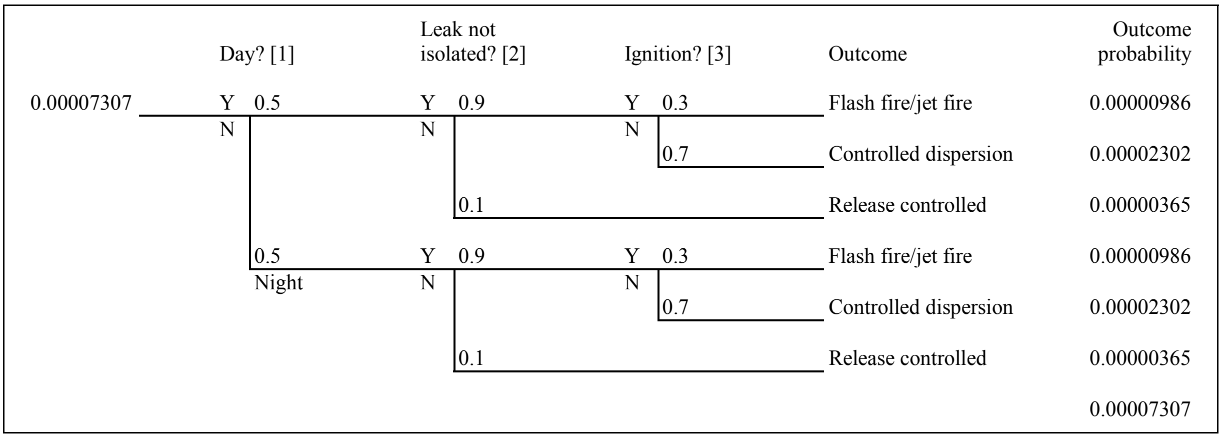

Figure 6.7 - Event Tree for Gas Pipe Failure Due to Excessive Displacement or Collapse

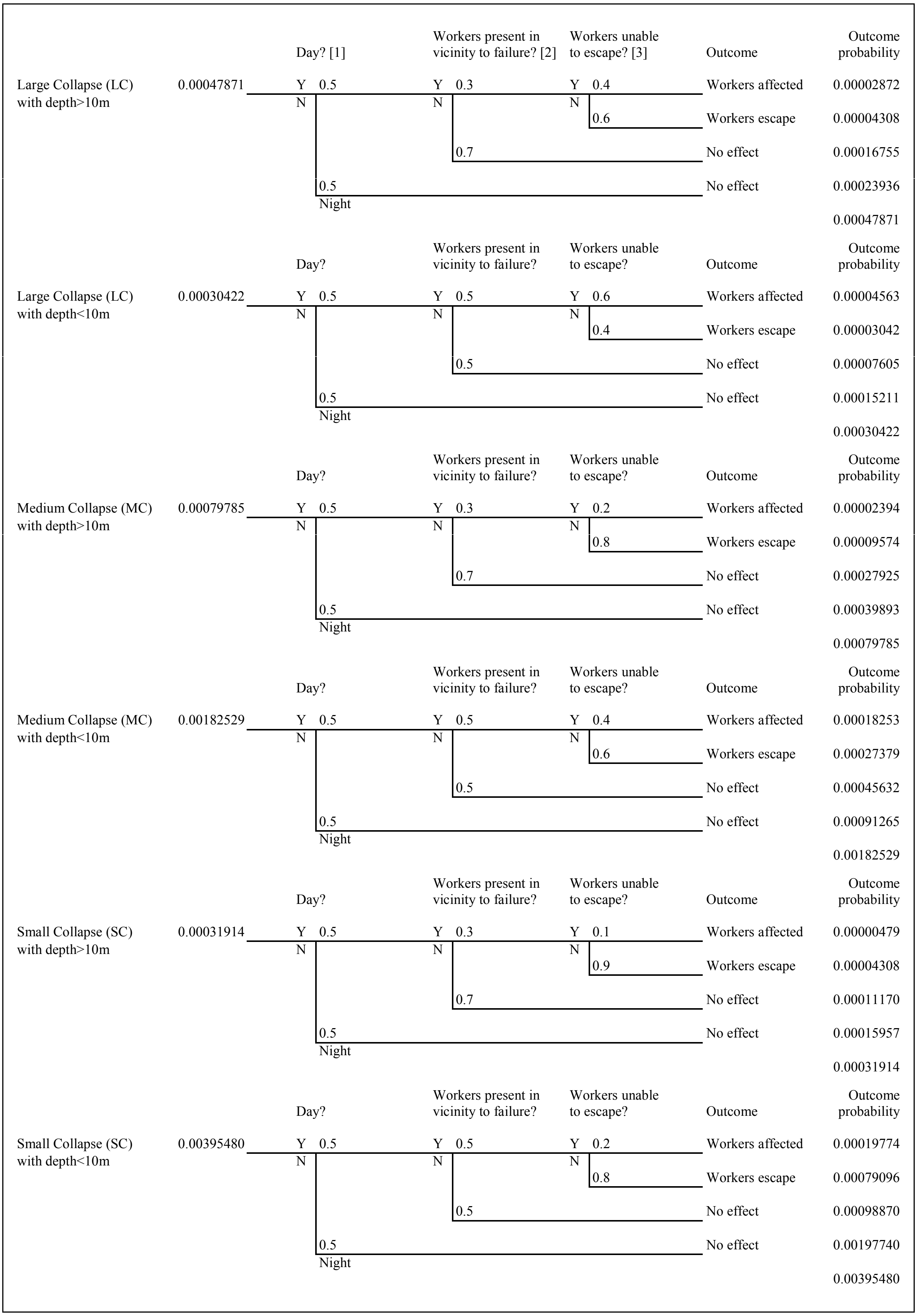

Figure 6.8 - Event Tree for Collapse of Excavation Affecting Workers

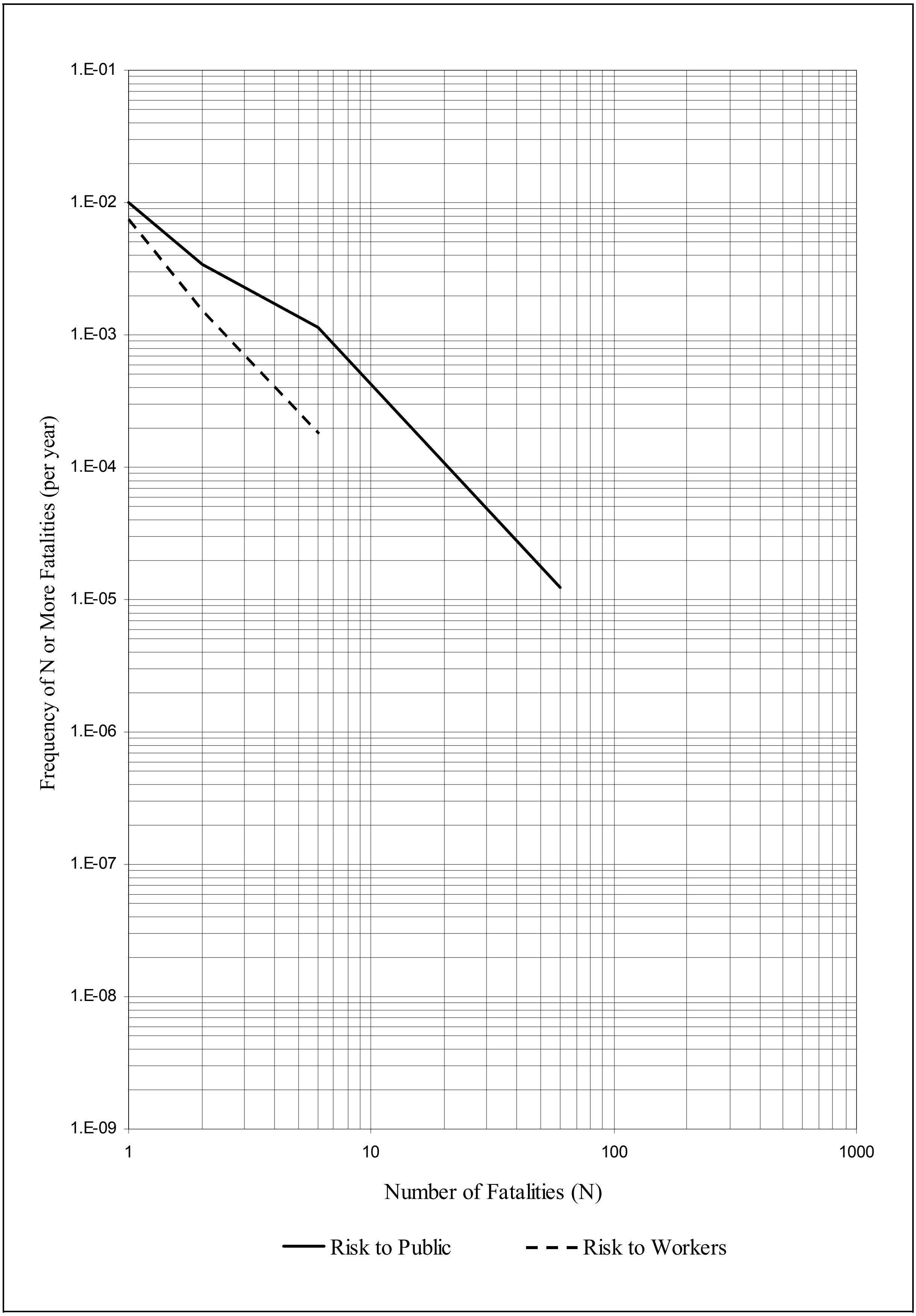

Figure 7.1 - Calculated Overall F-N Curves for Future Excavations in Hong Kong

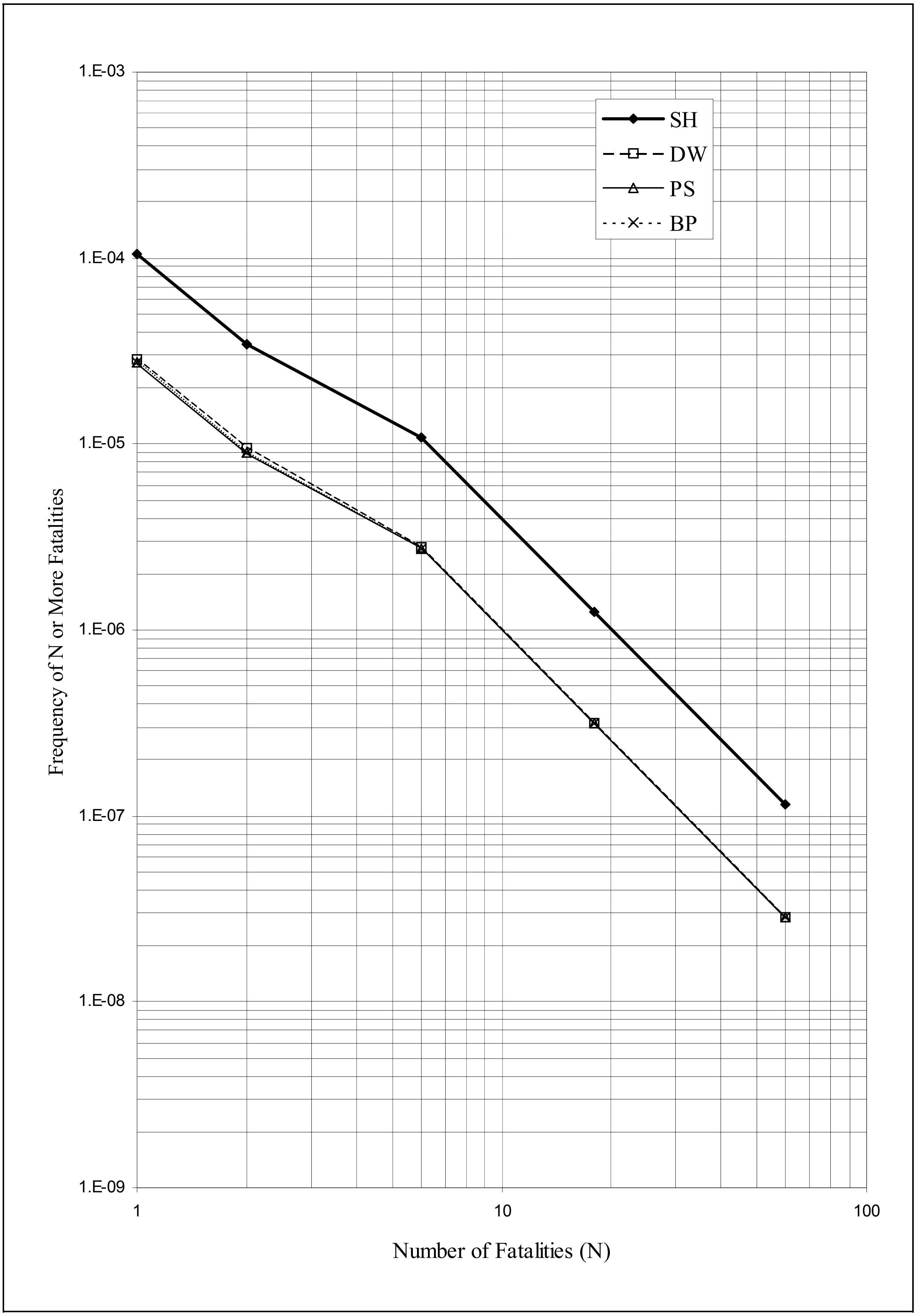

Figure 7.2 - Risk to Public for Individual Excavation Features (<10m)

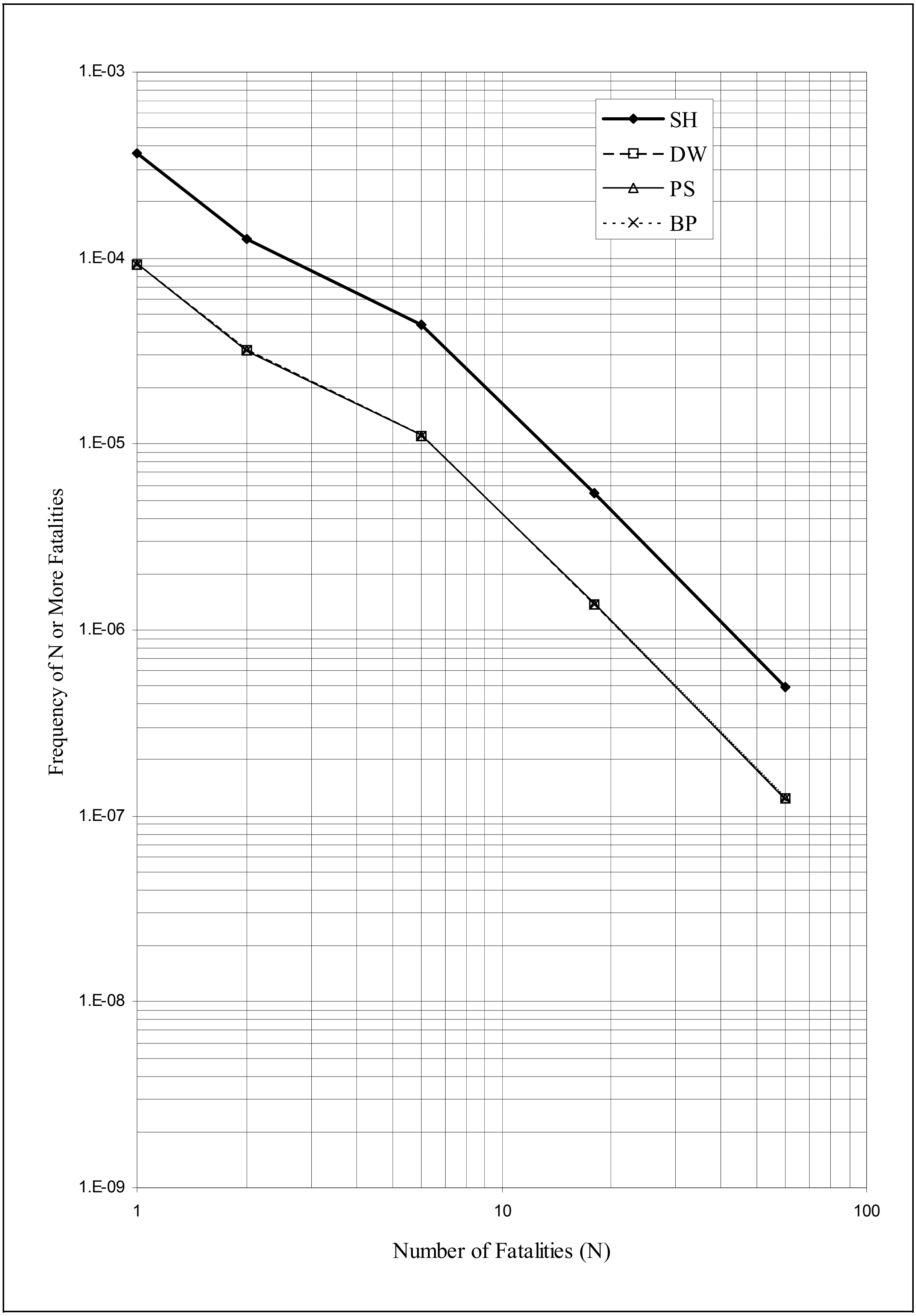

Figure 7.3 - Risk to Public for Individual Excavation Features (>10m)

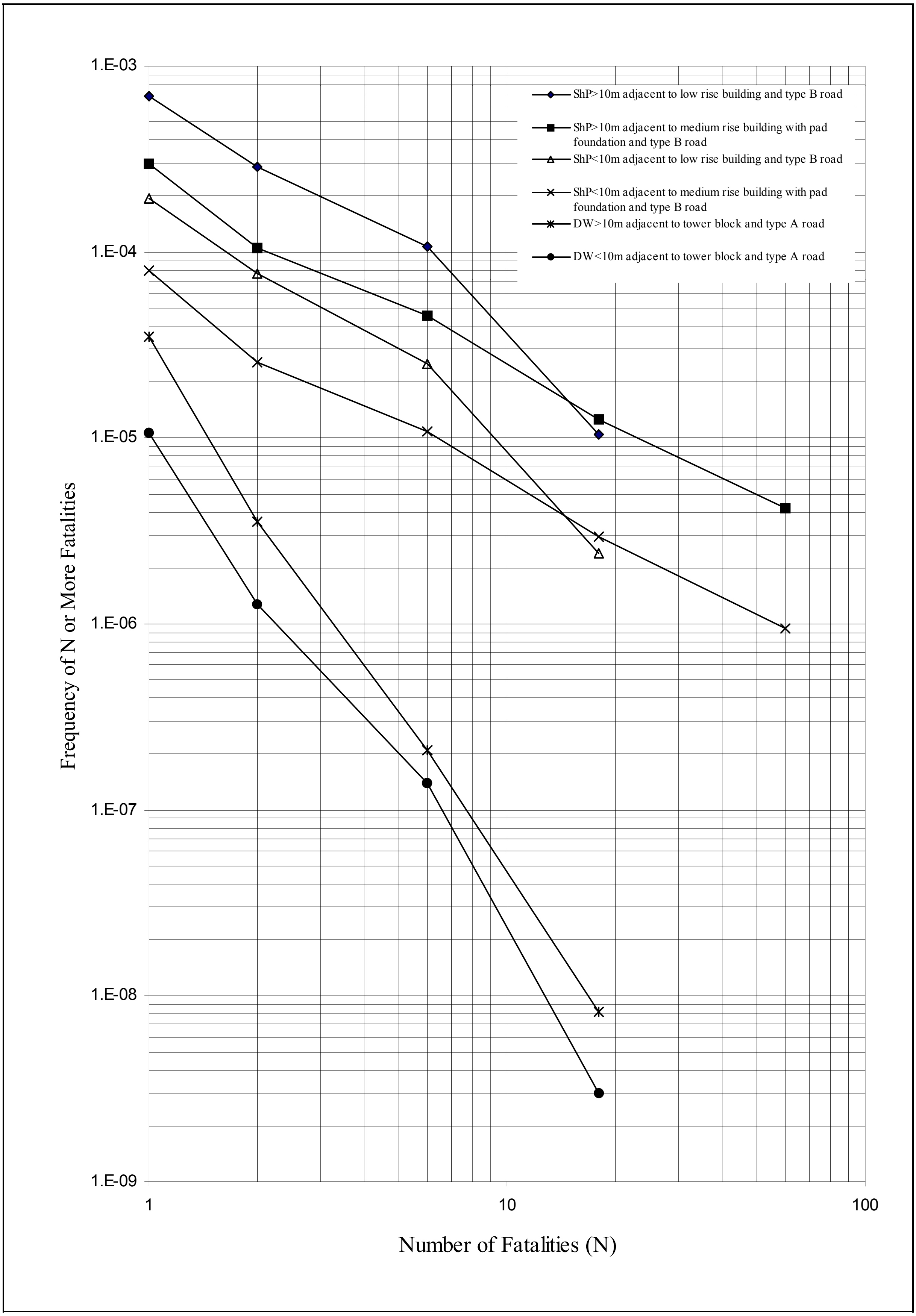

Figure 7.4 - Risk to Public for a Range of Excavation Scenarios

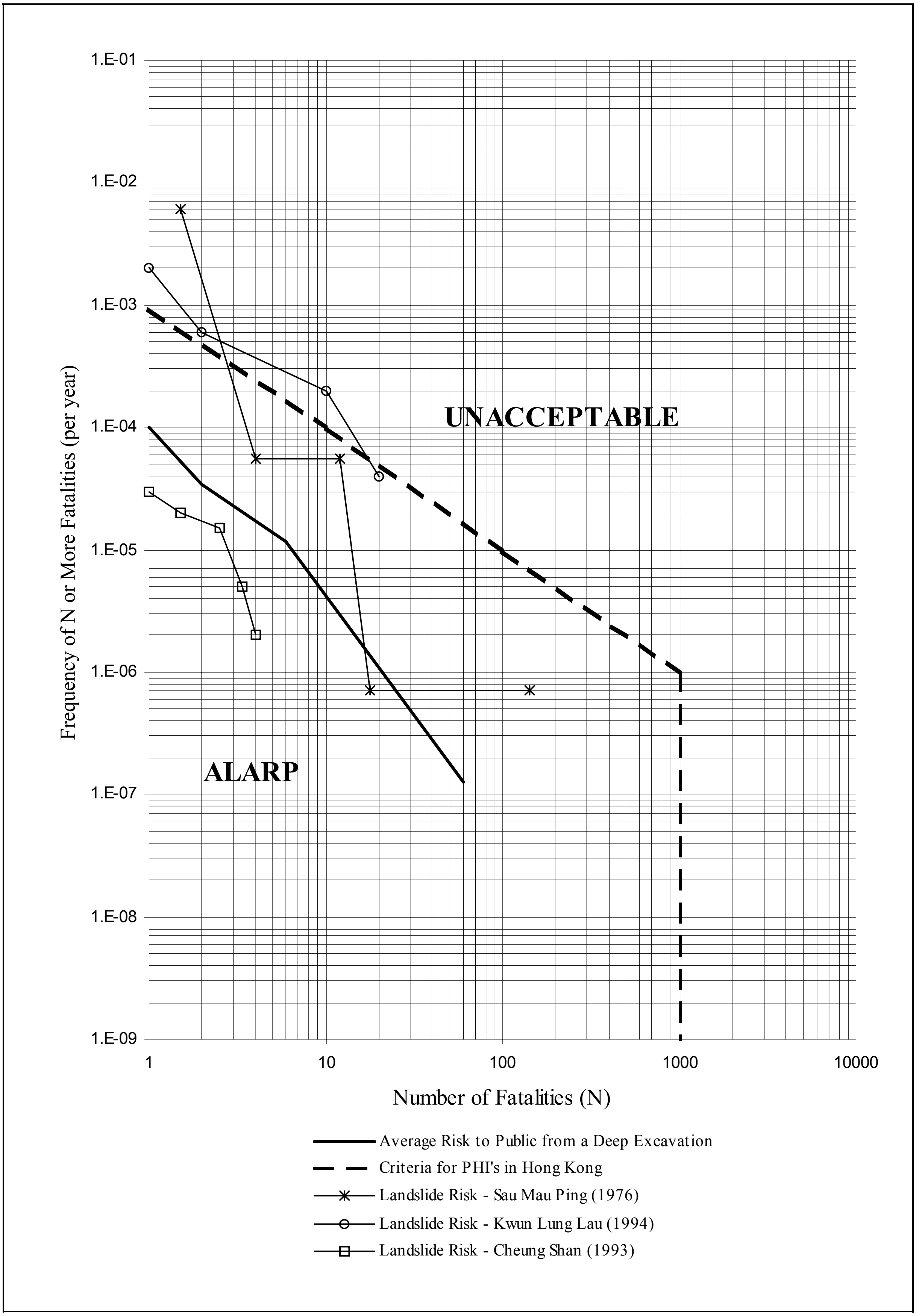

Figure 7.5 - Comparison of Risk for Excavation to Selected Existing Slopes

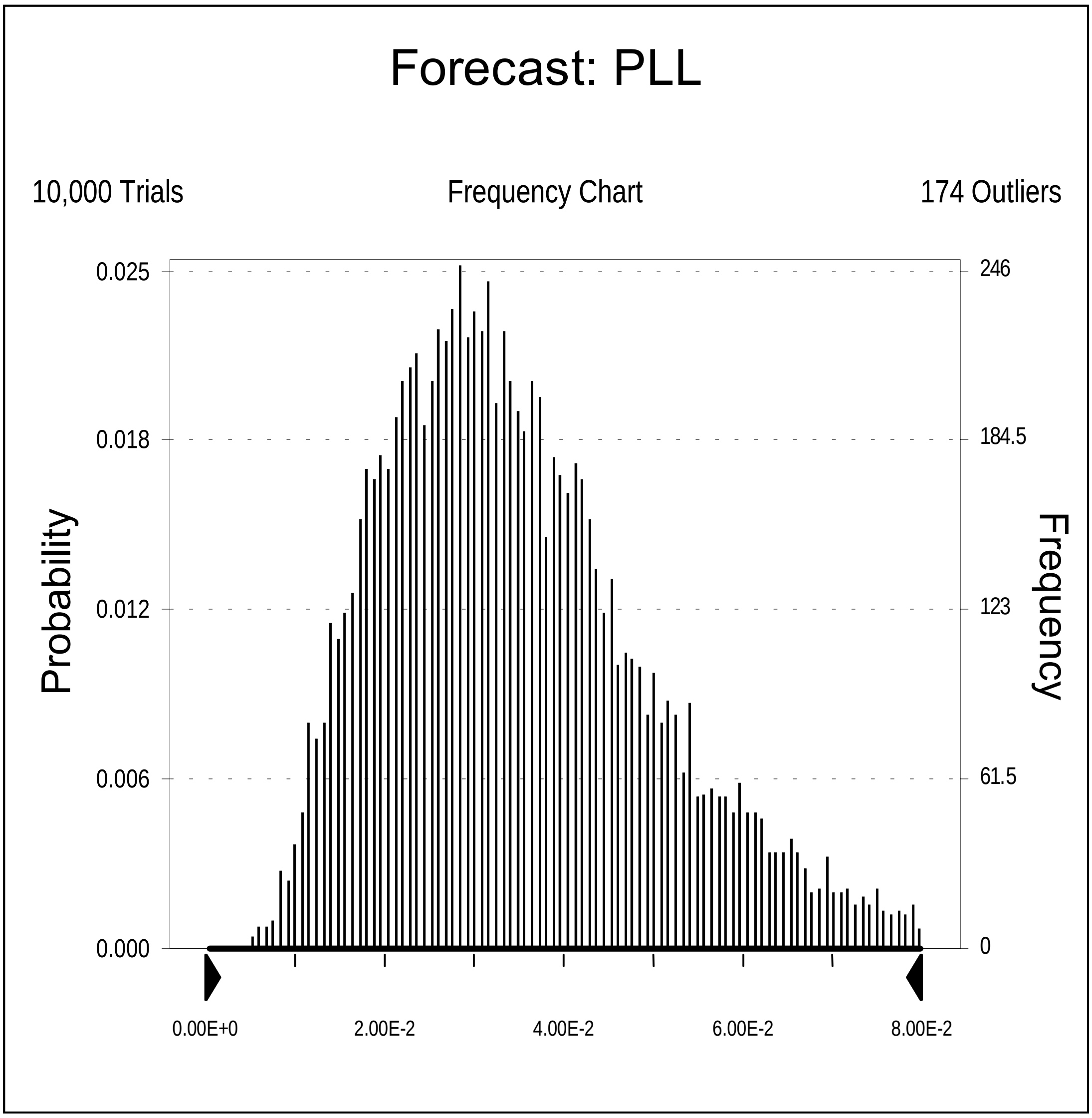

Figure 7.6 - Range of PLL Prediction for Monte Carlo Simulation

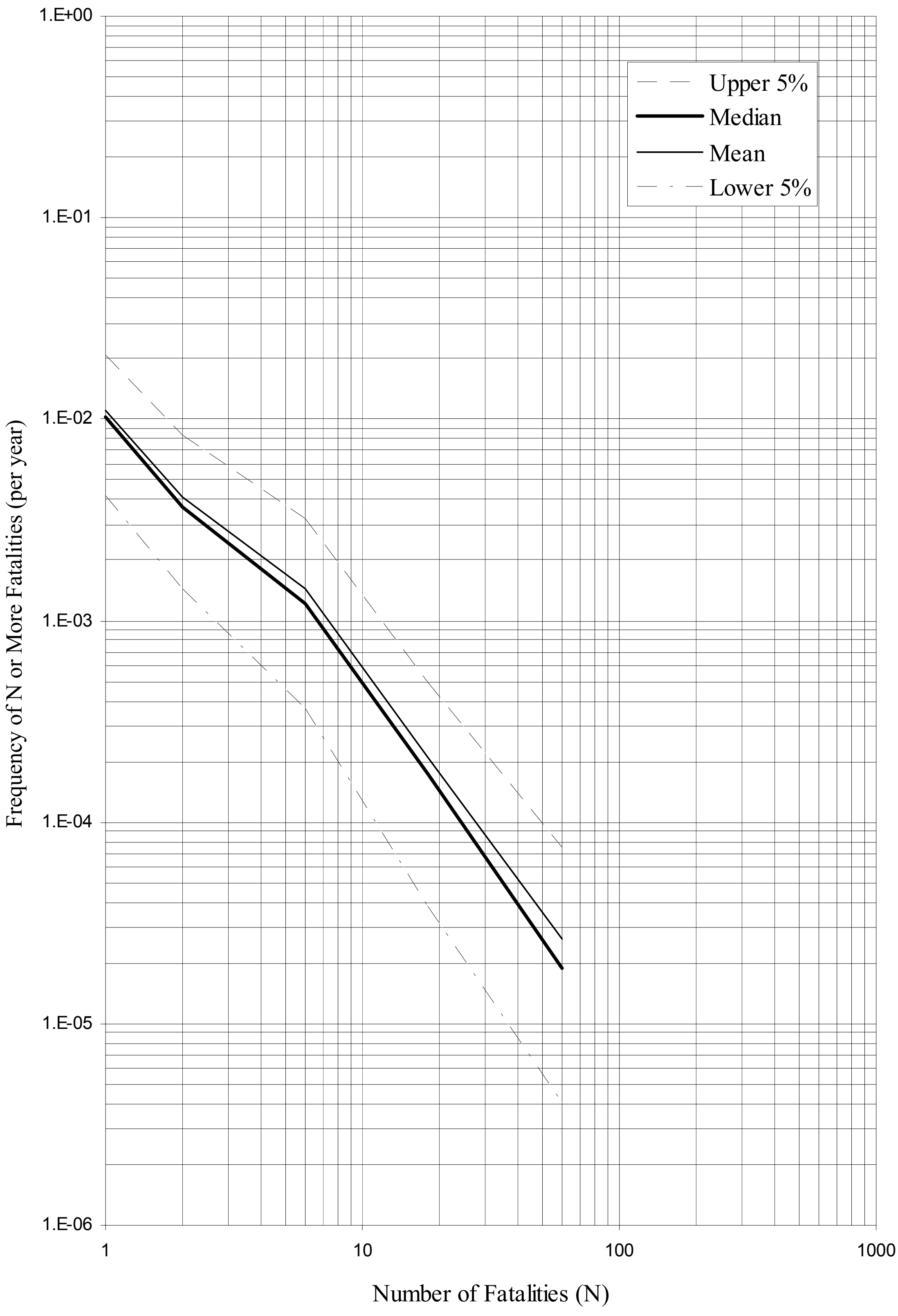

Figure 7.7 - F-N Curve and Confidence Lines Predicted by the Monte Carlo Simulation Risk to Public

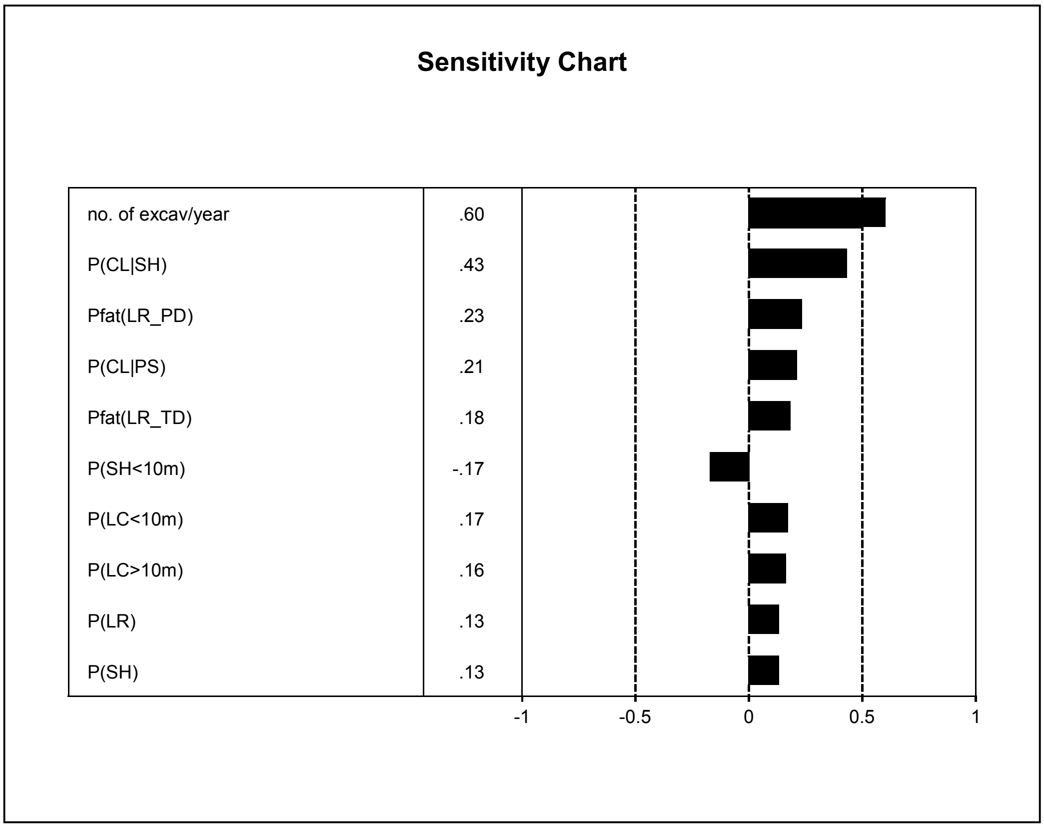

Figure 7.8 – Sensitivity Report from Monte Carlo Simulation

Explanations for Assigning Branch Probabilities
(Notes for Figures)

Figure 6.1, 6.2 & 6.3 "Event Tree for Collapse of Excavation Affecting Buildings"

[1] Building type

Four types of building are considered.

- Tower block, >10 storeys on pile;
- Medium rise building on pile, 5 to 10 storeys;
- Medium rise building on pad foundation, 5 to 10 storeys;
- Low rise building < 5 storeys.

The probability of excavation adjoining the different types of building structures is based on GEO questionnaire survey. (See Appendix D)

[2] Damage to building

The potential for a collapse to cause damage to a building and the type of damage is dependent on the type of collapse and the type of building.

Three levels of damage are considered:

- total collapse of the structure;
- partial collapse of the structure;
- damage affecting non-structural elements.

There is also a possibility that no damage may result.

It is assumed that even a large excavation collapse is unlikely to cause damage that can affect the structural elements of a tower block. Non-structural elements may however be affected.

It is also assumed that a large excavation collapse is unlikely to cause damage that can affect the complete structure of a medium rise building on piles although it may cause partial structural collapse.

Medium rise buildings on pad foundations and low rise buildings could suffer total collapse due to excavation collapse or suffer partial structural collapse and the failure of non-structure elements.

For low rise buildings, the probability of a large excavation collapse causing total structural collapse is assumed to be 0.1 and causing partial structural failure is assumed as 0.4. For a small excavation collapse, values of 0.0 and 0.01 are assumed. For a medium excavation collapse a value of 0.01 and 0.1 are derived as intermediate values.

A similar basis is used for estimating probability of failure for medium rise buildings on pad foundations.

There is a great deal of uncertainty with respect to these parameters and therefore a sensitivity test has been carried out.

Figure 6.4 “Event Tree for Collapse of Excavation Adjoining Road Affecting Footpath”

[1] Day/night probability

Probability of failure is assumed equal for day and night. No data is available to support that failure is caused by work activities (in which case probability of failure during day could be assumed higher). Some of the 34 failure cases record the time of day the collapse occurred but there is not sufficient data to carry out a meaningful analysis.

[2] Pedestrian fall probability

The probability of pedestrian fall is related to time of day, frequency of usage of footpath, extent of failure and type of road and footpath or more appropriately, the location of road and footpath.

The following assumptions are made:

(a) Pedestrian usage is higher in the day than in night;

(b) Probability of pedestrian fall is more likely in the case of large collapse than a small collapse, since a large collapse will involve a footpath length of 20m as against 10m for medium collapse and 3m for small collapse;

(c) For a large collapse, the probability during the day is assumed one, for a medium collapse, a probability of 0.5 and a value of 0.1 is assumed for a small collapse. A small probability of 0.05 is assumed in the event of large collapse during night.

Figure 6.5 “Event Tree for Collapse of Excavation Affecting Road”

[1] Type of road

3 types of road are considered to represent the density of traffic.

Type A road which has an AADT of 1000;
Type B road which has an AADT of 5000;
Type C road which has an AADT of 10,000 and above.

Roads could also be classified as major roads and minor roads. Under major roads, expressway, urban truck road, primary distributor, district distributor and local distributor are included. The proportion of these roads as given in the Traffic Census are as follows:

Major roads 56%
Minor roads 44%

However, the above data is based on roads covered by Census and it is not clear to what extent this represents the road network in Hong Kong.

It could be assumed that all major roads would have an AADT 10,000 and above. A Type C road is therefore assigned a probability of 0.6. It is further assumed that a Type B road has a probability of 0.3 while a Type A road has a probability of 0.1.

[2] Probability of vehicle fall

Probability of vehicle fall into a collapse is a function of the probability of vehicle present within the area affected by collapse.

The area affected by collapse is estimated based on the following assumptions:

- A small collapse is unlikely to affect the road as the collapse area does not extend more than 3m from the boundary of an excavation;
- A medium collapse (which extends 5m) may affect upto one lane while a large collapse (which extends 10m) may affect upto 2 lanes.

Based on the number of lanes affected and the traffic density corresponding to each type of road, the number of vehicles (or its probability) within an affected area can be estimated as follows:

P = W * A * F / (24 * V * 1000)
where
P = probability of vehicle fall
W = length of road (m) (equivalent to width of excavation, decision sight distance assumed as 20m)
F = AADT (vehicles/days)
V = vehicle speed (km/hr)
A = adjustment factor for weather

For LC, type A road	P	=	20 * 1000 * 0.82 / (24 * 35 * 1000)	=	0.02
For LC, type B road	P	=	20 * 5000 * 0.82 / (24 * 35 * 1000)	=	0.1
For LC, type C road	P	=	20 * 10000 * 0.82 / (24 * 35 * 1000)	=	0.2
For MC, type A road	P	=	20 * 500 * 0.82 / (24 * 35 * 1000)	=	0.01
For MC, type B road	P	=	20 * 2500 * 0.82 / (24 * 35 * 1000)	=	0.05
For MC, type C road	P	=	20 * 5000 * 0.82 / (24 * 35 * 1000)	=	0.1

LC = Large collapse
MC = Medium collapse

[3] Type of vehicle

The proportion of various types of vehicle is based on Traffic Census data.

Figure 6.6 “Event Tree for Collapse of Excavation or Significant Displacement Adjoining Road Affecting Gas Pipe”

[1] Gas pipe present

The probability of gas pipe on the road/footpath adjoining an excavation is based on the GEO questionnaire survey. (Appendix D)

[2] Gas pipe failure

The probability of gas pipe failure is a dependent on type of collapse and the design of pipeline. Pipelines could be steel or ductile iron/polyethylene depending on pressure level. Within built-up areas, pressure level is less than 7barg.

The values assumed are based on judgement.

Figure 6.7 “Gas Release Leading to Fire or Dispersion”

[1] Day/night probability

It is assumed that failure could occur equally during day and night. This event tree branch is only significant in determining the number of people on road who could be exposed to a gas release. The number of people exposed will be higher during the day than in the night;

[2] Leak isolated

Isolation is generally expected to take some time and may not be effective in some cases due to residual inventory. Isolation is also dependent on the network and the proximity of governors and other isolation devices. A probability of 0.1 is assumed.

[3] Immediate ignition

Ignition probability would depend on day or night time conditions and on the proximity of ignition sources such as food shop etc. Vehicular traffic could also ignite a gas cloud. However, towns gas is very light and buoyant and therefore will be dispersed easily. A low ignition probability of 0.3 is assumed.

Figure 6.8 “Event Tree for Collapse of Excavation Affecting Workers”

[1] Day/night probability

Probability of failure is assumed equal for day and night. No data is available to support that failure is caused by work activities (in which case probability of failure during day could be assumed higher). Some of the 31 failure cases record the time of day the collapse occurred but there is not sufficient data to carry out a meaningful analysis.

This branch event will determine whether workers would be affected or not. It is assumed that there are no activities during the night.

[2] Workers present in vicinity of failure

It is assumed that workers tend to work in groups in some areas while other areas may be unoccupied. The probability of workers in the vicinity of failure can be estimated based on a number of considerations:

- The area affected by failure (in terms of debris impact area) as a proportion of the whole area within the excavation which could depend on the size of excavation;
- However, workers could be distributed over a number of areas, some in the middle, some near the wall.

Probabilities for workers present have been based on the size of excavation (which is represented in terms of depth) while the number of workers present (which will determine the number of fatalities) is based on type of collapse. This is explained below.

Depth	Probability of workers present
<10m	0.5
>10m	0.3

[3] Workers unable to escape

This depends on the size of collapse relative to the size of excavation. A small collapse in a large excavation may provide a higher probability of escape while a large collapse in a small excavation will have a very low probability of escape. Size of excavation is represented in terms of depth.

The following matrix has been derived:

	Type of collapse		
Size/depth of excavation	Large	Medium	Small
<10m	0.6	0.4	0.2
>10m	0.4	0.2	0.1

LIST OF PLATES

Plate 3.1 - Failed Cofferdam Bangkok 1998

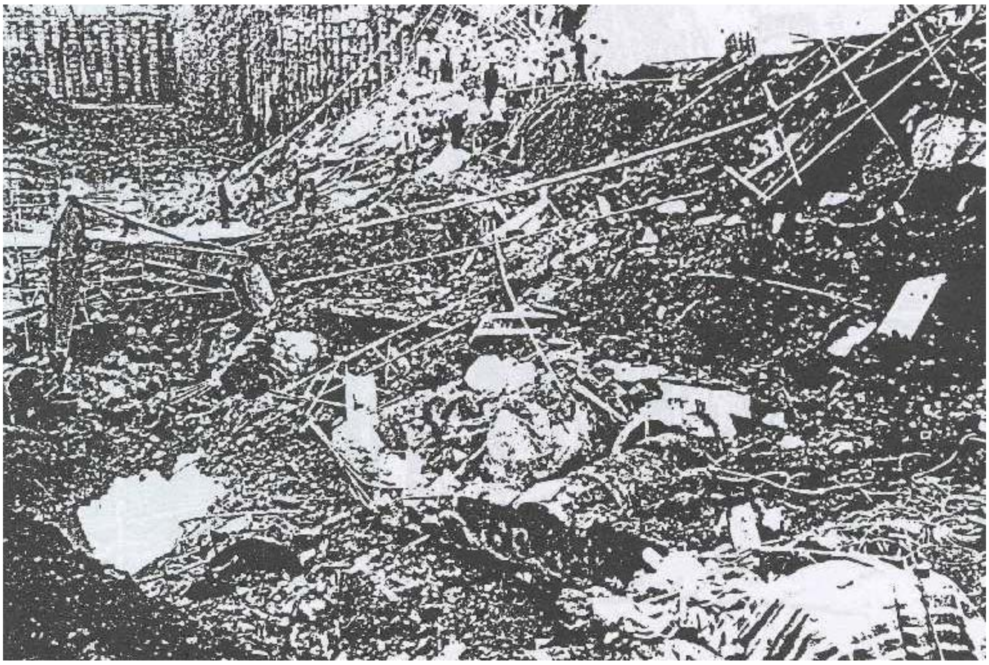

Plate 3.2 - Failed Cantilever Caisson Wall China,1995

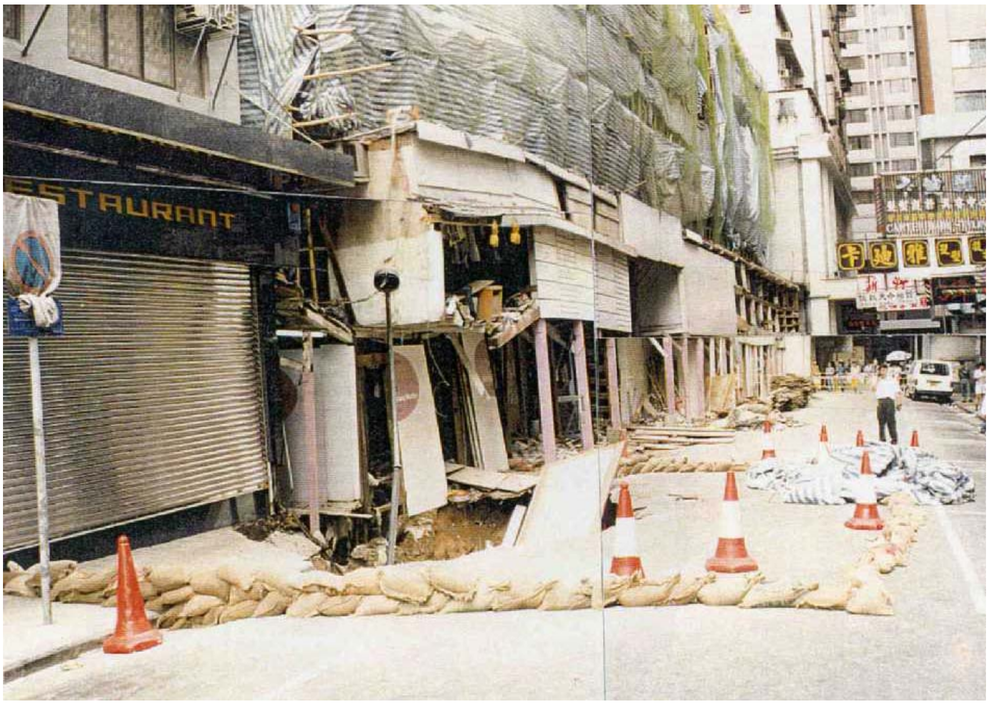

Plate 3.3 - Collapse of Mau Lam Street, Hong Kong June 1991

Plate 3.4 - Collapse of Mau Lam Street, Hong Kong June 1991

Plate 4.1 - Collapse of Queen’s Road Central

APPENDIX A

SAMPLE LETTER TO CONSULTANTS

LETTER SAMPLES

NOT ATTACHED

APPENDIX B

OASYS FREW ANALYSES OF CASE HISTORIES

For part of the study the computer program *Oasys* FREW has been used to re-analyse the failure. The cases, and preliminary conclusions, are as follows.

Case 2:- This is recorded as a medium collapse at No. 2 Yun Ping Road in 1990. A collapse about 9m by 5m occurred at one corner of the site affecting the adjacent road. The collapse was stated to be caused by the omission of struts. The total excavation depth was to be approximately 8.3m with a sheet pile wall and 5 layers of struts. As revealed from the GEO/BD records, the excavation works were in fill over colluvium over CDG. Sheet B2 shows the expected movement, earth pressure and strut forces for the design excavation condition (full details of the data input are included in B3 to B5). The expected deflection of the wall is about 27mm for the design condition. With the upper struts removed this increases to about 90mm as shown in sheet B6. This could well have lead to the adjacent water pipe bursting and the subsequent washout collapse that was observed.

Case 4:- This is recorded as excessive displacement at the Times Square development during excavation with an adjacent building being observed to tilt. The displacement was stated to be caused by ground water drawdown. The total excavation depth was approximately 26.5m with a diaphragm wall and 7 layers of struts. As revealed from the GEO/BD records, the excavation works were in fill over alluvium over CDG. Sheet B10 shows the expected movement, earth pressure and strut forces for the design excavation condition (full details of the data input are included in B11 to B15). The expected deflection of the wall is about 160mm for the design condition. This agrees well reasonably well with the stated prediction in the GEO summary report of 215mm. It is interesting to note from the FREW analysis that half of the predicted movement is due to the pumping test and the initial small excavation prior to installing the first strut. The magnitude of the predicted settlement is sufficient to account for the effects observed at the site in the incident report.

Case 8:- This is reported to be excessive displacement in Beech Street near the junction of Beech Street and Tai Kok Tsui Road. The maximum ground settlement as recorded is about 30mm. The excavation was about 8m deep with a sheet pile wall with 4 layers of strutting. The excessive displacement was recorded due to sheet piling deflection of poor workmanship and gaps were observed between waling and sheet piles. No record showing the non-compliance of works with the approval design. Sheet B17 shows the expected movement earth pressures and strut forces for the designed condition (full details of the data input are included in B18 to B20). As revealed from the analysis, the expected movement is about 80mm in the design condition and the expected maximum ground settlement is approximately ½ of the maximum deflection of the sheet pile wall (i.e. approximately 40mm). It is difficult to discern from this case history whether anything occurred that was not foreseen at design stage. Nevertheless an incident report was filed and poor workmanship in the strutting can not be ruled out or reliably modelled in the analysis without more details.

Case 9:- This is recorded as excessive displacement which caused bursting of the existing water main and damaged of the pavement along Cheung Sha Wan Road NKIL 5955. The excavation depth was approximately 8.4m with a sheet pile wall with 3 layers of struts. Sheet B22 shows the expected movement earth pressures and strut forces for the design condition (full details of the data input are included in B23 to B24). As revealed from the GEO/BD records, the excavation was mainly carried out in bouldery fill material. The predicted maximum deflection of the wall is about 60mm. The recorded maximum ground settlement was 133mm. The cause of the excessive displacement was stated to be due to non-compliance with the approval plan for the installation details for the shoring works. Again it is difficult to model this without better details.

Case 11:- This is the excessive displacement on the carriageway at 165 Argyle Street & 1-3 Stirling Road KIL 10796. The excavation depth is approximately 10.5m with a sheet pile wall with 5 layers of struts. As revealed from the GEO/BD records, the excavation was carried out mainly in fill and marine deposit. Sheet B26 shows the expected movement, earth pressures and strut forces for the designed condition (full details of the data input are included in B27 to B29). The excessive displacement was stated to be caused by over excavation prior to the installation of the third layer of strut. Sheet B30 shows the results of the movement of the sheet pile wall due to over excavation prior to the installation of the third layer of strut (sheets B31 to B32 give full details). The predicted movement of the sheet pile is about 100mm for this condition as compared to 80mm for the design condition. This movement could well have lead to the effects described in the incident report.

Case 17:- This is recorded as excessive displacement at Fanling adjacent to Railway Station FSSTL71. Cracks were found on a nearby carriageway and a newly constructed footbridge settled and was slightly tilted towards the site. The excessive displacement was stated to be caused by dewatering and caisson excavation. The total excavation depth was to be approximately 7.3m with a sheet pile wall and 3 layers of struts. As revealed from the GEO/BD records, the excavation works were mainly in Alluvium with SPT N values ranging from 14 to 60. Sheet B34 shows the expected movement, earth pressure and strut forces for the design excavation condition (full details of the data input are included in B35 to B36). The expected deflection of the wall is 15mm for the design condition. For the case with dewatering only, the expected deflection of the wall is only 3.5mm. The observed effects therefore are likely to be purely due to the dewatering induced settlements and have no connection with the sheet piling wall system.

Case 27:- This is a medium collapse of part of Mau Lam Street in June 1991 (see Plates 3.3 and 3.4) which affected the adjacent road and a three storey building at a corner of the site. The excavation was about 14m deep and relied upon soldier piles and lagging. The collapse was ascribed to over excavation in forming the lagging. A supplementary detailed report (Technical Note 1/92 by T. P. Chan) also observed that the S5 level propping was not complete even though the excavation was below S6 strut level. Sheet B40 shows the expected movement earth pressures and strut forces for the designed condition (full details of the data input are included in Sheet B39). Sheet B42 shows the results for the penultimate excavation depth with the S5 strut removed which shows that this does not have a major effect on the resulting movements. Sheet B44 however shows the case where a reasonable height of soil is not supported by the lagging. In this case the program predicts the soil is observed to be failing through the void in the same way as that likely to have occurred at the site.

Case 29:- This is recorded as excessive displacement at Woosung Street in 1991 with about 300mm settlement occurring. The settlement was stated to be caused by the omission of strutting. The total excavation depth was to be approximately 4.9m with a channel planking wall and 4 layers of struts. The excavation works were assumed to be in fill. Sheet B46 shows the expected movement, earth pressure and strut forces for the design excavation condition (full details of the data input are included in B47 to B48). The expected deflection of the wall is about 40mm for the design condition. No details are available as to the missing or inadequate struts and therefore a sensitivity study has been carried out by omitting the lower struts. If the lowest strut is omitted the deflection increases to about 50mm increasing to about 140mm if the two lowest levels are removed (see Sheets B51 to B59). The bending capacity of the wall is somewhat exceeded in the case of the omission of one strut and exceeded substantially if two levels are omitted.

In two of the six cases of excessive displacement studied , Cases 4 and 8, the re-analysis shows that the design displacement could possibly have been sufficient to give rise to the observed effects. In Case 17 the analysis suggests the cause of the observed effects had very little to do with the wall or excavation. In other cases of excessive displacement construction problems associated with the strutting are cited as the cause and, while the analysis will show this effect for certain assumptions of poor strutting, generally there is not sufficient details of the non compliance of the strutting to be confident the analysis has identified the correct cause.

For cases of collapse the back analysis has also proved useful. In Case 2 modelling the reported omission of struts shows the wall is likely to suffer distress. For Case 27 the analysis shows collapse in much the same way as that reported.

CALCULATION SHEETS

NOT ATTACHED

APPENDIX C

GEO SURVEY

Survey on Deep Excavation Work (For Excavation Depth > 4.5m)

In the absence of specific data on the types of excavation work and other details, which are required for estimation of risk, it is proposed to survey GEO engineers involved in design approval to obtain the following information. The information should be based on your memory of past experience on various types of excavation work that come up for review in the past 2 to 3 years.

Note: Questions a & b should be answered considering recent experience (over the last few years) which does not include the use of caisson walls.

a) What proportion of deep excavations relate to the following wall types:

1)	Sheet pile	______ %
2)	Soldier/pipe pile	______ %
3)	Diaphragm wall	______ %
4)	Large diameter bored pile (>1m diameter)	______ %
	Total	100%

b) What proportion of all excavation work of each wall type relate to depths

		Sheet pile	Solider/pipe pile	Diaphragm wall	Large diameter bored pile
1)	Less than 5 m	______ %	______ %	______ %	______ %
2)	5 to 10 m	______ %	______ %	______ %	______ %
3)	10 to 15 m	______ %	______ %	______ %	______ %
4)	> 15m	______ %	______ %	______ %	______ %
	Total	100%	100%	100%	100%

c) What proportion of all deep excavation work are

1)	Adjoining public roads	______ %
2)	Adjoining buildings	______ %
	Total	100%

d) In the case of excavation work adjoining buildings, what proportion of these buildings are

1)	Tower block (>10 storeys)	______ %
2)	High rise buildings on pile foundation (5 to 10 storeys)	______ %
3)	High rise buildings on pad foundation (5 to 10 storeys)	______ %
4)	Low rise buildings (<5 storeys)	______ %
	Total	100%

e) What proportion of the roads adjoining excavation have a water main ______ %
What proportion of the roads adjoining excavation have a gas pipe ______ %

f) What proportion of deep excavations work are on

1)	Weathered rock	______ %
2)	Reclamation	______ %
	Total	100%

g) Details on person filling this questionnaire
Approximate number of excavation design work reviewed ______
and over how many years ______

SURVEY ON DEEP EXCAVATION WORK (For Excavation Depth > 4.5m)

(a)

	1	2	3	4
	Sheet pile	Soldier/pipe pile	Diaphragm wall	Large diameter bored pile
	7	3	0	0
	6	5	1	1
	5	6	0	1
	6	8	2	0
	0	4	0	1
	3	7	0	0
	3	6	1	0
	1	2	1	0
	15	4	0	2
	7	4	0	0
	5	5	0	0
	5	8	1	1
	20	1	3	1
	12	1	5	1
	8	1	0	4
	10	7	2	1
	8	4	1	0
	3	6	0	1
	6	3	1	0
	5	12	0	1
	8	0	2	0
	10	10	0	3
	13	1	0	0
	2	5	0	3
	7	3	0	0
	8	2	3	2
	8	11	1	0
	2	0	0	0
	1	3	9	0
	13	0	0	0
	13	8	5	0
	3	1	11	0
	2	3	0	0
	9	3	2	1
	10	1	1	0
Total	244	148	52	24

(b)

	1	2	3	4
Sheet pile	< 5 m	5 to 10 m	10 to 15 m	> 15 m
	7	0	0	0
	5	1	0	0
	0	2	3	0
	3	2	1	0
	0	0	0	0
	2	1	0	0
	0	3	0	0
	1	0	0	0
	1	14	0	0
	7	0	0	0
	5	0	0	0
	4	1	0	0
	4	12	4	0
	3	8	1	0
	6	2	0	0
	2	7	1	0
	5	2	1	0
	1	2	0	0
	2	2	1	0
	0	3	2	0
	4	4	0	0
	5	0	5	0
	1	4	8	0
	2	0	0	0
	1	6	0	0
	5	2	1	0
	2	3	3	0
	0	1	1	0
	0	1	0	0
	1	9	3	0
	5	6	2	0
	2	1	0	0
	0	2	0	0
	5	3	1	0
	0	6	4	0
	91	110	42	0

	1	2	3	4
Soldier/pipe pile	< 5 m	5 to 10 m	10 to 15 m	> 15 m
	3	0	0	0
	3	2	0	0
	0	3	3	0
	1	2	4	1
	3	1	0	0
	2	4	1	0
	0	6	0	0
	1	1	0	0
	0	4	0	0
	4	0	0	0
	5	0	0	0
	4	3	1	0
	0	1	0	0
	0	1	0	0
	2	0	0	0
	6	1	0	0
	2	1	1	0
	4	2	0	0
	0	2	1	0
	7	4	1	0
	0	0	0	0
	5	0	5	0
	1	0	0	0
	2	3	0	0
	0	3	0	0
	0	2	0	0
	4	4	3	0
	0	0	0	0
	0	1	2	0
	0	0	0	0
	5	3	0	0
	1	0	0	0
	0	0	1	2
	1	2	0	0
	0	1	0	0
	66	57	23	3

SURVEY ON DEEP EXCAVATION WORK (For Excavation Depth > 4.5m)

	1	2	3	4
Diaphragm wall	< 5 m	5 to 10 m	10 to 15 m	> 15 m
	0	0	0	0
	0	0	1	0
	0	0	0	0
	0	0	0	2
	0	0	0	0
	0	0	0	0
	0	0	1	0
	0	0	1	0
	0	0	0	0
	0	0	0	0
	0	0	0	0
	0	0	0	1
	0	0	3	0
	0	2	3	0
	0	0	0	0
	0	0	0	2
	0	0	0	1
	0	0	0	0
	0	0	1	0
	0	0	0	0
	0	2	0	0
	0	0	0	0
	0	0	0	0
	0	0	0	0
	0	0	0	0
	0	0	1	2
	0	0	0	1
	0	0	0	0
	0	0	1	8
	0	0	0	0
	0	0	4	1
	0	3	7	1
	0	0	0	0
	0	0	0	2
	0	0	1	0
Total	0	7	24	21

	1	2	3	4
Large diameter bored pile	< 5 m	5 to 10 m	10 to 15 m	> 15 m
	0	0	0	0
	0	0	0	1
	0	0	1	0
	0	0	0	0
	0	0	1	0
	0	0	0	0
	0	0	0	0
	0	0	0	0
	0	0	2	0
	0	0	0	0
	0	0	0	0
	0	0	1	0
	0	0	0	1
	0	1	0	0
	0	4	0	0
	0	0	0	1
	0	0	0	0
	0	0	1	0
	0	0	0	0
	0	1	0	0
	0	0	0	0
	0	3	0	0
	0	0	0	0
	0	3	0	0
	0	0	0	0
	0	0	1	1
	0	0	2	0
	0	0	0	0
	0	0	0	0
	0	0	0	0
	0	0	0	0
	0	0	0	0
	0	0	0	0
	0	1	0	0
	0	0	0	0
	0	13	9	4

(c)

1	2
Adjoining roads	Adjoining bldgs
0	10
3	10
4	8
7	9
0	1
5	5
5	5
2	2
13	8
0	11
0	0
5	10
12	13
5	14
6	7
12	8
6	7
3	2
8	2
9	9
5	5
11	12
14	2
5	5
0	10
7	8
12	8
2	0
10	3
6	7
18	8
12	3
4	1
7	8
7	5
225	226

SURVEY ON DEEP EXCAVATION WORK (For Excavation Depth > 4.5m)

(d)

	1	2	3	4
	Tower block	High rise bldgs on pile	High rise bldgs on pad	Low rise bldgs
	0	0	0	10
	4	1	0	5
	2	2	1	3
	2	4	1	2
	0	0	0	1
	0	2	2	1
	0	5	0	0
	1	0	0	1
	2	5	0	1
	0	0	0	11
	0	0	0	0
	1	4	4	1
	2	4	4	3
	7	4	2	1
	0	0	0	7
	2	2	0	4
	2	1	2	2
	1	0	0	1
	0	0	0	0
	2	3	2	2
	0	2	0	3
	3	3	3	3
	0	1	1	0
	3	2	0	0
	0	4	0	6
	3	3	1	1
	4	2	1	1
	0	0	0	0
	2	0	1	0
	1	1	0	5
	5	1	0	2
	2	0	0	1
	1	0	0	0
	4	2	1	1
	3	2	0	0
Total	59	60	26	79

(e)

1	2
Water main	Gas pipe
0	0
2	2
3	2
6	5
0	0
4	2
2	2
2	0
4	9
0	0
0	0
3	1
6	5
3	3
1	0
10	6
3	3
0	0
0	0
5	4
4	1
1	1
1	1
5	5
0	0
2	1
8	6
1	0
9	8
0	0
18	9
4	1
1	2
6	4
6	4
120	87

(f)

1	2
Sapolite	Reclamation
10	0
10	3
6	6
12	4
5	0
10	0
8	2
4	0
13	8
11	0
10	0
10	5
18	7
10	9
13	0
18	2
6	7
10	0
0	0
16	2
10	0
11	12
8	8
8	2
10	0
6	9
16	4
0	2
3	10
10	3
13	13
3	12
5	0
12	4
8	4
323	138

(g)

1	2	3
No. Designs	Years	No. to be used
10	4	10
20	3	13
15	3	12
40	4	16
5	1.5	5
10	3	10
10	3	10
4	0.5	4
120	3	21
11	0.5	11
10	2	10
30	3	15
270	3	25
80	1.5	19
20	0.5	13
100	5	20
20	0.5	13
10	0.5	10
10	?	10
60	3	18
10	2	10
200	5	23
40	3	16
10	3	10
10	2.5	10
30	5	15
100	3	20
2	0.33	2
20	5	13
20	1	13
400	3.5	26
30	1.5	15
5	2	5
30	3	15
15	1.5	12
1777	85.83	470

APPENDIX D

DERIVATION OF FAILURE PROBABILITIES

Estimation of Probability of Failure

The number of incidents of collapse and excessive displacement together with the number of excavations undertaken over the last 15 years is given in Table 5.3.

As can be seen, there is limited data on failures (in some cases, there are zero failures). There is also limited data on number of excavations, particularly in the case of diaphragm wall as compared to other wall types. In the light of the above, the following approach is adopted to estimate the probability of failure. This is shown as below.

The probability of failure is derived as :

P(failure) = r/N

where r is the number of failures and N is the number of excavations.

The confidence limits of P(failure) at (1-∝) confidence level is estimated as:

$P = x^2_{f;1-\propto}/2N$

where, ∝ is confidence level factor [∝ = (100-A)/100, where A is percentage confidence.level)
f is degrees of freedom, f=2r or 2r+2 (if it is assumed that a further failure was just due to occur).
x^2 is chi-square distribution. Standard tables provide values for chi-square distribution as a function of ∝.

Based on the above, the 50% confidence level (ie, ∝ = 0.5) on the failure probability can be determined as follows:

Table D.1 : Derivation of Failure Probabilities Based on 50% Confidence Level

Parameter	Sheet pile wall	Diaphragm wall	Caisson wall	Soldier pile/pipe wall
No. of excavations	639	136	530	482
No. of collapses	8	0	0	1
Degrees of freedom(f)	16	2	2	4
$x^2_{f;0.5}$	15.3	1.39	1.39	3.36
Derived probability of collapse	0.012	0.005	0.001	0.003
No. of excessive displacements	10	4	1	1
Degrees of freedom	20	8	2	4
$x^2_{f;0.5}$	19.3	7.34	1.39	3.36
Derived probability of excessive displacement	0.015	0.027	0.001	0.003

The following assumptions are made for failures associated with bored pile wall,

- probability of collapse of bored pile wall is similar to the probability of collapse of diaphragm wall, ie 0.003 per excavation;
- probability of excessive displacement due to bored pile wall installation is similar to that derived for soldier pile/pipe pile wall, ie 0.003 per excavation.

APPENDIX E

ASSESSMENT OF HAZARDS DUE TO GAS PIPE FAILURE CAUSED BY EXCESSIVE DISPLACEMENT OR COLLAPSE FROM DEEP EXCAVATIONS

This appendix broadly addresses some of the issues involved in the assessment of hazards due to gas pipe failure.

The domestic gas (called the Town gas) distribution network is operated by The Hong Kong and China Gas Company. Gas is produced at a pressure of 3500kPa and transported in high pressure (HP) transmission pipelines at 3500kPa. The HP lines are permitted to be laid only in suburban areas away from residential developments.

Gas from these high pressure pipelines enters the intermediate pressure network (IPB, 400 to 700kPa) which is further stepped down to lower intermediate pressure (IPA, 240 to 400kPa) or the medium pressure network (MP, 7.5 to 240kPa). The MP pipes form the major reticulation network in built-up areas. For supply to consumers, the MP level is stepped down to low pressure level (either LPA, below 2kPa or LPB, 2 to 7.5kPa).

Gas pipeline in built up areas such as Hong Kong Island and Kowloon are therefore more likely to be operating at MP (7.5 to 240kPa) or LP (<7.5kPa) level.

The Town gas pipelines (IP/MP/LP levels) are constructed of ductile iron or polyethylene. Pipelines that were laid more recently, ie, during the past 3 to 5 years would be of polyethylene material while older pipelines would be of ductile iron. Whether excessive displacements or collapse can cause pipeline failure would possibly depend on the influence zone for example, in the event of a collapse.

In the event of pipeline failure, Town gas (molecular weight 15, consisting of 48% by volume hydrogen, 29% methane, 20% CO_2) which is lighter than air (molecular weight 29) will disperse quickly, rising upwards due to buoyancy effects. The potential for significant gas build-up (at street level) leading to a flash fire or an explosion is therefore considered to be very low. Even in the case of a gas release in built-up areas of the city, where gas pipelines are generally laid under the footpath of roads, such releases are not likely to result in significant confinement as to cause an explosion.

In the event of ignition of release, a jet flame will ensure due to internal pressure in the pipeline. Ignition may occur immediately upon failure due to electrostatic generation or due to presence of ignition sources nearby. Delayed ignition may be caused by other ignition sources such as passing vehicles.

The jet flame following a rupture will mostly be oriented upwards. The effects of thermal radiation from a vertical jet flame on humans will be limited to 5 to 10m from the source. It may cause burns but no fatality since persons exposed to radiation would escape to safer areas. A vertical jet flame could however, affect buildings on the edge of the road shoulder as it could set off secondary fires due to thermal radiation.

In addition to Town gas network, LPG pipelines may also be found but currently LPG pipelines in public roads operate only in 2 or 3 specific areas in Hong Kong and therefore not considered any further. The effects of a LPG release, however, would be different as it is a dense gas and therefore significant build-up may occur.

APPENDIX F

EXAMPLE CALCULATION

Sample Calculation

The starting point for computation is the event tree in Figure 5.3.

The initiating event probability is considered as 1 in this event tree. This makes it simple to vary the outcome based on number of excavations per year.

The outcome results are based on:

No. of excavations per year = 100/yr (see Section 5.6)

The distribution of wall types is based on analysis of GEO questionnaire survey. This is given in the last column of Table 5.6.

Sheet pile :	ShP	52%
Diaphragm wall:	DW	11%
Large bored pile wall:	BP	5%
Soldier pile/pipe pile:	PS	32%

The depth of excavation which is dependent on wall type is based on analysis of GEO questionnaire survey which is summarised in Table 5.6.

		Depth >10m	Depth <10m
Sheet pile	ShP	0.17	0.83
Diaphragm wall	DW	0.87	0.13
Large bored pile	BP	0.50	0.50
Soldier/Pipe	PS	0.18	0.82

The probability of excessive displacement or collapse is based on the values given in Table 5.7 and 5.8 which are themselves derived from Table 5.3. The probability of failure is dependent on wall type.

Excessive displacement	**ED**	
Sheet pile	ShP	0.015
Diaphragm	DW	0.027
Large bored pile	BP	0.003
Soldier/pipe	PS	0.003

Collapse	**CL**	
Sheet pile	ShP	0.012
Diaphragm	DW	0.003
Large bored pile	BP	0.003
Soldier/pipe	PS	0.003

The probability of various sizes of collapse (small, medium and large) are assumed as a function of depth. This is given in Table 5.5.

Depth >10m	Small collapse	0.2
	Medium collapse	0.5
	Large collapse	0.3
Depth <10m	Small collapse	0.65
	Medium collapse	0.3
	Large collapse	0.05

Outcome probabilities are given in Figure 5.3 for each outcome. However, it is to be noted that outcome probability for all similar outcomes in the event tree in Figure 5.3 are summed up as follows:

Excessive displacement =	0.01188
Small collapse =	0.004274
Medium collapse =	0.002623
Large collapse =	0.000783

It is also to be noted that collapse could affect a building or a road facility which includes road, footpath and gas pipe.

The probability that collapse affects a building is assumed as 0.5 and the probability that it affects a road/footpath/gas pipe (all simultaneously) is assumed as 0.5.

The above values are therefore multiplied by 0.5 and then carried forward into other event trees.

Collapse

Figures 6.1, 6.2 and 6.3 model the outcome of collapse affecting buildings for small, medium and large collapse. Figure 6.4 model the outcome of collapse affecting footpath. Figure 6.5 model the outcome for collapse affecting road. Figures 6.6 & 6.7 model the outcome of collapse affecting gas pipe.

Excessive displacement

Figure 6.6 models the outcome of excessive displacement affecting a gas pipe in a road. It is assumed that excessive displacement has no significant effect on buildings with potential for fatalities.

Fatalities

Table 6.1 provides the number of fatalities and the associated probabilities for collapses affecting buildings resulting in failure - structural or non-structural.

Table 6.2 provides the number of fatalities and the associated probabilities for collapses affecting roads resulting in vehicle fall.

Table 6.3 provides the number of fatalities and the associated probabilities in the event of pedestrian fall.

Table 6.4 provides the number of fatalities and the associated probabilities in the event of a gas release.

Table 6.5 provides the number of fatalities to workers and the associated probabilities.

Illustration

Take the case of sheet pile, depth >10m that leads to a large collapse.

From Figure 5.3, the outcome probability for the above scenario is 0.00031824 (ie, 3.2×10^{-4}).

Go to Figure 6.3 for a large collapse affecting buildings. Assume a medium rise building on pad foundation (this has a probability of 0.12). The probability of total structure damage is

$0.12 \times 0.01 = 0.0012$ (ie, 1.2×10^{-3}).

Go to Table 6.1 to estimate the probability of fatality. The probability of total structure damage of medium rise building to cause 60 deaths is given as 0.2.

The outcome frequency (f) $= 100$ excavations/yr $\times 3.2 \times 10^{-4} \times 1.2 \times 10^{-3} \times 0.2 \times 0.5$
$= 3.8 \times 10^{-6}$ per year

The number of fatalities (N) = 60

The multiplication factor 0.5 is to account for 50% chance of collapse affecting buildings.

In order to compute the cumulative frequency (F), ie the sum of the frequencies (f) of all events that cause 60 or more deaths, the outcome frequencies (f) for all events that can cause 60 fatalities need to be estimated.

60 fatalities result only from total collapse of medium rise buildings on pad foundation.

This can be caused by large and medium collapses only. The scenarios resulting in 60 fatalities for a sheet pile excavation, >10m are:

- large collapse affecting medium rise buildings on pad;
- medium collapse affecting medium rise buildings on pad.

The outcome frequency for the former case is given above. The outcome frequency for the latter case is estimated similar to the above as:

f (medium collapse) = 100/yr x 5.3 x 10^{-4} x 1.2 x 10^{-4} x 0.2 x 0.5 = 6.4 x 10^{-7} /yr

N = 60

The cumulative frequency F = 0.6 x 10^{-6} + 3.8 x 10^{-6} = 4.4 x 10^{-6} per year.

To calculate the overall probability of 60 deaths in any one incident the overall rate of Medium and Large collapses need to be considered as follows:

Rate of Medium collapse x rate of collapse of a medium rise on pads x risk of 60 fatalities

0.002623x 0.5 x 0.12 x 0.001 (Fig 6.2) x 0.2 = 3.15E-8

Rate of Large collapse x rate of collapse of a medium rise on pads x risk of 60 fatalities

0.000783x 0.5 x 0.12 x 0.01 (Fig 6.3) x 0.2 = 9.40E-8

to give a total risk of 60 deaths of 1.25E-7 per excavation or 1.25E-5 per year assuming 100 excavations each year. It can be seen that this is the value plotted on Figure 7.1.