STUDY OF OLD MASONRY RETAINING WALLS IN HONG KONG

GEO REPORT No. 31

Y.C. Chan

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

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

A.W. Malone
Principal Government Geotechnical Engineer
May 1996

FOREWORD

This report records a comprehensive study I carried out in 1981 on masonry retaining walls in Hong Kong. It includes a review of the construction practice and structure of masonry retaining walls, analyses of case histories of wall failures, an examination of the structural behaviour of masonry walls and suggestions on the approach to investigate stability of masonry retaining walls and the follow up research. Of these, the findings on wall structure have been useful to the planning and interpretation of ground investigation. The concept of structural instability has the greatest impact on the stability assessment of the masonry retaining walls in Hong Kong. The suggestions for research were imaginative but some are no longer appropriate given the technological advancement in the past 15 years. The procedures described in the report have been influencing stability investigation of masonry retaining walls to this date.

Mr Andrew Hui assisted me in the stress analysis of masonry walls. The support and encouragement of Mr H B Phillipson and Mr M C Tang were important for the completion of the project. Their contributions are grateful acknowledged.

Y.C. Chan
Chief Geotechnical Engineer/Special Projects

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1. INTRODUCTION

1.1 Background

Ever since the Hong Kong Island was described as "a barren island with hardly a house upon it" by Palmerston in 1841, it has developed into a large urban area that houses as many as 1 million people. Most of the houses were built on lands reclaimed from the sea as well as terraces formed on steep hill sides. To a less extent similar terraces are also found in Kowloon and the New Territories. Such terraces, in their early form, are usually bound by high masonry retaining walls at the rear, and occasionally, with walls in front as well. These walls support a variety of materials ranging from insitu decomposed rock and colluvium of various ages to backfills derived from these materials. In the development of Hong Kong the need to maximise the use of land has resulted in houses being constructed extremely near to these retaining walls, in some cases as close as one metre.

Failure of these walls are infrequent but when they occur they can be catastrophic inflicting extremely heavy losses to property and human life. This was the case in 1925 when the Po Hing Fong failure destroyed 7 brick buildings with a loss of 150 lives. Drastic improvements in the structural standards of buildings reconstructed after the World War II have largely reduced the likelihood of damages of a similar scale. Yet, the fact that a retaining wall behind one's house is an uncertain threat to its owner is understandably regarded as an unacceptable risk by the public.

In the territory, there are a total registration of 2584 retaining walls of which 1764 are masonry in construction.

To deal with such a large number of walls with resources that can reasonably be mobilised, investigation has to be done on a priority basis. This depends on the combined consideration of the likelihood and the consequence of failure of individual walls. This is embodied in the Ranking System in which subjective formulae have been used to calculate various 'scores' from basic wall parameters measured during the Phase 1A Study on Cut Slopes and Retaining Walls.

In GCO, the procedure for studying retaining walls starts with the selection of batches of high priority walls from the ranking list and then subject them to a multistage study scheme. The Stage 1 studies consist of more detailed inspection accompanied by desk studies. Walls that show positive signs of possible instability will be recommended for Stage 2 detailed studies in which site investigations, laboratory testings and other ground-condition evaluations techniques are used to determine the stability of the walls. Walls that are proved to be liable to fail, in foreseeable unfavourable conditions, will then proceed through a Stage 3 study in which stabilisation measures are designed. The remedial works will then be carried out and maintained by the responsible office.

For the successful operation of such a finely balanced study system, accurate diagnosis and recommendations are required. This demands investigation engineers with extensive experience in the behaviour of old masonry retaining walls. Such experienced engineers are not readily available in Hong Kong. Moreover, technical literature in English language on these types of structures is rare and therefore a new investigation engineer will have to go through a lengthy trial-and-error process before he can acquire sufficient experience to make the correct decisions. This is not an ideal way and can be avoided if the responsible offices can organise operations to collect relevant information on old masonry retaining walls so that new engineers can acquire the necessary know-hows in a reasonably short period of time. Such is the aspiration behind and the aim of the present study programme.

1.2 Scope of the Study

The present study starts with reviews on past studies of masonry retaining walls, and on the structure and methods of construction of these walls. Observations of failure of masonry retaining walls in Hong Kong and in England are then examined for possible mode of failure of these walls and the common features associated with the unstable walls. Possible factors affecting the stability of masonry retaining wall against static, foundation and structural failures are then analysed to determine their relative importance, as well as the possible causes of formation of bulges prior to failure. Information on the physical features of trees most commonly found on masonry walls in Hong Kong is collected in an attempt to rationalise the evaluation of effect of trees on masonry walls. Some field techniques for the investigation of walls are tried or considered.

2. REVIEW OF PAST STUDIES

2.1 Study by Binnie & Partners (Hong Kong) (1978)

In April 1977, Binnie & Partners (Hong Kong), acting as the engineering consultant to the Public Works Department, commenced a study on a 0.2 square kilometre area in Sheung Wan. The area was bound by Queen's Road in the north, Hospital Road, Seymour Road and Caine Road to the south, Aberdeen Street to the east and Possession Street, New Street, and the Tung Wa Hospital to the west. There was a total of 135 walls in the area of which 131 were of old masonry type.

The study comprised a review of past records and archives, site inspections, as well as ground condition/wall thickness evaluation through the execution of a site investigation programme on walls showing some signs of instability. The site investigation consisted of 11 vertical and 3 horizontal drill holes, 34 horizontal probes by pneumatic drills as well as 2 trial pits. Triaxial tests and other index tests were carried out on samples recovered from the investigations. The soil parameters used in subsequent analysis, as summarised in Table 2.1 were derived either from these tests or from test results on similar materials in other study projects.

Analyses of the stability of these masonry retaining walls were by conventional approaches. The factors of safety calculated from these analyses were generally below 1.0 and may have inspired the conclusion that old masonry walls do not conform with the contemporary design standards.

This conclusion was supported, among other arguments, by that the walls are "generally too thin and in many cases form only a facing". Examination of the drill hole logs in the present study revealed wrong interpretation of wall thickness in at least one case, probably due to insufficient knowledge then on the general structure of masonry retaining walls. Also, the ample width of many of these old walls have been demonstrated by recent investigations by Geotechnical Control Branch (GCB) of the Buildings Ordinate Office on high "consequence score" walls. The comments concerning wall thickness and their factors of safety made in this area study report should thus be treated with scepticism.

On the other hand, this report contains very good factual description of the geology, hydrogeology, topography of the area, as well as the conditions of 45 walls. These, together with the site investigation records, should form good reference for future studies on walls in this area.

The Phase 1A Study on Cut and Natural Slope and Retaining Walls was also commenced in April 1977 by Binnie and Partners (Hong Kong) and took one year to complete. In this exercise all cut slopes and retaining structures exceeding 3 m in height in the urban area and the vehicle accessible rural area were identified, and numbered. Based on a set of basic parameter collected on site, recommendations designated with relative priority were made on works necessary to maintain their stability. This basic parameters were recorded in field sheets accompanied by photographs, and form the bulk of the Phase 1A study report.

2.2 GCB's Report on Study of Old Masonry Retaining Walls (1980)

In March 1979, a proposal for a study of old masonry retaining walls was composed in the Geotechnical Control Branch of the Buildings Ordinance Office to learn more about the standard forms of such old walls, the possible presence of an historical or geographical pattern, and how each principal type of retaining walls fail. This, it was perceived, might provide a more rational and logical way of assessing the safety of the existing walls with some savings in time and cost. The project was carried out in approximately one year's time comprising survey of forms of construction from partially demolished/collapsed walls and BOO file records, dummy analyses with assumed wall configuration and parameters, as well as surveys on past failure cases.

Dimensions of some of the failed and stable walls were obtained in the study. These, when plotted on a height vs base width chart, showed that walls with base width less than 1/3 of height were liable of instability (Figure 2.1). This was related with the requirement of no tension in the wall base under a particular wall configuration and soil strength parameters in the absence of groundwater.

Two techniques aiming at quick measurement of wall thicknesses were also tried. The first made use of a straight edge to measure depths of weepholes (weephole probe). The measured value was found to be compatible with the thickness of the wall measured by core drilling. The second technique was to measure wall thickness by seismic reflection method. It was abandoned because interpretation of the result required an assumed velocity of propagation of compression wave (V_p), a physical quantity which varies over wide ranges for different wall materials.

In the report, it was also stated that virtually all wall failures are associated with heavy rainfall or burst water mains, and that bulges of walls may be indicative of development of failures involving circular failure surfaces and wall shearing.

Apart from the report, the study also led to the collection of structural details of over 20 old walls. The descriptions combined with photographs of exposed sections of wall are very helpful for an understanding of the structures of old masonry retaining walls.

2.3 GCO's Study on Old Masonry Retaining Walls (1980)

A study of old masonry retaining walls was carried out in the Geotechnical Control Office in July 1980. The study was aimed at the "establishment of a criterion based on simple site investigation and desk studies for deciding whether a wall is safe or not". It lasted 6 months with attempts on the establishment of techniques for investigating walls, the application of such techniques in an area study, back analysis of failed walls, and the formulation of relationships between proximity of water bearing services to walls and bulging failures.

Although nothing came out of the planned techniques for investigating walls, studies were nevertheless carried out on 65 walls in map areas 11SW-A & B which showed some bulging or cracking. These studies were similar to that of the Phase 1A but with a wider scope on more accurately measured quantities, sketches and sections as well as more photographs. The experience gained in this study was summarised in the notes as attached to Appendix A.

3. STRUCTURE OF MASONRY RETAINING WALLS

3.1 General

A general knowledge of the structure of masonry retaining walls is useful in two aspects. It is a prerequisite for the understanding of the structural behaviour of the walls, their failure mechanism and the deformations that precede their failure. Also, by knowing what to expect or look for in the walls, site investigations can be designed and interpreted more effectively.

In the absence of publications or other forms of records on the structure of masonry walls constructed in the early history of Hong Kong, their exact details are not known. A general inference however, can be made from photographs of sections of retaining walls occasionally exposed by demolition or wall failures.

To help interpretation of these photographs, a review on the structures of masonry retaining walls in other parts of the world was made to provide some knowledge on the probable components and structures of the walls. In all, three regional areas were investigated; United Kingdom, Japan and Korea as well as China. China should have the greatest influence on wall constructions in Hong Kong. This is because most of the early contractors in Hong Kong were immigrants from Wu Hua, a place in China renowned for it hard rock masonry works (Lo, 1971). The United Kingdom wall construction practice might have some influences in Hong Kong through the executions of construction works by British engineers for the armies and the businessmen. However, the degree of influences is not presently known. In modern Japan and Korea, masonry retaining walls similar to the ancient styles are still being built extensively. Their wall structures must therefore represent a form well proven by ages of experience and modern soil mechanics theories. Consequently, the structures of masonry retaining walls in those two areas are also examined even though the Japanese did not influence the local wall building practices during their very brief occupation of Hong Kong in the World War II.

3.2 Description of Masonry Retaining Walls

In the report on Phase 1A Re-appraisal Study on Cut Slopes and Retaining Walls, B & P classified masonry wall types in Hong Kong according to the nature of the front blocks, whether they are mortared or dry packed and whether "horizontal beams" (horizontal tie course) are used. A total of 7 types were identified as shown in Table 3.1. Photographs of each type of walls are shown in Appendix B.

The use of surface features to classify walls is a logical step. However, poorly defined distinctions between blocks of different categories has led to ambiguities in classifications of certain types of walls, noticeably, random-rubble and squared rubble walls. In B & P's system, there is little reference to the fitness of the masonry blocks at the beds and joints which has greater structural significance than whether the blocks are dressed or squared. Also, some of the walls described as mortared are actually pointed. Hence, this system is inadequate when comes to the correlation with structural behaviours of the walls. However, because this system has been in use extensively since 1977 and was also adopted in the field sheets of the Phase 1A report, no attempt is made to modify it. Instead, it is suggested that in the future, a retaining structure must be described by its wall type as well as details of its observable structural elements.

In the past, when geotechnical engineers described wall elements, they use terms they were familiar with even though they might not be suitable names in the trade of masonry works. Self made-up terms were also used. Consequently, their written descriptions were usually difficult to be interpreted by other engineers. Therefore, a glossary of terms commonly used in masonry works was prepared. This is based mainly on BS 5390 (BSI, 1976), BS 5628 : Part 1 (BSI, 1978) and the AREA's (American Railway Engineer's Association) specification on masonry works. It is presented in Appendix C.

3.3 Structure of Masonry Walls in Other Parts of the World

3.3.1 United Kingdom

Despite the large number of retaining walls constructed in the United Kingdom in the 19th Century, very little was published on their structure. Only a crude picture of the wall structures could be drawn from the information collected.

Anon (1845), in a sketch of a retaining wall used in an experiment on earth pressure, showed brickwork with either a Dutch bond or English cross bond. Burgoyne (1853) discussed full scale tests on 4 retaining walls composed of squared rubble brought to courses. In both cases, the blocks were dry packed or bedded with wet sand.

Jones (1979) described the structure of dry stone retaining walls commonly found in the Yorkshire region as

> "The stones used for the face were of medium/large size, carefully graded and placed by hand to fit; behind these, smaller flat stones were used, laid in horizontal planes grading back from the face."

A schematic picture of the wall by the same author is shown in Figure 3.1. It is a zoned structure markedly different from the solid masonry structures by Burgoyne and Hope. Because smaller size blocks could be used in the zoned structure, it would be cheaper than the solid walls necessary for military purpose. It should therefore be the more likely structure of walls incorporated in civil engineering works. Hart (1871) also mentioned this type of zoned wall structure. He suggested that good quality materials be used at the front part of the wall to take the higher compressive stresses. He warned against possible differential settlement between the zones and proposed the use of long headers to improve the integrity of the walls.

3.3.2 Japan and Korea

In modern Japan, masonry retaining walls are divided into dry packed walls and mortared walls. The structure of the dry packed walls is similar to that observed by Jones (1979) in Yorkshire, namely, a front zone of carefully laid, large size, well squared blocks carefully wedged in position, with small size granular material at the rear (Figure 3.2). In mortared walls, weak concrete mortar is used to bind the face blocks together. The remaining thickness of the walls behind the face blocks are made up of separate layers of concrete and granular materials.

For the dry packed walls, the Japanese use, at the face, blocks neatly squared at the front but with a rough tapering rear. Small angular stone pieces are placed between these stone blocks to keep them in position. For the mortared walls, the face blocks are of different qualities ranging from natural rubble to good fitting blocks similar to that for dry packed walls. Table 3.2 shows the shape and recommended dimensions for such well squared tapered blocks.

The Japanese masonry retaining walls have very low face angles between 63° to 73° and it is usual for high walls to have a concavely curved profile with flatter face angle at the toe. Figure 3.3 shows the recommended thickness of wall for $\theta = 15°$ to $35°$ and, δ (angle of friction at rear of wall) = θ (inclination of the wall).

Apart from the columnar arrangement of stone blocks usually found in other parts of the world, the Japanese also use an "arrow-feather" arrangement in which the beds and joints are arranged to incline to the horizontal (Figure 3.4). The advantage of such arrangement is uncertain at present.

In a paper on stability analysis of masonry retaining walls by Bishop's simplified method, Kim (1975) presented typical sections of masonry walls recommended by the Ministry of Construction, Republic of Korea (Figure 3.5). The wall structure is similar to the Japanese mortared wall though more generous in the thickness of granular material.

3.3.3 <u>China</u>

Although discussions on structures of Chinese walls and the Great Walls have been the subject of lengthy publications (e.g. Hommel, 1937; Needham et al, 1971), not much was said on the structure of masonry retaining walls. However, during the study, opportunities were made available by the Antiquity and Monument Section of the U.S.D. for an inspection of two Chinese forts in Tung Lung Island and Tung Chung. The Tung Lung Fort was constructed in the late 17th Century. It has collapsed extensively and has provided good exposures for the study of the wall structure.

There are two types of walls in the Tung Lung Fort; the thin rubble boundary wall and the broad platform wall. The latter consists of compacted soil fill retained by rubble retaining walls on both sides. The retaining walls have a core of small size, random rubble bound by a face layer of well squared blocks and a rear layer of round, unworked blocks (Figure 3.6, Plate 3.1).

Similar cored structures were observed in the Tung Chung Fort wall, although no exposed sections were available for more detailed study.

A type of masonry wall that can be seen in old works in China consists of strips of well squared stones laid criss-cross each other in the "Chinese box-bond" pattern

(Figure 3.7). These walls were usually used in forts, city gates and foundation platforms of expensive buildings because of their higher ability to resist damage from flooding and mechanical attacks. Plate 3.2 shows such a wall dating 400 years in Huashan, immediately south of the Yellow River, China.

3.4 Structure of Masonry Retaining Walls in Hong Kong

3.4.1 Tied Face Walls

These are Chinese box-bonded masonry walls (Figure 3.7) adopted to retain cuttings in Hong Kong. They were most popular in Hong Kong in the 1840's. Typically, the front layer of blocks are very well squared and dressed to good fitting on 5 faces, with the face adjacent to the void very rough and irregular. They are normally bonded by very thin layers of good quality lime/sand mortar, although some dry packed walls can also be found. The rear layer of blocks in contact with soil are usually not much squared and are dry packed over each other. The ties are always well squared at the front to fit neatly into the front blocks although the rear portion may be as rough as the rear blocks. Typically, the front blocks have sizes of 1.2 m x 0.3 m x 0.3 m high whilst the ties average 1.0 m x 0.15 m x 0.3 m high (Figure 3.8).

The tied face walls are mostly found on sites immediately south of the original shoreline of the Island which were development in the first half decade of the history of Hong Kong. It soon lost its popularity possibly because of the high costs of cutting, dressing the granite strips and transportation from the quarry. They were later used mainly for foundation platforms of prestigious houses and walls where very smooth surfaces were desirable. Some examples of these uses can be found in Castle Road at the junction of Caine Road, Plates 3.3, 3.4.

A variation to the usual grillage structure of tied face walls was the use of stone strips to fill up the wall. This was observed in walls number 11SW-A/R457 & R458 (Walls W6, W7 of GCB's retaining wall inspection cards). Such walls are extremely strong and rigid for taking earth pressure.

3.4.2 Stone Rubble Walls

This category of wall covers the wide range of wall types described as random rubble, squared rubble and dressed block walls by B & P in the Phase 1A study. They usually have a cored structure similar to that observed in Tung Lung Fort i.e. each wall comprises a front layer, a rear layer, and a core (Figure 3.6). The best of the materials, in terms of the degree of squaring, dressing and strength (usually associated with the freshness of the rock) are used in the front layer. Less squared blocks, with shapes varying from cuboid to platy, are stacked at the rear to form a straight rear plane. The space between the two is infilled with core materials that may range from angular gravel to boulders of different degree of roundness and different sizes. Some of the core materials can be as large in size as the side blocks.

This wall structure can perhaps be better perceived if one can imagine the manner in which they were constructed. Due to the absence of hoisting machines, construction of the whole wall prior to backfilling would be very expensive and tedious. Consequently, the walls were constructed slightly ahead of backfilling behind the wall. The workmen first prepared the wall foundation. They then laid a few courses of the front blocks and the rear blocks. The best materials were used for the front blocks because they were going to be the exposed face and would have to stand the full height of the wall unsupported. As for the rear courses, anything that could be stacked stably on one another to the height of a few courses could be used. The spaced between the face layer and the rear layer of blocks would then be infilled with any material at hand, including stone chips from the working of the side blocks, natural and building debris. After infilling the core, the backfill were then brought up by tipping soft materials carried by baskets or wheel barrows. After compaction of the backfill, if any, the whole construction cycle was repeated till the completion of the wall. This method of wall construction is summarised in Figure 3.9. Sometimes, instead of retaining the core materials by the rear blocks, the materials were allowed to spread out partly on to the previous surface of backfilling. The resulting wall, after completion, has a saw-teeth rear profile (Figure 3.10, Plate 3.5).

For squared rubble and dressed block walls, the front blocks were usually laid on lime/ soil beds. There was a lime industry in the coastal parts of the New Territories well developed since the 18th century to supply lime for construction purposes (Yim, 1981). Tests on bedding materials recovered from walls 11SW-A/R333 & R354 showed that the lime contents averages 6% by weight. The front blocks of random rubble walls were frequently dry packed. All these stone rubble walls might or might not be pointed with a lime/sand mortar. The function of the pointing was probably to avoid vegetation growth on the wall surfaces.

There are a number of variations on this general scheme. Lime-stabilised soil might be used to bind the core materials. In certain cases, it may completely replace the core rubble and became stabilised soil walls with stone facings. Walls of this type can be found in the region of Caine Lane and Ladder Street adjacent to Caine Road.

Sometimes, granite strips of rectangular section were inserted regularly into the face of a rubble wall to act as headers (Figure 3.11). The lengths of these headers are not known. They usually distinguish themselves from other face blocks by their neatly squared rectangular ends (Plate 3.6). This type of wall should be distinguished from others by a prefix "Tied", e.g. tied squared rubble wall. This type of wall is most commonly found in the Mid-levels along Caine Road and Robinson Road near the University of Hong Kong.

A large number of stone rubble retaining walls in Hong Kong were constructed with horizontal tie course at regular vertical intervals. These are described by B & P as walls with "horizontal beams". The tie course may either be strong lime-stabilised soil or concrete of a wide range of strength.

There are two types of wall that are not specially classified by B & P but may need special considerations. The first type is the recent masonry retaining walls. They are usually constructed of well squared, poorly dressed rectangular blocks. The size of these blocks is much smaller than those used in old walls. This can be used as a means of identification (Plate 3.7). This type of walls was usually constructed by masons who did not know much about earth pressure and did not have the same experiences on masonry retaining structures as the earlier masons had. The resulting walls can be very thin in section and may fail catastrophically under unfavourable conditions.

The second type was found among those classified by B & P as random rubble walls. They have face blocks of very irregular sizes and shapes laid in a completely uncoursed manner. The joint widths are generally large. Some of the blocks are substantially decomposed and do not possess much strength (Plate 3.8). On close inspection, some of the blocks are found to have been formed simply by splitting large boulders into halves. The resulting blocks have rounded rears and shallow thickness when compared with their height. A face layer consists of these blocks is not very stable even in the absence of stresses induced by earth pressure. Walls of this type are abundant along Caine Road, Bonham Road and Robinson Road. This type of wall resembles a rough-picked polygonal wall in appearance and should be referred to by this name in future studies on masonry retaining walls.

3.4.3 Stone Pitchings

A stone pitching is a layer of stone blocks laid on formed slopes to prevent erosion and infiltration. A thin layer of concrete is usually provided as backings to the stone blocks. It is characterized by gentler surface angles and thin layer thickness. The surface angles are between 35° to 65°. The former is mainly for fill slopes while the latter is for cut slopes. The usual thickness is 300 mm. Figure 3.12 shows an example of a stone pitching.

This is very similar to the Japanese masonry retaining walls (Figure 3.2). The possible difference is that the stone pitchings are sometimes not provided with any granular material layer. Hence, the stone pitchings should also be treated as a masonry retaining wall and should be checked for static and structural stabilities.

4. CASE STUDIES ON MASONRY RETAINING WALL FAILURES

4.1 General

In this Chapter case histories and observations on wall failures in Hong Kong and other parts of the world are examined. Some common features of wall failures in Hong Kong are identified. This would enable engineers to avoid conditions unfavourable to masonry retaining wall stabilities in the future. Observations on signs of distress preceding wall failures have been collected from the case studies and they are useful for interpreting the results of stability analyses in Chapters 5 & 6, as well as to provide criteria for defining and monitoring dangerous walls.

4.2 Case Histories of Wall Failures in Hong Kong

In the study on old masonry retaining walls carried out by GCB in 1979, 41 cases of wall failures or walls showing such distress as to demand immediate remedial works were recorded. In the present study, it was intended to examine these cases further for details on signs of distress prior to and the damages caused by the failures. Not all the files listed were read because many could not be obtained in the short period of time available. A few other cases not mentioned in GCB's list were also examined where information was available. Table 4.1 shows the list of the incidents and the corresponding files that have been examined. Among these 16 cases, 6 contain so little information as to make them not worth discussing.

Details of each of the 10 cases considered are presented in Appendix D. The particulars of each failure are abstracted in Table 4.2. The location of the failures are shown in Figure 4.1. From these cases, some common features can be seen.

(a) A line representing the boundary of the Mid-level Development Restriction Area recommended in 1979 is also shown in Figure 4.1. It was fixed by terrain evaluation and marks the positions of changes in ground gradient from above to below 15° in the area covered by colluvial deposits. The ground south of this line was classified as geotechnically not suitable for development. It would therefore be expected that most of the failures should be located there. Instead, they are found on the line itself. A possible explanation to this comes from the findings of Lai (1980), Huntley and Randall (1981) on the different episodes of colluvium in Hong Kong. The older deposits are denser, more decomposed and have gentler ground surfaces as a result of prolonged period of degradation. The young deposits, on the other hand, are loose, weak and stand near to the angle of repose. The usual undesirable properties regarded as typical of colluvium are actually those of the younger colluvial deposits. Because of its characteristically steep ground surface angles, its boundary with the older ground surface is likely to be marked by an abrupt change in ground profile. At these locations, the weak seam of the previous top soil layer is near to the ground surface. The perched groundwater table is also highest and nearest to the ground surface so that the whole geotechnical environment is more unfavourable to ground stability.

(b) Of the 10 collapsed walls documented, two are tied face walls and four are random rubble walls. Three of remaining four are stabilised soil walls. No masonry retaining walls with tied courses were involved. This may either be a matter of coincidence or a result of the greater structural efficiency of masonry walls with tie courses (horizontal beams) as is discussed in Chapter 6.

(c) Over half of the failures were triggered off by works carried out in adjacent areas. The crest platform was being repaved in Case 1. There were trench works at the crest in cases 2, 3, 7 and 8. These trenches, either open or loosely backfilled, permitted fast infiltration of rainfall which caused local built up of groundwater level. Case 7, however, is a bit ambiguous. It is not known whether the trench work affected the stability of the wall or that the loose backfill to the trench caused sideward movement of the subgrade which in turn caused the longitudinal crack on the road. The crack caught the attention of the inspection engineer who then discovered that the wall was bulged. Actually, cases of longitudinal cracks caused by uncompacted backfill to trenches are not uncommon.

There is little doubt that driving of sheetpiles was the immediate cause of failure of the wall at Wing Wa Terrace (Case 9). The operation caused falling of pointings, cracking at the crest and bulging of the wall at the location that later collapsed. There are a number of possible ways that the pile driving might have upset the stability of the wall. The vibration might have loosened the wall structure, or broken water carrying utilities at the crest platform resulting in a local rise in water table. The latter, however, was less likely because the horizontal drains were functioning and the contractor did not observe increased flows from them. Random rubble walls were sometimes found on a spread footing of granular material. The sheetpiles, being driven too close to the toe of the wall, might have damaged this granular layer and led to the collapse of the wall.

There is a close relationship between the forward movements of the wall at Circular Path and the construction activities at the toe platform. Demolition of the toe buildings caused continuous opening of cracks at the crest which was stopped only by the construction of a supporting embankment at the toe.

Although the failure of the masonry retaining wall at 1, May Road (case 4) occurred at a time when the building at the crest was being demolished, it was not caused by the demolition work. Instead, the immediate reason of the collapse was weakening of the wall foundation by a slip at the toe slope of the wall.

(d) The walls might deform appreciably before failure. Among the 10 cases considered, 5 failed without recorded signs. Out of these 5 cases, 3 occurred in early morning or mid-night so that wall movements prior to failure might have escaped the attentions of the public.

For the remaining 5 cases, the walls bulged and/or cracks opened at the crest before failures occurred. The bulges were most noticeable at mid-height of the wall and might exist for some period of time before the wall collapsed. The crest cracks were sub-parallel to the wall and extended for great distance. Their widths were of the order of 20 mm.

(e) Five of the failed walls were with high groundwater table (Cases 3, 4, 5, 6, 9). Groundwater level at the wall at Po Hing Fong was not high but the quantity of groundwater was enough to support continuous flow to a spring in one location at the toe of the wall.

(f) Out of the 10 cases, eight caused partial or complete closures of the road at the crest or the toe of the retaining walls, some for as long as 10 months. Three of the failures caused severe structural damages to the buildings at the toe of the wall while another one was saved from doing so only by an earth bund down slope of the wall which retained the debris. Three other walls failed into demolished sites. They might otherwise have caused similar damages to the toe buildings. Of the remaining 3 cases, two did not actually fail but were regarded as in a state of marginal stability. From the limited number of cases, it appears that the consequence of retaining wall failures is generally severe.

(g) The 15 day and 24 hour cumulative rainfall quantities for each of the 10 cases at the time of their instabilities are shown in Table 4.3. They are also plotted on the predictive chart in Figure 4.2. This chart is based on a similar one in Lumb (1975) in which he related landslip potentials with the 24 hour and the previous 15 day rainfalls in a particular day.

From Figure 4.2, two of the failures occurred on days that severe or disastrous landslips were predicted while another two occurred on the following days after such rainfall conditions. The time lag might have been caused by the slower response of groundwater table to heavy rainfall. Of the remaining six events that occurred on days with minor to isolated landslip potentials, construction activities in the vicinities of the walls were the immediate cause of instability of five of them (cases no. 3, 6, 7, 8, 9).

4.3 Observations on Wall Failures in the United Kingdom

In 1845, Hope (Lt) of the Royal Engineer's Establishment carried out a series of tests on retaining wall designs. Details of the experiment are described in Anon (1845). Hope ordered 10 ft high walls to be constructed to different geometries with bricks laid on wet sand. He then backfilled the walls and monitored the accompanied deformation. In particular, he kept records of the deformation profile of a 10 ft x 1 ft 11 inch rectangular wall immediately before failure. The profile is plotted on Figure 4.3. A triangular piece of the brickwork remained after the wall toppled forward. This was sketched and was accompanied by a note that the sketch was approximate. It is also reproduced in Figure 4.3.

In 1853, Burgoyne (Lt) carried out another series of experiments to find the performance of different retaining wall geometries (Burgoyne, 1853). Four 20 ft high walls each constructed of equal volumes of granite blocks to different geometries were loaded by backfilling with wet earth in a wet weather. The dimensions of the walls are shown in Figure 4.4.

Burgoyne kept close observations of the deformation of the walls as they were backfilled. Wall A, which had both the face and the rear leaning at 5 on 1 backward, did not deform noticeably when fully backfilled. The sloping wall which had a vertical rear and a face slope of 5 on 1, tilted forward by 2½ inches when fully backfilled. Fissures were also observed on the face of the wall. Before failure, the counter slope wall overhung 10 inches and 5 inches at the crest and quarter height from the toe respectively. When falling, it burst out at about 5.5 ft from its base, with two-third of the wall from the top downwards kept in an upright position until it reached and was crushed on the ground.

When the rectangular wall D reached its limiting equilibrium, it overhung 1 ft, with a convexity on the face measured more than 4 inches. It then tilted forward gradually for an additional 6 inches before it toppled forward in a unit. Based on the above descriptions, the deformation profiles were drawn and shown in Figure 4.3. Sketches of the form of the walls at falling are reproduced in Figure 4.4.

In 1874, Constable Casimar constructed 16 in. high model retaining walls with wood blocks and observed their behaviours under granular backfills. He noted that before the walls failed, they bulged with centres of curvature approximately at mid-height of the wall. After the walls failed, triangular pieces of wall similar to that sketched by Lt. Hope could be observed to remain.

In an article on practices in the design of earth retaining structures, Jones (1979) discussed the failure of Victorian stone retaining walls. He observed that there were increasing number of failures. This was attributed in part to the deterioration of the stones with time and also a tendency for the walls to change shape. He said that these failures were unpredictable and might be preceded by a long period in which the wall retains a bowed shape. Figure 4.5 is a schematic diagram on the suggested mode of failure according to Jones (1979). Plates 4.1, 4.2 also from the same author, show some features of these failures.

5. STATIC AND FOUNDATION INSTABILITY OF MASONRY RETAINING WALLS

5.1 General

When a retaining wall is too thin in section, static equilibrium cannot be maintained between the earth pressure and the stabilising forces so that the wall moves forward. The movement may either cause a decrease in earth pressure great enough to allow the equilibrium to be re-established or it will continue to such an extent that the wall can no longer retain the earth. In the latter case, the wall is regarded as having failed statically. Such failures are preceded by the formation and widening of cracks at the crest platform, together with overhanging of the crest of the wall. From the observations in Hope's and Burgoyne's experiments and in some of the failure cases discussed in section 4.2.2, masonry retaining walls are liable to these modes of failure.

In the following sections, the distribution of earth pressure behind old masonry walls is first considered. This is followed by discussions on the influence of different factors on the static instability of retaining walls through the comparison of the results of a series of generalised analyses. For uniformity of results, the parameters used in these analyses are similar to those used in GCB's Study on masonry retaining walls. In particular, the 'no tension at base' condition is also taken as the criterion for stability.

Apart from instability due to insufficient wall section, a retaining wall may also fail as a result of overstressing of the foundation. This is especially likely to occur when the wall stands on the crest of a wet slope. In the last part of this Chapter, the effects of ground conditions on retaining wall stability are investigated.

The aim of this whole series of analyses is to provide the investigation engineers with a sense of the relative importance of the different parameters that can be collected in the inspection of retaining walls.

5.2 Earth Pressure on Old Masonry Retaining Wall

The static stability of masonry retaining walls can be evaluated by treating the wall as an integral body and then consider the criteria of static equilibrium. The accuracy of this process depends very much on the knowledge on the earth pressure on the wall, including its magnitude and distribution. There are a number of approaches in conventional soil mechanics for calculations of the earth pressure pattern from ground geometry and strength parameters. There methods are discussed in details in most books on soil mechanics and foundation engineering e.g. Huntington (1957), Shelton et al (1980). Triangular pressure distributions are assumed in such methods.

However, in recent experiments and measurements on earth pressure behind retaining walls, pressure patterns very much different from those predicted by the conventional methods were recorded. On close examinations, the conventional approaches are found to be applicable only to a particular situation, namely, that the backfills are deposited naturally and that wall movements do not occur till the completion of the backfilling process. Any other changes in the methods of construction, such as the use of compaction plants, and deformation of the wall as backfilling proceeds, will modify the pressure patterns on the wall.

Figure 5.1 shows some of the probable pressure patterns.

In Chapter 3, it was mentioned that most of the old masonry retaining walls in Hong Kong were backfilled in layers without the employment of heavy compaction plants. The compaction induced pressure is therefore small. Also, dry soil was usually used for backfilling because the surfaces of wet backfilling material would not be strong enough to support the construction activities. The strength of dry soil would be higher than that usually adopted for earth pressure evaluation which corresponds to the saturated strength. As a result of these two factors the overall pressure on an old masonry wall is probably smaller than that calculated by the conventional approach (Figure 5.2).

Sometimes, the backfill behind a completed wall may be saturated, either by infiltration from unpaved surface or leaking pipes, or due to rise in groundwater table. When saturated, the soil strength decreases, this is accompanied by increases in earth pressures on the wall. If the walls are supported by structures at the toe, such as houses, the increase in pressure will be taken by the toe structures and an at-rest state exists. For unsupported walls, the wall would yield slightly under the increased pressure. In doing so, a condition necessary for the validity of the conventional earth pressure theories is achieved. This is the movement of the wall as a unit to mobilise the internal resistance of the soil mass. The resulting earth pressure is the active pressure. In other words, conventional earth pressure theories can be used to calculate the highest active pressure to which an old masonry wall may be subject to.

5.3 Factors Affecting Static Stability of Masonry Retaining Walls

In all, 4 factors were considered. These are the strength of the retaining soil, the geometry of the retaining wall, the ground slope at the crest and the groundwater behind the wall. The results of sensitivity analyses for each of the 4 parameters are expressed as the maximum height/base width ratio of a wall for no tension to be developed at the base in the particular ground conditions considered. They are presented in the graphs in Figures 5.3 & 5.4. For retaining walls under unfavourable combination of the parameters, the H/B ratios are smaller implying that a thicker wall is required. The degree of influence of a certain parameter can therefore be expressed as the percentage change in the required wall thickness when compared to a standard wall. The retaining wall back analysed by GCB in its study on old masonry retaining walls is again used as the standard. The design parameters of this wall are shown in Table 5.1. The usual limits of the parameters and the corresponding changes in the required wall thickness are shown in Table 5.2.

The effect of variations in the strength of the retained soil is unexpectedly small on the stability of the wall. With other parameters unchanged, variations of soil strengths within the usual range cause less than 10% differences in the permissible height/base-width ratio. The most significant factor is groundwater. When the groundwater is at full height of the wall, the wall section is needed to be 170% thicker than that of the normal wall. However, it is very seldom that seepage on masonry retaining walls is observed to that height. More often, the seepage is observed at half height of the wall in which case a 25% increase in wall thickness is enough to maintain stability of the wall.

The influence of crest slope angle on the required wall thickness is also significant. For a crest slope of 30°, the wall needs to be 20% thicker in order to remain stable. If the slope angle increases to the limiting angle of 39°, the wall would need to be 50% thicker.

For the normal range of front face slope angle of retaining walls, the variation of required wall thickness with the face angle is very small. Changes in the rear slope angle, however, cause great differences in the required width of a retaining wall. When the rear face of a wall is countersloped forward by 10°, (i.e. near face angle = 100°) the wall would need to be 20% thicker than the standard wall. Further increases in the countersloping angle do not cause a decrease in the permissible H/B ratio as a result of the stabilising effect of the downward component of the earth pressure. If the rear face of the wall leans towards the retained soil, great improvements in stability are achieved. A wall can be 62% thinner for a 10° leaning (i.e. rear face angle = 80°). This is the reason why Japanese retaining walls are stable despite the thin sections. Similarly, many of the stone pitchings in Hong Kong may be providing a significant stabilisation effect to the slopes they covered. It is of course not possible to know the rear face angle of a wall from surface inspection. We should however keep in mind that walls with a leaning rear usually have gentle sloping front face.

One component of the shear strength parameters that has not been considered in the above discussions is that of cohesion. Figure 5.5 is a graph showing the free standing height of a vertical cut against cohesion of the soil. It can be seen from the figure that with a cohesion of 10 kPa, a 4.5 m high wall can stand satisfactorily even if it is a nominal face layer of blocks. Cohesion is normally found in insitu decomposed materials and to a smaller amount in unsaturated backfills as a result of soil suction. This may be the reason why some old masonry retaining walls stand with a relatively thin wall section. The cohesion component, however, may be destroyed by saturation of the soil. It should not be relied on if the soil behind a retaining wall is liable to be saturated.

The amount of friction at the back of retaining wall depends on the downward movement of the soil with respect to the wall. Under normal circumstances, δ, the frictional angle between the wall and the retained fill, ranges between $\frac{1}{2}\phi$ and $\frac{2}{3}\phi$. If the wall settles, the δ value decreases. In the extreme case where the wall sinks more than the soil, δ can be negative. The earth pressure is higher in such case so that the stability of an otherwise adequate wall may be endangered. From Figure 5.3(a), the permissible H/B ratio for $\delta = 0$ and $\phi = 39°$ is 2.5, i.e. it has to be 20% wider than a normal wall.

5.4 Factors Affecting Foundation Stability

The bearing capacity of a soil foundation depends on the soil strength, groundwater location, the applied load characteristics, the buried depth of the foundation, its distance from a slope and the gradient of the slope. For a retaining wall, the load characteristics are governed by the wall configuration, height and the properties of the retained soil. The main effect of groundwater on the ground properties is to reduce soil density and its strength. But for soil strength parameters derived from normal laboratory testing on saturated samples, the effect of saturation has already been accounted for.

In this section, sensitivity analyses are carried out on all the factors with the exception of the load characteristics which is a constant if a single wall configuration is considered. Again, the wall configuration used in GCB's back analysis in the Study on Old Masonry Retaining Walls is adopted. The Vesic's equation for bearing capacity is used as recommended by Shelton et al (1980). The results of the analyses are expressed as the critical toe slope angle above which the ultimate failure of the wall foundation will occur. They are presented in the graphs in Figure 5.6. For favourable ground conditions, the critical toe slope angle is larger. A method of measuring the improvement due to changes in one of the ground parameters is to compare the resulting critical toe slope angle with that associated with a generalised ground condition. The comparison can be expressed as a percentage change in critical toe slope angle when a parameter is at its usual limit of value. The generalised set of ground parameters are shown in Table 5.3. The usual limit of values and the corresponding percentage changes in the critical toe slope angles are presented in Table 5.4.

For the particular foundation configuration considered, the foundation stability is independent of the height of the wall. Contrary to the minor influence of soil strength to the static stability of retaining walls, the influence of soil strength on foundation stability is very large especially in the case of high groundwater table. With a frictional soil shear strength of 35°, which is not uncommon in colluvial deposits, the maximum toe slope angle is reduced to 8° for high groundwater situation. It is thus not surprising to see the retaining wall at 1, May Road failed in its foundation (Case 3 in Chapter 4).

When the distance of a retaining wall from the crest of the slope increases, the foundation stability increases quickly so that at a separation of 2.5 m, the presence of a toe slope does not affect the bearing capacity of the foundation of a 10 m high wall. The improvements in the bearing capacity by burying the wall foundation is small. The maximum toe slope angle is only increased by 25% for a 1 m embedded depth; a value not normally provided in old masonry retaining walls. A buried foundation, however, is less likely to be undermined by a surface slip of the toe slope.

5.5 Relative Importance of Factors Affecting Static Stability of Retaining Walls

From Sections 5.3 and 5.4, it can be seen that the relative importance of factors affecting retaining wall stability comes in the order of groundwater level, crest slope of the retained soil, and at the least, the soil strength parameter if it is a cohesionless material. Special attention must be paid to walls with seepage over half of the height of the wall or with crest slopes steeper than 30°. The surface geometry of a retaining wall does not affect its static stability much. However, walls with gently sloping fronts are usually associated with backward leaning rear faces. The leaning wall is a more efficient form of retaining structure.

A statically stable retaining wall standing on a slope with gradient larger than a critical value is liable to foundation failure. The critical gradient depends on the distance of the toe of the wall from the crest of the slope, the soil shear strength, the groundwater location and the depth of burial of the wall foundation, in decreasing degree of importance. In the worst case of submerged cohesionless soil with an angle of internal friction of 35°, the critical toe slope angle can be as low as 8° if the wall stands on the edge of the slope.

6. STRUCTURAL STABILITY OF MASONRY RETAINING WALLS

6.1 General

In the draft Guide on Retaining Wall Design (Shelton et al, 1980), it is specified that apart from static instability, a retaining wall must also be checked for the possibility of structural failure. In the design of reinforced concrete retaining walls, this is always done to estimate the amounts of reinforcement and concrete required. The stresses that can be induced in a mass concrete retaining wall are usually small when compared with the material strength. Consequently, the procedures for checking the structural adequacy are omitted in the design of mass concrete walls. However, for masonry retaining walls comprising blocks loosely bonded together, the structural strength may be exceeded. Therefore, when evaluating the stabilities of masonry retaining walls, the likelihood of structural failure must also be examined.

In this Chapter, the strengths of masonry are first studied, followed by calculations on the stresses in a gravity retaining wall. They are then compared with each other to identify possible modes of structural instability. Based on this comparison, failure mechanisms are put forward taking into account the 'more' commonly observed deformation of masonry retaining walls prior to failure.

6.2 Strength of Masonry

6.2.1 Sources of Information

When subjected to a combination of stresses, a material may fail in compression, tension, shear or local buckling. The likelihood of material failure under a specific stress condition depends on the strength of the material and the magnitude of the applied stresses. To find the permissible strengths of masonry, the Chinese, British and the American building standards for masonry works are reviewed. The relevant tables and clauses are summarised in Appendix E and are briefly discussed below. For a particular type of masonry whose strength is not discussed in these building codes, strength criteria well established in geotechnical and structural engineering are employed to give rough estimations of their behaviour.

In all, the Chinese Building Standard (1973) provides the most comprehensive information. Its content covers masonry composed of wide ranges of block strength, mortar strength, and block sizes and shapes. From the American source available (Cross, 1976), only the compressive strength of masonry works is described. On the other hand, the British Standard is mainly for brick works and other artificial blocks. It is very conservative when stone works are involved.

6.2.2 Compressive Strength

The compressive strength of masonry depends on the intrinsic strength of the building blocks and the mortar, as well as the shape and size of the blocks. The Chinese Building Standard requires that the compressive strength of random rubble walls is to be between 10% to 16% of that of ashlar walls of the same material. The lower percentages are associated with lower mortar strengths. The British and U.S. standards specify values of 75% and 16% respectively for the same difference in block shapes. It appears that the permitted strength of random rubble masonry is too high in the British Standard.

When compared with the other two building standards, the American building codes are very conservative in the allowable compressive strength. Their values are always below half of those allowed for similar materials in the Chinese and British standards.

The British Standard agrees well with that of the Chinese on the strength of masonry with standard format bricks. For walls with stone blocks, strength values for artificial blocks of similar dimensions are recommended in the British Standard and they are always smaller than the strength values for stone block masonry in the Chinese Standard. The British Standard also mentioned that when large, carefully shaped natural stones are laid with relatively thin joints, values higher than the tabulated strengths can be used. That is, the higher strength value in the Chinese code is more reasonable. Therefore, the Chinese Building Standard is used in the present study to provide some guidance on the compressive strength of masonry.

To deal with the effect of block shapes, the Chinese Building Standard specifies four classes of stone blocks, each with the following features :

(a) Ashlar : Blocks finely dressed to very regular shapes and with width and height not less than the smaller of 200 mm or 1/3 of the length. The surface irregularities are not to exceed 2 mm.

(b) Coarse ashlar : Blocks similar to ashlar but with surface irregularities not exceeding 20 mm.

(c) Squared rubble : Blocks that are squared and picked to approximate cuboids. They are usually slightly dressed or undressed and with height not less than 200 mm.

(d) Random rubble : Stone blocks of irregular shapes with height not less than 150 mm.

When the above descriptions are compared with the face blocks of walls assigned as random rubble walls in the Phase 1A study, many of them are actually squared rubble. Therefore, the strength of masonry walls must be assigned from the inspection of the block conditions instead of from the wall type designations.

Table 6.1 shows the compressive strengths of walls composed of blocks of different shapes and bonded by mortar with a range of strengths. The intact strength of the blocks is taken as 100 MPa. An average block height of 350 mm is adopted in the estimation.

For most old masonry retaining walls in Hong Kong, this Table only applies to the face layer of blocks. The behaviour of the core materials behind the face layer is much more complicated. It depends on the sizes and shapes of the core material as well as the manner in which it was placed. If randomly dumped in position, the core material would behave as a granular material. A lateral pressure would be necessary to maintain the equilibrium of the core against the vertical pressure. For materials sizes ranging from gravel to 150 mm diameter boulders, ϕ values of 35° to 70° are quoted (e.g. Patwardhan et al, 1970). The resulting lateral pressure/vertical stress ratio varies from 0.27 to 0.03.

6.2.3 Tensile Strength

Neither the British nor the Chinese building standards have mentioned any tensile strength of dry packed masonry. Since the tensile strength of a mortared joint comes mainly from the cohesion of the mortar, the tensile resistance across an unbonded surface of a dry packed masonry should approach zero. However, in the presence of headers through the failure surface, some tensile strength exists. The magnitude of this strength depends on the tensile strength and the cross-sectional area of the headers, the embedded lengths and the friction between the headers and the masonry blocks (Figure 6.1).

6.2.4 Shear Strength

The British Standard specifies Mohr-Coulomb failure criteria for shearing along the joints of masonry. This is better than those tabulated in the Chinese Building Standard in which the shear resistance is regarded as independent of the stresses normal to the shear plane. According to the British Standard, the cohesion component of shear resistance varies from 0.15 MPa to 0.35 MPa depending on the strength of mortar used. A uniform value of $\mu = 0.6$, corresponding to a frictional angle of 31°, is to be adopted independent of the strength of mortar used to join the blocks. However, these are characteristic strength as is discussed in Appendix E. To convert them into allowable stresses, they have to be divided by an equivalent load factors with an approximate value of 4.2. The resulting shear strength is extremely small, with an equivalent frictional angle of 9°. This is too low. Even if the mortared beds are treated as rock joints infilled with strengthless sand mixtures, a minimum friction angle of 30° is expected ($\mu = 0.6$). Hence, it is more reasonable to apply the load factor of 4.2 to the cohesion component only. Table 6.2 shows the expected shear strength according to the modified criteria at different magnitudes of compression across the shear surface. The tabulated values should only be treated as very rough estimates.

The failure criteria of dry packed masonry is not specified in either specification. This, however, is likely to be similar to that of rough rock joints. A zero cohesion and a 45° angle of friction may be appropriate in such case.

The above strength criteria are applicable to shearing along a planar surface. If the stone blocks are so bonded that shearing is only possible along an irregular surface, the shear strength will be very different. This difference in strength is similar to that between a smooth rock joint and a rough one. When a joint with smooth side walls is sheared, the frictional resistance depends solely on the nature of material in contact. When shearing is along a non-planar surface, the surface irregularities introduce an additional component of frictional resistance. This was explained by Patton (1966) as the additional force necessary for moving against inclined planes formed by inclined contacts across the irregular surfaces (Figure 6.2).

Movements across the inclined contact points also cause dilatancy of the joint perpendicular to the direction of movement. If the joint is restrained against lateral movements a large lateral pressure as well as higher frictional resistance will be induced (Goodman, 1976). There is a limiting value of inclination (i_c) between the surface contacts above which sliding along the inclined contact is not statically possible. If the applied shear force is large enough, the material at the location of the steeply inclined contacts will be sheared off. This introduces a cohesion component to the strength of the irregular joint, Figure 6.3.

The similar behaviour of a rock joint and a rockfill with shearing through an irregular surface is discussed by Barton & Kjaernsli (1981). Patwardhan et al (1970) reported results of large shear box tests on shearing along irregular surfaces in a bouldery material. He recorded frictional angles as high as 70° accompanied by dilatancy of 50% to 80% of the average particle sizes.

There may also be some cohesive resistance against shearing through a bonded masonry, with mechanism similar to that of the rough joints (See Figure 6.4). However, instead of having to shear through the intact material, the steep contact points in a masonry can be surmounted if the shear force is large enough to cause re-orientation of the blocks to contact at gentler angles. The resulting apparent cohesion depends on the sizes, shapes and packing of the blocks. Because vibrations facilitate re-orientation of blocks, this cohesion is liable to be reduced by vibration.

6.3 Stresses in Stone Rubble Retaining Wall

The next step to the evaluation of the structural stability of a retaining wall is to find the magnitude of the stresses in it. For a masonry structure, an accurate stress distribution analysis would require detail knowledge of the bonding pattern of the building blocks as well as the various mechanical properties of them. The set of equations required to account for all these factors will be very difficult to be set up and solved. Such tedious solution is hardly worthwhile because then, every solution will be a particular solution. Therefore, as a first approximation for a general case, the assumptions that the wall materials are homogeneous, isotropic and elastic are adopted. These are of course not true. However, if the masonry block sizes are very much smaller than the wall dimensions, and if no tension is developed, these assumptions are more acceptable than would otherwise be regarded. Because in such cases, a macroscopic uniformity exists.

A package computer programme STRAND 2 developed by HECB (Highways Engineering Computer Branch, Dept of Environment) allows stress analyses to be carried out by the finite element technique. This is on hire to the Highways Office of the Public Works Department. The programme was primarily written for bridge deck analysis but is so generalised that it can be used for analysis of retaining walls. Early in this present study attempts were made to use it. However, these were unsuccessful apparently due to certain flaws in the programme because book examples were input into the programme for trial and the output were far from the expected results.

Analytical methods were then attempted. Differential equations were set up from stress equilibrium conditions. The equations were then solved for the simple boundary conditions of a rectangular wall with triangular pressure distributions. The mathematical solution is presented in Appendix F. A programmable calculator was employed to do the calculations. Both the analytical solution and the programme have been partly tested against hand calculation using graphical methods. Two loading cases were considered. The material parameters and the corresponding factors of safety against static instability are shown in Table 6.3. These parameters were selected to conform with those use in GCB's back analyses.

For each wall, the analytical results are presented in contour plots of $\sigma_1/0.1H$, $\sigma_3/0.1H$, principal stress trajectories, $\sigma_x/0.1H$, $\sigma_y/0.1H$, $\tau/0.1H$, as well as factors of safety against sliding in the horizontal, vertical directions and the direction of maximum shear stresses (See Figure 6.5 to Figure 6.10). The 0.1H terms are used to normalise and to give higher values for contour plotting. The terms used are defined as

- σ_1 major principal stress
- σ_3 minor principal stress
- σ_x horizontal direct stress
- σ_y vertical direct stress
- τ shear stress in the horizontal and vertical directions
- H height of the wall

The sign conventions are shown in the figures.

For masonry walls, there is a preferred shear plane in the horizontal direction through the beds. In some cases, such as at the interfaces between the face layer, the core and the rear layer, shear in a vertical direction is possible. These are the reasons why factors of safety against sliding in these two directions are calculated and presented. Sliding in the direction of maximum shear is only possible for stabilised soil walls. For masonry wall, sliding in this direction would involve shearing through the intact stone blocks which is not very likely. The factors of safety against sliding in the director of maximum shear are expressed in the figures as multiples of 10H/S where S is the shear strength of stabilised soil fill.

6.4 Possible Modes of Structural Instability of Stone Rubble Retaining Walls

6.4.1 Compressive Failure

The compressive stresses in a gravity retaining wall are shown in Figures 6.5(a) and 6.8(a). The maximum stresses that may act on a wall with adequate factor of safety against overturning is approximately 60 H kPa, where H is the height of the wall in metre. When this stress is compared with the allowable compressive strength of masonry in Table 6.1, the allowable height of each type of masonry retaining wall can be found. These are presented in Table 6.4.

Old masonry retaining walls in Hong Kong are 6 m high on the average. They rarely exceed 12 metres high. From Table 6.4, it is seen that with the exception of dry packed random rubble retaining walls, compressive failure of masonry retaining walls is unlikely. For dry packed random rubble walls exceeding 4 to 5 metres high, the margin against compressive failure depends on the shape of the blocks as well as the quality of the joints and beds. The stone blocks of these walls should be carefully examined and compared with the physical characteristics of random rubble and squared rubble described in Section 6.2.1.

Tables 6.1 and 6.4 are prepared for masonry with stone blocks of intact strength of 100 MPa or higher. If partially weathered stone blocks are used, as is the case of some poor quality random rubble walls, the allowable height will be smaller. The compressive strength tables in Appendix E can then be used to estimate the new allowable heights. If a masonry wall has some unfavourably shaped blocks, there will be local distresses even if the compressive stresses are less than the strength of the masonry (Figure 6.11).

6.4.2 Tensile Failure

When Hope (Anon, 1845), Burgoyne (1853) and Casimar (1974) carried out destructive loading tests on masonry retaining walls, they all observed that triangular fragments of the masonry remained at the lower inner corner of the walls. This was attributed to that masonry fail along the angle of repose of the material (Casimar, 1874). This is not true because regularly shaped, hand stacked material such as masonry does not possess an angle of repose. A more logical explanation can be seen from Figures 6.5(b), (c) and 6.8(b), (c) which show the minor principal stress distributions in a wall. At the lower inner corner of the wall, the minor principal stresses act at an inclined direction with a tensile nature. The extent and magnitude of this tension region increases rapidly with decrease in factor of safety against overturning. This inclined tension would induce cracks along a stepped combination of joints and beds (See Figure 6.12). The blocks below this line detach from the main body of the wall and remain as a triangular panel when the wall overturns.

The minor principal stresses act at a sub-horizontal direction at the front of the wall. Whether the wall can take this tension or not depends on the horizontal bonding of the wall. If the masonry well bonded, the tension will not affect the integrity of the wall (Section 6.2.3). If the masonry is poorly bonded, the tension will separate the masonry into different sub-vertical columns. This can be the case with untied stone rubble walls with small size core materials. The bonding between the face layer of blocks and the core material of this type of wall is usually poor. In Figures 6.5(b) and 6.8(b), it can be seen that the height of these sub-vertical columns increases rapidly with reductions in stability against overturning. If these columns are sufficiently high, the outermost one may buckle under the vertical compressive stresses. When this happens, the wall can no longer take the stresses and may collapse structurally. This mode of instability is associated with bulging at the lower end of the wall. The tendency to buckle also depends on the state of the stone blocks and the orientation of beds. Irregularly orientated beds are more detrimental to stability (See Figure 6.13). If the wall is tied, the stone headers will prevent the formation of isolated stone layers and hence prevent this mode of buckling instability.

6.4.3 Shear Failure

It was discussed in Section 6.2.4 that the shear strength of a stone masonry depends on the direction of movement and whether mortar is used or not. Shear movement usually takes place along the continuous sub-horizontal joints of the masonry along which shear resistance is smaller. Stability against sliding in this direction is thus considered. In the following paragraphs, the performance of mortared masonry walls is discussed first, followed by that on dry packed walls.

Figures 6.6(c) and 6.9(c) show the magnitude of horizontal shear stresses in a gravity retaining wall. For a 10 m high wall, the maximum shear stress is of the order of 80 and 130 kPa for the dry wall and the wet wall respectively. The corresponding local vertical stresses are 200 kPa and 150 kPa. The smaller vertical stresses in the wet wall is caused by the upthrust of the groundwater flowing through the masonry wall. For the case of a mortared wall with mortar strength of 1 MPa, the shear resistance is shown in Table 6.2. The factors of safety against local shearing are calculated to be 1.95 and 0.97 for the dry and the wet wall respectively. That is local slips will not occur at the joints.

However, if the groundwater level is higher, the shear force would be larger but the vertical compression stress at the location of maximum horizontal shear stress would decrease. As a result, the factor of safety against local slip will drop below 1 and local slip occurs. In doing so, some of the shear stresses will be redistributed to the front of the wall where extra shear resisting capacity is present. In such case, an average factor of safety against sliding should be calculated in the normal manner in retaining wall design using the shear strength properties of the masonry joints. If the factor of safety drops below 1.5 sliding across the masonry may be critical. However, with the shear strengths quoted in Table 6.2, this mode of shearing across a mortared masonry wall is less likely than the usual sliding at the base.

If the masonry is dry packed, the shear strength can be represented by a frictional angle of approximately 45° (Section 6.2.4). The resulting factors of safety against local slip are shown in Figures 6.7(a) and 6.10(a). Although some local slip is likely at the lower inner corner where the vertical compression is low, the stability of the section as a whole should be satisfactory.

When the beds between the stone blocks are subjected to shear stresses, there will be shear displacement at each bed. The amount of movement depends on the shear modulus of the beds and the magnitude of the stresses. As a result of the shear displacements, at each level of beds, the wall moves forward. The amount of shear displacement is greater at the bottom. Consequently, the wall will take up a curved profile similar to that observed in Burgoyne's wall C prior to failure (Section 4.3 and Figure 6.14).

In the absence of information on the shear modulus of masonry, it is not known whether this deformation is of significance or not. However, this is a possible mechanism of the bulging of masonry retaining walls.

If the masonry wall is composed of stabilised soil filled with stone facing, shear failure will be in the direction of maximum shear. Figures 6.7(c) and 6.10(c) show the factors of safety against such shear failure. The factor of safety 'F' is defined by

$$F = \frac{S}{\frac{1}{2}(\sigma_1 - \sigma_3)}$$

where S = shear strength of the material
= ½ (σ_c)
σ_1 = major principal stress
σ_3 = minor principal stress
σ_c = uniaxial compressive strength of the material

This set of definitions has implicitly assumed the Tresca's (maximum shear stress) failure criterion for the material. This failure criterion is a very approximate one but should be satisfactory for the present general analysis. GCB has carried out tests on samples of the stabilised soil from wall no. 11SW-B/R617. A mean uniaxial compressive strength of 2.0 MPa was recorded. The corresponding shear strength is 1.0 MPa. From this, the permissible height of wall without local shearing for the case of the wetted wall is

$$H = \frac{0.5S}{10 \text{ x } F} = \frac{0.5 \text{ x } 1000}{10 \text{ x } 1.0} = 50 \text{ m}$$

Conversely, for a 12 m high wall, the strength of the material should be no cause for concern.

The samples of stabilised soil tested by GCB were dry. The material is liable to weakening by saturation. Therefore, if the groundwater table behind a wall is high, test results of saturated samples should be used in the analysis.

6.4.4 Structural Instability Involving Cored Wall Structures

Up to this point, the effect of the cored structures of masonry on the various mode of structural instability has not been discussed. The structural behaviour of the core materials varies over a wide range depending on their sizes and the way they were deposited. The mechanisms by which they cause structural instabilities are complicated. Therefore, it is more desirable to discuss their structural performance under different stresses conditions as a whole.

If the core material is of gravel size, it will behave as a granular material with an internal angle of friction of approximately 35°. Under compressive stress, a lateral pressure will be induced on the face layer (Section 6.2.2). For a 10 m high wall, this lateral force varies from 0 at the top to 135 kPa at the bottom. Unless the face layer of masonry is adequately bonded (tied) to the core, it is not strong enough to resist the lateral pressure. As a result, the face layer of blocks will bulge out and even collapse.

The gravel material in the core would be too weak to resist the internal shear in the vertical direction. From Figures 6.7(b) and 6.10(b), the factors of safety against vertical internal slip for material with $\phi = 35°$ are far much below 1 for most of the height of the wall. As a result, internal slip will take place. This would lead to a loss in the resistance against overturning and the wall may fail. The internal slip is accompanied by lateral dilation of the material which would cause bulging of the face layer of the wall. However, not all of the surface layer can dilate freely. The top and the bottom of the layer is restrained from doing so. The result is a bulge more prominent at mid-height than at the ends. If the face layer is strong enough to provide restraint against dilation, the shear strength of the gravel would be much higher. In that case, the stability of the wall may be maintained. The best way to maintain structural integrity in this case is by adequate bonds between the surface layers and the core. These bonds would provide some cohesion to the material as well as to restrain the core material against dilation.

It should be noted that towards the rear of the wall, the factor of safety against vertical internal slip rises to unity. At this location, gravel material can be used without any problem of internal shearing.

Bulging of the face layer of the masonry is detrimental to the structural stability of the wall in two respects. The arched masonry column has a reduced compressive strength. The amount of reduction depends on the amount the bulged profile deviates from the mean alignment and on the thickness of the blocks. Secondly, when the gravels dilate under shear, there is a peak amount of dilation above which the shear strength decreases to a residual value. This was found to be 80% to 50% of the mean particle size (Patwardhan et al, 1970). If the face block layer bulges by more than this amount, there will be a local reduction in shear strength. Internal slip will follow with the likely result of complete collapse of the wall.

If the core material consists of large size (bouldery) particles randomly dumped into position, it will also act as an isotropic granular material. It will have structural problems similar to gravel core materials. However, the internal shear angle of this material can be as high as 70°. Consequently, the size of the problems is much smaller.

For such materials, the lateral pressure that can be induced on the face layer of a 10 m high wall is around 14 kPa. This is usually too small to cause distress of the face layer. The factor of safety against internal slip in the vertical direction is still too small. However, greater dilations of the material is needed before the peak shear strength is reached, thereby causing larger bulging of the wall before the wall fails. Also, the increase in shear resistance due to restraints against dilation would be larger.

If the large size core materials are slightly slabby or were hand packed in position, the behaviour would approach that of random rubble masonry. Very small lateral pressure, if any, will be induced on the face layer of blocks by the vertical compressions. The material will also possess some amount of cohesive strength due to interlocking of the blocks (Section 6.2.4). This may be enough to resist the vertical shear stresses.

6.4.5 Other Factors Affecting Structural Instabilities

In the above analyses, the number of factors that have been considered were necessarily restricted. These factors are the properties of the stone blocks and the beds, the strength of mortar, the wall structure, and the effects of groundwater. There are other aspects of a masonry wall that may affect the likelihood of structural instability.

The stress analyses in Section 6.3 are for rectangular walls with a height/base width ratio of 3. For walls with wider bases, the magnitude of the stresses will be smaller. The converse is true for walls of smaller base widths. The H/B ratio of 3 was adopted in the analysis because this was found by GCB to be the critical values for stable walls in

Hong Kong.

When the face of masonry wall is battered, the stress distribution pattern will also change. Generally, both the maximum values of the shear and compression stress will be reduced. The more important effect, however, comes from the changes in the inclination of beds in the masonry.

It was customary to lay masonry blocks with beds perpendicular to the front face. When the face battered backward, the beds incline against the direction of earth pressure. The result is an increase in the resistance against horizontal sliding. Because of the restraint of the face blocks, the masonry cannot slide in a vertical direction. Instead, it has to slide sub-parallel to the face of the wall. In that orientation, the self weight of the blocks would contribute to the normal stresses at the potential slip plane. This would cause a corresponding increase in the shear resistance of the masonry. For walls that batter as much as the Japanese walls (65°), this increased shear strength enables gravel material to be employed behind the face blocks without using headers for reinforcement. For the same reason of the effect of the self weight of the blocks, the battered face layer is less likely to buckle outward (Section 6.4.2).

6.5 Structural Behaviour of Tied Face Walls

With its criss cross units of headers and stretchers and with the random soil/rubble infill to the cavities, the tied face wall resembles a modern crib wall in behaviour. The lengths and sizes of the members are also comparable to that of the crib walls.

In the tied face wall, the stretchers are placed in consecutive courses. This is a big improvement over the normal crib wall where the stretchers are supported on the headers. In normal crib walls, there are wide separations between each course of stretchers. Consequently, the reinforced concrete stretchers have to take up bending moments induced by the self weight and by the earth pressures. Also, there will be concentration of compression at the contacts between the headers and stretchers. This concentration of force will in the end control the maximum height a normal crib wall that can be economically constructed. The tied face wall, on the other hand, always possesses adequate compressive strength in the normal range of wall height.

For good quality granite strips, the tensile strength can easily exceed 10 MPa. With the normal sectional area of 0.30 x 0.15 m^2, the tensile resistance of an average header unit in a tied face wall is 0.45 MN. It is equivalent to the tensile resistance of ten 20 mm diameter mild steel bars of 140 MPa permissible stress. This amount of reinforcement is larger than that normally provided in a reinforced concrete header. Also, the concrete headers in a crib wall are usually at wider spacings. Therefore, the granite headers in a tied face wall should have very adequate tensile resistance.

There is, however, the problem of poor mechanical anchorage between the granite headers and the stretches. Under normal circumstances, the contact pressure between the blocks generate sufficient friction between the headers and stretchers to ensure the integrity of the wall. Because of the tight, close fitting stretcher strips, the wall possesses considerable longitudinal rigidity. In the presence of differential movement, the strips provide good arch effects. The contact pressure between the blocks would subsequently be reduced in the lower part of the wall where the settlement occurs. This causes loss in friction between the headers and the stretchers. Consequently, the stretchers move out with respect to the headers and cause bulging of the wall. Once bulged, the wall would lose some of its structural strength.

7. INVESTIGATION TECHNIQUES

7.1 General

There are a number of parameters that cannot be obtained in surface inspection and yet must be known before the stability of a masonry retaining wall can be judged with certainty. Among these, wall thickness which affects the static stability of a retaining wall is the most important one. Knowledge of the structure of a masonry wall is also significant in the evaluation of the structural performance of the wall. The third parameter is the source of seepage on a wall. When the seepage on a wall is persistently high, it may be caused by leakage from water carrying services. If the leaking pipe can be found and repaired, the improvements in the stability of the wall may be very large. This can be more easily done if the nature of the seepage water is known. In this Chapter, some techniques for obtaining information on the mentioned parameters are described.

When old masonry retaining walls are inspected, cracks and fissures are commonly found on the wall and the crest platform. In the last part of the Chapter, common causes of crack formation are discussed so that cracks that are related to wall instability can be duly identified and stabilisation works can be carried out in time.

7.2 Seismic Probing

There have been past attempts by both GCO and GCB on the use of seismic reflection methods to find the thickness of masonry retaining walls. The outcomes were not encouraging because of difficulties encountered in the interpretation of the results. In particular, it depends on an assumed value of V_p, the velocity of propagation of compression waves. V_p varies over a range of values depending on the void ratio and the strength of the material.

It was felt that if the range of V_p (velocity of propagation of compression wave) values could be correlated with the nature of the masonry, it could be adopted as a means of geophysical investigation. In particular, if V_p of the core material of the masonry retaining wall could be measured, it could be used to define the lateral variations of the core material. This would be an important supplement to the information from conventional drill holes for the assessment of the structural behaviour of the wall.

The proposed method was by direct measurement of the time required by a compression wave to travel between two weep holes. An equipment was developed by the Electronics and Geophysics Service Ltd (EGS). It consists of two transducers, one acting as the source and the other as a receiver, mounted on the ends of two poles. They are connected by cables to a timer with digital display (Plates 7.1, 7.2, Figure 7.1). After both transducers are inserted into the weepholes, a shock can be given to the inside of the weephole adjacent to the source transducer. This activates the transducers and causes compression waves to propagate in the masonry. When this wave is intercepted by the receiver transducer, the time taken for the wave to transverse the distance between the two holes is displayed in the timer. From this, the velocity V_p can be calculated. Usually other forms of waves are also generated together with the compression wave. However, the compression wave, being stronger and faster, are always intercepted first so that V_p is the most usual calculated value. The face layer of masonry walls in Hong Kong are usually denser and of better quality. Consequently, they have higher V_p values than the core materials. If the transducers are placed too near to the face blocks, V_p of the face layer will be measured instead of the core. The same happens if the inducers are too widely spaced. A spacing equal to the width of the wall is deemed satisfactory.

A trial was carried out in November. The measured results were unsatisfactory because of poor contacts between the inducers and the walls of the weepholes. Modifications to the contact arrangements are being carried out for a further trial.

7.3 Weephole Probes

This method was first adopted by GCB in the investigation of old masonry retaining walls. It is done by pushing a straight edge through a weephole to measure its length. Good relation was found between this and the wall thickness. The measured length, however, was always smaller than the thickness of the wall as deduced from conventional core drillings. Debris, especially soft drink cans and glass bottles are often found in these holes. It is possible that the debris obstructed the passage of the straight edge and so caused a smaller reading.

In this study, the extension rods and the sharp point of the GCO probe were used to probe the weepholes. The pointed tip of the probe has good penetration abilities. However, they also tend to penetrate deep into the soft backfill and so cause high measured values. A flat end piece was thus made and gave better results (Plate 7.3). The later adopted procedure consists of probing with the pointed end first to break through the obstacles followed by probing with the flat end to measure the length of the weephole. The difference in the measurements is an indication of the nature of the backfill material.

7.4 Drilling Equipment

Drilling of cores remains the only method by which the structure of a wall can be examined and the thickness of the wall can be measured with higher certainty. It can either be carried out by a normal site investigation drilling rig or by an electric core cutter. Both methods employ water as the flushing agent.

The electric core cutter is portable (Plates 7.4, 7.5) and can be used in limited spaces. Penetration of 5 m has been achieved by this type of machine. However, being a single tube core barrel, the recovered core samples are very much disturbed. This is especially the case for the material behind the face blocks of which finer portions may be completely washed away by the flushing water. Whilst these low quality cores can still be used to find the thickness of the wall, the structure of the masonry is no longer observable. This causes difficulties in assessing the structural behaviour of the wall. The ordinary site investigation drill rigs are more powerful and can be employed to recover undisturbed samples of the backfill. The quality of the samples of the masonry, however, is still much disturbed by the flushing water.

There are examples that drilling in a wall reactivates old cracks, causing fresh differential settlements of the wall and dislodging of face blocks (e.g. wall 11SW-A/R73, Plates 7.6, 7.7). This is due partly to the vibrations induced by the drilling machine and also the flushing water which washes away the finer materials in the wall and in the backfill. Such disturbances to the wall are undesirable and can be minimised by the use of better machines with better controls of flush water pressure. Foam drilling, if employed, can avoid loss of the fine material and allows the structure of the masonry to be retained in the core for inspections.

7.5 Seepage Source Identification

When a retaining wall shows signs of persistent seepage, the possibility of leakage from water carrying service pipes should be investigated. This can be done by analysing the chemical contents of water samples collected from the seepage. For accurate diagnosis, one litre volume samples are required. Such large samples are usually difficult to be collected from the minor flows from weepholes.

An alternative is to send smaller water samples for identification of "tell-tale" chemicals. These are summarised in Table 7.1.

The absolute minimum volume of samples necessary for these tests is 300 c.c. If part of these tests give positive results, larger volume samples should be collected for more detailed analyses in the Government Laboratory.

7.6 Crack Diagnosis

Cracks can often be found in old masonry retaining walls and on the crest and toe platforms. Some of these cracks are the results of changes unfavourable to the stability of the wall. Others may have been caused by completely unrelated agents. It is thus very important that the nature of the cracks around a wall is properly diagnosed.

7.6.1 Cracks on Masonry Retaining Walls

The most commonly observed cracks on masonry retaining walls are those caused by restraints against contraction. The width of the cracks depends on the amount of contraction. Their spacing depends on the nature and magnitude of the restraining force as well as the tensile strength of the masonry. Figure 7.2 shows some possible crack patterns.

The contraction may be a result of shrinkage, seasonal temperature variations, and early thermal movement. Shrinkage is caused by dry out of the wall constituent material. In Hong Kong, the amount of shrinkage movement is much less than that caused by seasonal variation in temperature. When a lime/cement bound material sets, it liberates heat of hydration which raises the body temperature. When it cools, the accompanied thermal contraction causes cracks similar to the normal thermal cracks and are distinguished from them by the name early thermal movement cracks. These early thermal movement cracks, being always formed at a time before the materials fully gained its strength, are closer in spacing and are narrower.

The "horizontal beams" in masonry retaining walls are most susceptible to the early thermal movement cracks because of the thin member sizes and the large restraint at the base. When the bulk of the wall contracts at a later stage, the new contraction cracks would pass through some of the older cracks in the horizontal beams and widen them. Therefore, it is usual to observe cracks at regular spacing but of different widths on the "horizontal beams" of the walls. Some good examples of this can be seen in wall 11SW-B/R271 (in Kennedy Road near to the Peak Tram way) and wall 11SE-A/R58 (New Orient Terrace). In wall 11SW-B/R271, cracks are present on the horizontal beam at every 7 number of the stone blocks (approx. 2.4 m spacing). In wall 11SE/A/R58, the crack spacings are approximately 1 m (3 no. stone blocks). The crack spacing may be an indication of the strength of the "horizontal beams". The wider spacings are usually associated with the stronger materials.

The effect of these normal contraction cracks on a wall is to divide them into individual sections of walls. Basically they are not detrimental to the stability of the walls. Nor are they signs of instability unless relative forward wall movements are observed across them. In that case, the wall with the larger forward movement may posses smaller margin against instability.

For walls composed of concrete or stabilised soil, horizontal fissure may also be found. These walls were usually constructed in layers. If bonding between consecutive layers is weak, early thermal movements may cause sliding across the interface and form the fissure. The effect of this fissure is to weaken the shear resistance of the wall locally by the removal of the cohesion component of the constituent material of the wall.

The other common type of cracks is caused by differential settlement. They form fissures that spread at an angle from the vertical away from the point of larger settlements. The magnitude of this angle depends on the force producing the differential settlement, and the direct shear strength of the material. The larger the force or the weaker the material, the steeper the orientation of the fissure will be.

It is not exactly known how this types of cracks may affect the stability of a wall. Apparently, if they are narrow or if the core of the wall is flexible so that some structural interlockings are present at the crack, they would not affect the wall stability to any significant extent. Otherwise, the wall would be divided into two structural limits and stability evaluation has to be proceeded separately.

Detail examination of the cracks and fissures can also provide useful information. Cracks which widen towards the top indicates that the two halves of the wall across it have rotated away from each other. The converse is true for a crack that widens at the bottom. The wall behind Hok Sze Terrace (adjacent to wall 11SW-A/R332) has a prominent inclined crack across it. Close examination of the crack showed evidence of horizontal relative movement only. The crack is therefore a contraction one. Further examination shows that it is along the interface between an old wall and a later extension. This explains why the crack is inclined (Plates 7.8, 7.9).

When a masonry retaining wall is continued around a corner, subvertical fractures may appear near the corner (See Figure 7.3). These may be contraction cracks although they are more likely to be caused by forward movement of the front walls. Being longitudinal to the direction of movement, the side wall cannot cope with the movement of the front wall and consequently cracks. This may or may not be a sign of incipient instability. If it is caused by the forward movement of the front wall necessary for the mobilisation of the active state of pressure, the summed width of the cracks should not exceed 0.001 H at any point. Otherwise, the crack is caused by excessive movement of the front wall. Further information can be gained by monitoring the width of the crack for a period of time to see if it is active.

7.6.2 Cracks on Crest Platforms

Before a retaining wall fails, cracks sub-parallel to the wall are usually found on the crest platform. These cracks are continuous for long lengths, and may widen to great separations. From the case studies on wall instabilities (Chapter 4), none of the observed cracks were narrower than 20 mm and some were as wide as 150 mm before the wall collapsed. Some photographs of this type of cracks are incorporated into the descriptions on Case 6 (Circular Pathway) of Appendix D.

The above described cracks which are caused by forward tilting of retaining walls should not be confused with fissures originated from structural defects of the pavement slabs. Early thermal movement and thermal contractions can induce fissures on a slab if it is not adequately reinforced. The resulting fissures are usually randomly distributed and orientated. They may change their orientations appreciably along their length. Near the joints of the slabs, the contraction fissures may take up sub-parallel orientations. These type of cracks and fissures seldom exceed one mm in width.

If the subgrade to a thin pavement slab subsides, the slab will fracture. These settlement cracks are accompanied by variations in levels or surface gradients across the cracks. The subsidence may be the results of wall movements. But more often, it is due to loose filling or poorly prepared subgrade.

Long, continuous and relatively straight clefts are often found adjacent to newly backfilled trenches. They are usually a few mm wide and are mostly the result of loose backfilling to the trench. Under surcharge, the subgrade moves towards the trench and form the clefts.

Trees on the crest platform may also cause fracturing of the pavement slab by the growth of their roots. The lateral extends of such cracks can usually be traced back to the locations of the trees. They are sometimes accompanied by slight upheaval of the slabs.

Apart from the type of cracks caused by forward tilting of retaining walls, all the other cracks, fissures or fractures described above are not signs of possible instability of retaining walls. They may be detrimental to the stability of the wall mainly because they allow infiltration of water into the retained soil and may saturate it. The amount of infiltrations from these paving slab defects is unlikely to be significant unless they are very wide and are covered by ponds of surface water.

An useful tool for the accurate diagnosis of the origin of cracks is to draw plans showing the location of the cracks together with the ground and wall features for an overall appraisal. Ground features such as trees, ground subsidence, surface channels, and signs of recent trench work are worth recording. Changes in wall type, wall face slope angle, cracks, and bulges on the wall should also be marked on the drawing.

8. EFFECT OF TREES ON STABILITY OF MASONRY RETAINING WALLS

8.1 General

The effects of trees on the stability of masonry retaining walls are largely unknown. Little research, if any, has been devoted to this area. This Chapter sets out to discuss some background information and thoughts on the tree/wall interaction with a perspective view to a better understanding of the behaviour of walls under the action of trees.

8.2 Background Information on Trees

The most abundant type of trees occurring naturally in masonry walls in Hong Kong is Ficus Microcarpa, commonly known as Chinese Banyan. It is an evergreen tree typically 6-15 m high with a crown span of 16-30 m supported by a trunk of 300-500 mm in diameter (Hill, 1967). This type of tree can easily be recognised by its characteristic abundance of aerial roots (Plate 8.1). It has a shallow and widespread root system. Under normal conditions, the roots are confined to a shallow depth probably less than 3 metres. The spread of the roots are approximated by the crown of the trees (Yung, 1980). When growing on walls, the tree usually develops a surface network of ramifying roots for support (Hill, 1976) (Plate 8.2).

The tree can survive poor environment and can grow in almost any site given the availability of moisture. In particular, the tree is indifferent to the action of lime and can thrive in stabilised soil fill which makes up the core of some of the masonry retaining walls (Ho, 1981).

From observations, the growth of the Chinese Banyan depends very much on the type of wall where it takes root. It grows most readily on dry packed random rubble walls because the large gaps between the rubble provide ready access to the seeds, and allow free passage of air and moisture for the thriving of the tree.

For stone rubble walls with narrow joints and beds, the normal tap cannot grow properly. The resulting reduction in support to the tree is compensated by the better developed system of ramifying roots. If in the absence of a strong main root the growth of the tree is stunted to a smaller size.

8.3 Thoughts on the Effect of Trees on Retaining Walls

Trees on retaining walls can affect wall stability in three main ways. The growth of free roots may disrupt the masonry structure, the tree roots may interact with the retained soil and strengthen it. Lastly, the weight of the tree causes additional forces and moments to act on the wall.

When a tree grows in size, its roots expand and exert force on the stone blocks. If the stone blocks pressed against are firmly interlocked to the bulk of the masonry, they may resist the pressure and limit the growth of the roots. Otherwise, the wedge action of the growing roots may cause displacement of the blocks and weakens the masonry locally

(Plate 8.3). Therefore, whether root growth can disrupt the masonry structure depends very much on the quality of the face layer of the masonry.

If the tree root system penetrates a wall into the retained soil, it will reinforce the soil locally and increase the friction between the soil and the wall. This reinforcement effect should be more prominent for dense soils. Again, the amount of this effect is not known and it is unlikely that it can be found analytically. However, for walls with a dense structure and well packed face blocks, the poorly developed main root system would not be able to penetrate the wall and the reinforcing effect would be negligible. The same is true for walls thicker than 3 m which exceeds the usual depth of penetration of Chinese Banyan trees.

A tree on a retaining wall is an additional surcharge to the wall. It increases the overturning moment and the toe pressures. Consequently, the factor of safety against overturning of the wall is reduced. Whether this reduction is critical or not depends on the ratio of the surcharge effect of the trees to the restoring moment and forces of the original wall. If this ratio is small, the effect of trees on the wall stability can be neglected.

Therefore, before the surcharge effect of a tree on a wall can be evaluated, the order of magnitude of the forces and moments that can be induced by the trees must be roughly known. At present, such knowledge is completely absent. However, there are some possible ways of estimating it. The load carrying capacity of a tree trunk can be calculated from its mean diameter assuming that the wood is at its yield stresses. This would provide an upper limit to the magnitude of forces. Also, terrestrial photogrammetric techniques may be employed to find the spatial distribution and the average diameters of the branches and trunks of a tree. The density of the wood can be found by cutting a core or a branch from the tree. The approximate moment and forces that act at the head of the tree can then be calculated. This gives the lower bound value of the surcharges from the tree.

9. GENERAL METHODS OF STABILISING OLD MASONRY RETAINING WALLS

9.1 Methods

There are different methods of stabilising a masonry retaining wall depending on the different possible modes of failure. Some examples of the methods are shown in Figure 9.1 and discussed below.

(a) Partial demolition of the wall - With the method, the upper part of the wall is demolished. The retained ground behind the demolished portion of wall are cut back to a stable angle resulting in a reduction in the area of the crest platform. This approach improves stability against all modes of failure although the improvement against internal shear failure in a vertical direction is unlikely to be significant.

(b) Provision of drainage behind the retaining wall - For walls with high groundwater behind, this is a method by which stability against all modes of failure can be improved, with the possible exception of internal slip in a sub-vertical direction. The most common method to lower groundwater table is by the provision of horizontal drains through the wall. One serious problem with the horizontal drains is that it cannot be installed at lower than 1 m above the toe of the wall. This would mean that the wall has to sustain at least 1 m of groundwater. Unless some new methods are developed to install drains at lower levels, the use of horizontal drain may not be the final answer to the stability improvement works.

For retaining walls standing on a slope with water table at or above its base, signification improvements in the stability against foundation failure can be achieved by the installation of horizontal drains into the toe slope.

(c) Skin walls - This methods involves the construction of a reinforced concrete skin to the front of the masonry retaining wall. It improves the compressive capacity of the wall by reducing the maximum compressive stress on the masonry and by taking up additional stresses where local compressive failures occurred in the masonry. The skin wall also resists bulging of the masonry, and thus improves the shear resistance of the core material by restraining the dilatancy of the material.

For masonry retaining walls with unsatisfactory stability against sliding and overturning, improvements can be achieved by providing properly founded and keyed footing to the reinforced skin wall. Sometimes pile or caisson foundations may have to be provided. In all cases, the skin wall must be adequately dowelled to the old masonry wall. A rough guide to the number of bars required is that they should provide a shear resistance in excess of the shear force across the concrete masonry interface.

For retaining walls on a slope with danger of foundation failure, the skin wall cannot be used unless very substantial foundation works are incorporated. The preventive work is then similar to that of underpinning.

(d) Additional retaining walls - Retaining walls situated on a slope may suffer foundation failures. The best method to stabilise such wall is by the construction of another retaining wall downslope. The ground behind the new wall can then be brought up to a gentle gradient and to meet the old wall with a bench in front of its toe. The width of the bench should best be around 1/3 of the height of the old masonry retaining wall. The new retaining wall, however, may reduce the stability of the original slope. This must be checked against by analysing the overall stability of the slope and the retaining walls in the usual manner.

10. SUMMARY OF THE PRESENT STUDY AND PROPOSALS FOR FURTHER RESEARCH

10.1 Summary

Chapter 1 - The aim of the study is to collect information on the structures and modes of failures of old masonry retaining walls, to identify signs which are associated with incipient failures of these walls, to find methods of assessing their stabilities and to define the relative importance of different factors affecting stability of the retaining walls.

Chapter 2 - A review of past studies including those carried out by Binnie and Partners, GCB and GCO.

Chapter 3 - Composite construction is a common feature of masonry retaining walls in England, Japan, Korea and China. The English, Japanese and Korean walls all consist of good quality masonry blocks at the front and coarse granular material of various sizes at the rear. The Chinese walls have cores of granular material between a front layer of good quality blocks and a rear layer of fair quality blocks.

A glossary of terms for describing structures of masonry retaining walls is compiled and included as an Appendix C.

In Hong Kong the tied-face wall consists of stone strips 'box-bonded' together to form a cavity structure. The cavities are infilled with rubble and earth.

The stone rubble walls have cored structures similar to the Chinese walls. The quality of the face blocks varies from random rubble to well-dressed blocks. The nature of the core material also varies widely. Some walls are provided with stone headers while some others are provided with horizontal tie courses locally known as 'horizontal beams'. These improve the structural integrity of the walls.

Stone pitchings can also be treated as masonry retaining walls.

Chapter 4 - Ten cases of instability of retaining walls were examined. It was found that most of the failures in the Mid-level area lied on the north boundary of Mid-level Development Restriction Area recommended in 1979. Over half of the failures were triggered off by earthworks, mostly trench-works, in the vicinity of the walls. The wall failures were preceded by bulges of the walls and opening of cracks at the crest platforms. None of the failures reviewed involved stone rubble walls with tie courses.

The consequences of most of the wall failures were serious.

Observations of masonry retaining wall failures in Victorian England showed that the walls tilted forward and bulged before failure. Bulge profiles of three walls prior to failure were collected. A bulged wall might stand for a long time before ultimate failure.

Chapter 5 - From observations in Chapter 4, masonry retaining walls are found liable to static failures. Due to low compaction pressure during construction, the conventional earth pressure formulae can be used to estimate the pressure. The stability of retaining walls is affected by groundwater table, crest slope angle and soil shear strength parameters, in descending order of significance. Retaining walls which lean backwards can remain stable at small thickness.

Foundation failures are possible in retaining walls standing on slopes with gradients exceeding some critical values. The critical toe slope angle depends on the distance of the toe of the retaining wall from the edge of the slope, soil strength, ground water location and the buried depth of the wall in a descending order of influence.

Chapter 6 - The permissible strength of masonry is examined. The stresses in a masonry wall are calculated for simple boundary conditions. When the two are compared, it is found that for masonry retaining walls, structural failures are possible. Dry packed random rubble walls may fail in compression if higher than 5 m. The main mode of failure for all stone rubble walls is internal slip in a sub-vertical direction. Resistance against this shear failure is by the interlocking of the masonry, header stones and tie courses (horizontal beams) in order of increasing efficiency. Walls with gravel cores are structurally less stable. Mechanisms are put forward to explain bulging of walls prior to failure.

Tied-face walls are similar to crib walls in behaviour. Differential settlement of such walls may affect the linkage between the headers and stretchers and causes bulging.

Chapter 7 - Seismic probing between weepholes is a potential geophysical method for investigating internal structure of masonry retaining walls. Mechanical probing into weepholes may be used to measure wall thickness. Normal drilling method may affect structural integrity of masonry walls. The quality of the cores recovered from ordinary drilling do not allow close examination of the masonry structure. High quality foam drilling may be more preferable. Common causes of cracks on masonry retaining walls are restraints against contraction and differential settlement. Most cracks of these natures are not detrimental to wall stability. Sub-vertical through cracks at return walls may be caused by movements of the front walls. Long cracks may appear on the crest platform of the wall before it fails. It can be distinguished from cracks and fissures of other harmless origins.

Chapter 8 - The most common trees growing on masonry retaining walls are Chinese Banyans. Their typical features and member sizes are collected from literatures in Botany. Trees may affect wall stability in three ways : dislodging of stone blocks by roots, reinforcement of the retained earth by roots penetrating into the walls and addition of loads on the walls. The magnitudes of the additional loadings must be known for evaluation of their effects on wall stability. They may be found by estimating the load carrying capacity of the trunks or by photogrammetric measurements of the sizes and distributions of the branches.

Chapter 9 - Some general means of stabilising masonry retaining walls are proposed.

10.2 Further Research and Studies

Chapter 1 - none.

Chapter 2 - none.

Chapter 3 - The collection of sections and construction details of old masonry retaining wall should be continued especially when wall sections are occasionally exposed by the now more frequent preventive works.

Chapter 4 - Death enquiries were carried out after the failure of retaining walls at St. Joseph College and Po Hing Fong (Cases 1, 2). Although it is not possible to find the court record of these hearings, important abstracts were published in the newspapers at the times of the enquiries. A search in old newspapers would yield more information on the contemporary views of the engineers on the design and construction of masonry retaining walls.

Chapter 5 - Sensitivity analyses similar to those in this Chapter may be used to check the relevancy of the present score arrangements to the various components in the ranking system.

Chapter 6 - The American standards on masonry works especially the relevant A.S.T.M. standards, should be examined in greater details. A research into the various requirements on headers in masonry in various building standard would provide criteria by which a tied stone rubble wall can be regarded as satisfactory in shear resistance or not. The mechanism of bulging of walls in this Chapter are put forward mainly on theoretical/analytical basis. They should be proved either by detail observations of unstable walls identified in the future or by carrying out small scale model tests. A technique of using thin aluminium pieces between two glass (perspex) plates to represent the array of blocks in a stone rubble masonry can be employed to form the model (See Figure 10.1). This is similar to the method used by Terzaghi (1920) to observe intergranular movements when a granular soil is sheared. Typical dimensions (especially the lengths) of the stone headers should be collected together with typical strength of the concrete in the horizontal beams in the walls.

Additional stress analyses should be carried out on gravity wall of other shapes and loading conditions and with allowances that masonry cannot take direct tension.

Chapter 7 - Further trials and improvements on the techniques described in this Chapter should be carried out in association with the investigations on masonry retaining walls. Before the seismic probing method can be utilised in actual site investigations, the velocity (V_p) has to be calibrated against the structures of masonry walls. The methods of monitoring movements of retaining walls were not discussed in the present study. With the likely increase in the number of walls to be monitored, the usual methods should be reviewed and improved to provide methods that are more reliable and easier to operate.

Chapter 8 - Programme to be started to collect observations on the characteristics of root systems of Chinese Banyan especially when they are exposed during the execution of preventive works on old walls. The assistance from some institutes on Botany is required. Further studies on the magnitude of loadings that Chinese Banyans can induce on retaining walls are necessary.

Chapter 9 - Stabilisation Methods.

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Table 2.1 - Geotechnical Parameters Adopted in Caine Road Area Study

Materials	Density (t/m^3)	Cohesion c' (kPa)	Angle of Friction ϕ' (degree)	Remarks
Decomposed Granite	2.0	0	38°	CD test gives ϕ' values 3 deg. higher than CU test
Colluvium	2.0	1.0	33°	From Robinson Rd. Area Study
Fill		0	35°	Assumed, considered too variable to be generalised
Masonry	2.4	0	30°	Assumed
Soil Cement Backing	2.0	4.0	35°	Assumed
Wall friction angle (δ)			20°	Assumed with reference to I.C.E. (1951)

Table 3.1 - Types of Masonry Retaining Walls According to B & P

Wall Type Designation Number for the Computerised Phase 1A Data	Wall Type
1	Dry Random Rubble Wall
1	Mortared Random Rubble Wall
2	Dry Squared Rubble Wall
2	Mortared Squared Rubble Wall
3	Dry Squared Rubble Wall with Horizontal Beams
3	Mortared Squared Rubble Wall with Horizontal Beams
4	Dressed Block Wall
5	Dressed Block Wall with Horizontal Beams
6	Tied Face Wall
7	Tied Face Wall with Horizontal Beams

Table 3.2 - Dimensions of Face Blocks of Japanese Stone Retaining Walls

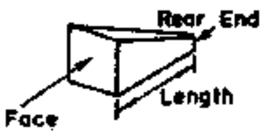
	Vertical Height of Wall (m)	Length (m)	Rear End (cm^2)	Maximum No. of Blocks/m^2
	0 ~ 1.8	0.45	50 ~ 100	11
	1.8 ~ 4.5	0.60	65 ~ 130	9
	4.5 ~ 7.2	0.75	75 ~ 160	7.5
	7.2 ~	0.95	75 ~ 200	6
Note : Table based on Yamada (1975).				

Table 4.1 - Case Studies - Sources of Information

Location	GCB's Case Number	Date	Sources of Information
St. Joseph Terrace	-	10.7.1917	Mid-level Studies. A review of the sources of information available in Hong Kong. Addendum : Newspaper Reports
Po Hing Fong	-	17.7.1925	Ditto
Alberose, H.K.U.	F20	1961	1,2,3/3032/59
10 Castle Road (I.L. 7976)	F14	19.6.1970	D204/70/H/K/ 13/2943/63
Thorpe Manor 1, May Road	F32	2.9.1973	D186/78/H.K. 1,2,3/2180/72
Caine Lane J/O Caine Road	-	25.8.1976	H.H.C2, Aerial photographs
Furniture Factory, 20, Lung Wah St., Off Pokfield Road	F19	25.8.1976	1,3/2357/54
Circular Pathway 3-10, (I.L. 4490)	F1	8.1977	D167/77/H.K. 1,2,3/2558/58
22 Old Peak Road (I.L. 1146)	F16	11.5.1978	D191/76/H.K.
14-16 Fat Hing St. Adj. 48-56, Queen's Road West	F31	29.7.1978	D26/72/H.K. 1,2,3/2101/76
Po Lo Che, Sai Kung	F27	29.7.1978	D357/78/K
1-10 Wing Wa Terrace (6-8, Hospital Road)	F17	13.11.1978	D232/74/H.K. Discussion with the Contractor (Wing Tai Co.)
Shing Mun Road	F25	15.6.1979	(W) D 179/79/H.K.
14, Shek Pai Wan Road	F41	1.7.1979	D 290/79/H.K.
14 Broadwood Road	F30	26.9.1979	D 105/77/H.K.
Jewish Recreation Club, Robinson Road	-	3.8.1979	

Table 4.2 - Summary of Case Study of Old Masonry Retaining Walls (Sheet 1 of 2)

Case Number	Location	Failure Date (Time)	Wall Type	Height	Normal Water Table/ Seepage Level	Adjacent Works Immediately Prior to Failure		Signs of Distress	Consequence of Failure	Remarks
						Location	Nature			
1	St. Joseph's Terrance (18-24A, Caine Rd)	16.7.1917 (11:00)	Stone wall with soil cement bound rubble infill	15 m	Not known	Crest	Paving on half of platform removed for reconstruction.	2 inch wide crack at corner of the wall, which widened to 6 inches in 3¼ hr. The wall collapsed after another 1¼ hr.	The rear structure of two houses was torn from the main building and buried beneath a great mass of earth and stone.	
2	Po Hing Fong	17.7.1925	Random rubble	Not known	At toe level	Crest	Excavation for the foundation of a new building was underway. The crest and toe platform were generally flooded.	The main failure was preceded by collapse of two matsheds at the rim of the crest platforms This was followed by 30 sec. to 1 min. of loud rumbling noises before the lowest wall failed.	The failure caused collapse of 7 three-storey buildings with 200 lives inside	Failure of a stake of 3 walls
3	10, Castle Road (I.L. 7976)	19.6.70	Stabilised soil with rubble plums	Not known	High	Construction site at toe Road at crest	Foundation works for I.L. 7976. The excavation was supported by sheet piles. Recent trench works by Gas Co. newly backfilled. Bursting of 2 water mains was observed in the failure, no evidence that they have been leaking before the failure.	Not known	Temporary closure of half of Castle Road.	Immediate cause of failure not known. Both the sheetpiling and the excavation had been there for sometime. The inspection engineer was with the view that the trenchwork permitted the fast infiltration and movement of water which led to the failure.
4	Thorpe Manor 1, May Rd	2.9.1973 (13:45)	Squared rubbles with horizontal beams	6.5 m	Seepage appeared on the toe slope after the failure	Crest	Demolition of Thorpe Manor in progress. A small slip at the toe slope took place immediately before the failure of the wall.	Not known	May Road was closed for over two months. Had the debris not been stopped by an earth bunk, it would have caused great damage to the Grenville House below.	The wall failed in large sections. From enlarged photographs, it appears to be composed of stabilised fill/concrete with stone facing.
5	Caine Lane	25.8.1976	Dressed block facing with soil cement bounded rubble infill		High	None	None	Not known	One lane of Caine Road closed for ten months. Stones and failure debris rushed into the rear structure of the buildings caused collapse of a canopy.	
6	3-9, Circular Pathway	8.77	Tied face wall	8 m	High (half height i.e. 4 m)	Toe	Demolition of 3-7, Circular Pathway together with the removal of arches between the wall and the building.	Longitudinal cracks along Circular Pathway at the crest of the wall, one immediately between the wall and the lane, with a width of 1¼", the second was near the middle of the road with widths varying from ¾" to 1¼".	Temporary closure of two buildings. Temporary closure of Circular Pathway.	Wall not failed. When the longitudinal cracks were observed to have grown in width and extent, a free draining embankment was built at the toe. This successfully terminated further wall movements.

Table 4.2 - Summary of Case Study of Old Masonry Retaining Walls (Sheet 2 of 2)

Case Number	Location	Failure Date (Time)	Wall Type	Height	Normal Water Table/ Seepage Level	Adjacent Works Immediately Prior to Failure		Signs of Distress	Consequence of Failure	Remarks
						Location	Nature			
7	22, Old Peak Road	11.5.78	Random rubble	5 m (inferred from photo)	Not significant	Old Peak Road at the crest	Newly reinstated telephone trench.	Bulged wall, cracked concrete parapet at the crest of the bulged wall, crack parallel to wall along the middle of the road.	Temporary realignment of Old Peak Road at the crest.	Wall not failed, but discovered by the inspection engineer to have 'bulged'. The age of the bulge was not known, the crack may have been caused by the outward movement of the wall or due to loose backfill to the trench.
8	14-16 Fat Hing St adjacent 48-56 Queen's Rd West	29.7.1978 (23:00)	Tied face wall	3.6 m	Not significant	Crest Toe	Trench perpendicular to the face of the failed wall, excavated for a 4" dia. water pipe. Sheetpiling to the adjacent wall at right angle to the failed section, completed for at least 6 months.	Not known	Temporary closure of right of way at the crest. Demolition of 1 building at the crest.	Only a small amount of soil collapsed with the wall. The exposed soil face stood at steep angles.
9	1-10 Wing Wa Terrace (6-8, Hospital Road)	13.11.78 (1:00)	Random rubble	10.3 m	3 metres, water flowed out near the base of the wall at several locations	Toe	Twelve horizontal drains were installed to draw down groundwater level. Sheetpiling to stabilise the wall, which was terminated prior to the failure after renewed distresses appeared on the wall.	¾" wide crack parallel to and extended for half length of the crest. 'Bulge' developed at 10 ft below crest at the location which later failed.	Tempoary closure of the rear lane at the crest.	The drains were discharging steady flow of water.
10	Jewish Recreation Club, Robinson Rd	3.8.79	Random rubble	3 m	Not significant	None	Recorded	Wall bulged for some period of time.	Not significant.	

Table 4.3 - Case Studies - Weather Conditions at Time of Failure

Case Number	Location	Failure Date	Weather	Rainfall before Failure Date	
				15 days Cummulative before the Event (mm)	24 hrs Cummulative on the Day of the Event (mm)
· 1	St. Joseph's Terrace (18-24A Caine Road)	11:00 16.7.19	(After 2 days of heavy rain)	294	47 (207)
2	Po Hing Fong	17.7.25		217	280
3	10, Castle Rd (I.L. 7976)	19.6.70		104.5	4.5
4	Thorpe Manor 1, May Road	13:45 2.9.73	Typhoon 'Ellen'	336.0	25.2
5	Caine Lane	25.8.76		213.3	448.4
6	3-9, Circular Pathway	8.77	1.8.77 (Typhoon 'Vera') 16.8.77 (a trough of low pressure)	1-15.8.77 141.4 16-30.8.77 23.3	N.A.
7	22, Old Peak Rd	11.5.78		229.7	NIL
8	14-16 Fat Hing St. Adjacent 48-56 Queen's Road West	23:00 29.7.78	Typhoon 'Agnes'	364.9	71
9	1-10, Wing Wa Terrace (6-8 Hospital Road)	1:00 13.11.78		55.7	NIL
10	Jewish Recreation Club Robinson Road	3.8.79	Typhoon 'Hope'	453.4	31.2 (142.4)

Note: ( ) denote rainfalls in the previous day.

Table 5.1 - Wall Parameters of the Standard Retaining Wall Section

Soil Parameter	ϕ'	39°
	c'	0
	δ	20°
Crest Slope Angle		0°
Groundwater		0
Wall Geometry Front face angle Rear face angle		 85° 90°

Table 5.2 - Influence of Wall Parameters on the Required Thickness of Retaining Wall

		Usual Limit of Value	Max. H/B Ratio for No Tension at the Base	Change in Minimum Wall Thickness
Soil Parameter		$\phi' = 35°$, $\delta = ½\phi'$	2.8	+7%
		$\phi' = 40°$, $\delta = ⅔\phi'$	3.3	-9%
Crest Slope Angle		30°	2.5	20%
Groundwater Location		0.5H	2.4	25%
Wall Geomety	Front face angle	75°	Depends on the rear face angle	
	Rear face angle	100°	2.5	20%
		80°	8	-62%

Table 5.3 - Generalised Set of Ground Condition Parameters

Soil Strength	c'	0 kN/m²
	φ'	39°
Soil Density	Bulk	19.0 kN/m³
	Submerged	9.2 kN/m³
Buried Depth of Foundation		0 m
Applied Load Characteristics	Inclination	0.275
	Eccentricity	0.156 m
Distance from Crest of Slope		0 m
Wall Geometry	Height	10 m
	H/B	3
Calculated Critical Toe Slope Angle	Dry foundation	29.6°
	Submerged foundation	20.7°

Table 5.4 - Influence of Ground Condition Parameters on the Critical Toe Slope Angle to a Retaining Wall

	Usual Limit of Value	Critical Toe Slope Angle		Change in Critical Toe Slope Angle (%)	
		Submerged	Dry	Submerged	Dry
Height of Wall	1 - 12 m	20.7°	29.6°	0%	0%
Soil Shear Strength	35°	8°	22°	-61%	-26%
Buried Depth of Wall	1 m	26°	32.6°	25%	10%
Distance of Wall from Crest of Toe Slope	2 m	34.6°	39° (max.)	67%	max.

Table 6.1 - Allowable Compressive Strength of Masonry Walls

Mortar Strength	Ashlar	Coarse Ashlar	Squared Rubble	Random Rubble
2.5	12.5	8.7	7.5	1.4
1.0	11.7	8.2	7.0	1
dry packed	10.1	7.0	6.0	0.3
Note : All units are in MPa.				

Table 6.2 - Shear Strength of Masonry Wall (Movement Along Joints)

Mortar Designation	Mortar Strength	Normal Stresses				
		0	0.50	0.100	0.150	0.200
I	11	0.083	0.113	0.143	0.173	0.203
II	4.5	0.083	0.113	0.143	0.173	0.203
III	2.5	0.083	0.113	0.143	0.173	0.203
IV	1.0	0.036	0.066	0.096	0.126	0.156
Note : All units are in MPa.						

Table 6.3 - Parameters for Stress Analysis of Gravity Retaining Walls

Height/base width		= 3
K_a, Coeff. of active pressure (assume $\phi' = 40°$)		= 0.2
γ_b, Bulk density of soil		= 20 kN/m³
γ_m, Bulk density of masonry		= 22 kN/m³
δ, Frictional angle between the wall and the backfill		= 20°
μ, Coeff. of friction at the base of the wall ($\tan \frac{2}{3} \times 40°$)		= 0.5
	Wall A	Wall B
Groundwater	Dry	Half height
F.O.S. vs Sliding	2.24	1.46
F.O.S. vs Overturning	2.88	1.35

Table 6.4 - Allowable Height of Different Types of Masonry Retaining Wall to Avoid Compression Failure

Mortar Strength (MPa)	Types of Wall			
	Ashlar	Coarse Ashlar	Squared Rubble	Random Rubble
2.5	208m	145m	125m	23m
1.0	195m	137m	117m	17m
dry packed	168m	116m	100m	5m

Table 7.1 - Chemical Tests for Nature of Seepage Water

Fresh Water	Sea Water	Sewage
Fluorine	Sodium chloride	Ammoniacal nitrogen
Residual chlorine	Conductivities	Oxygen absorption

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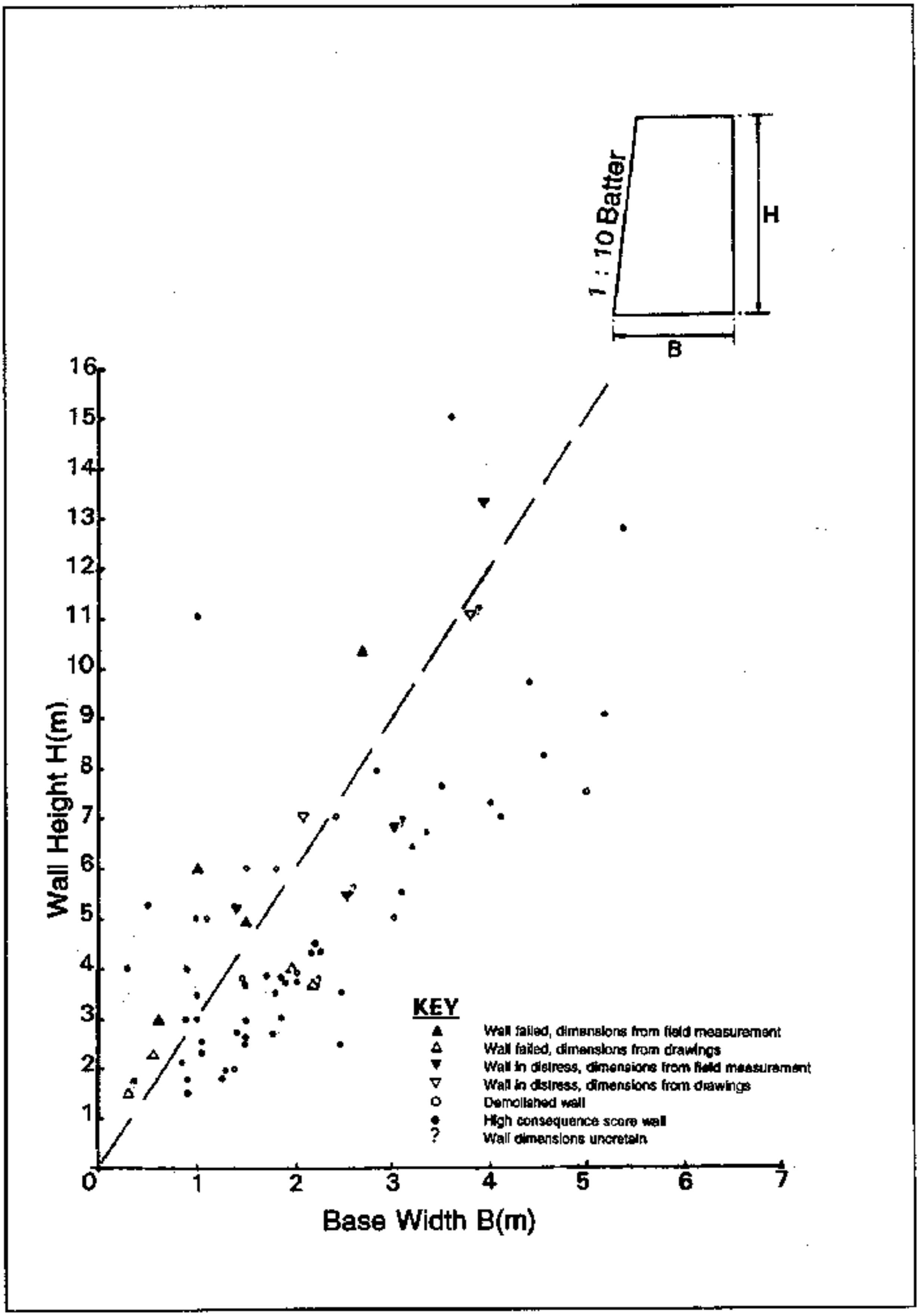


Figure 2.1 - Survey of Old Retaining Walls by GCB - Height/Base-width Plot

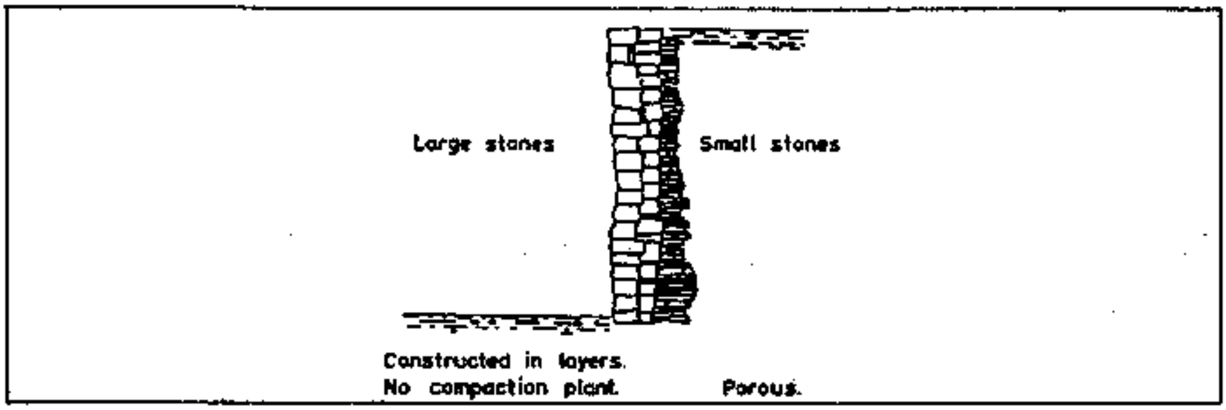


Figure 3.1 - Victorian Stone Retaining Walls in the Yorkshire Region

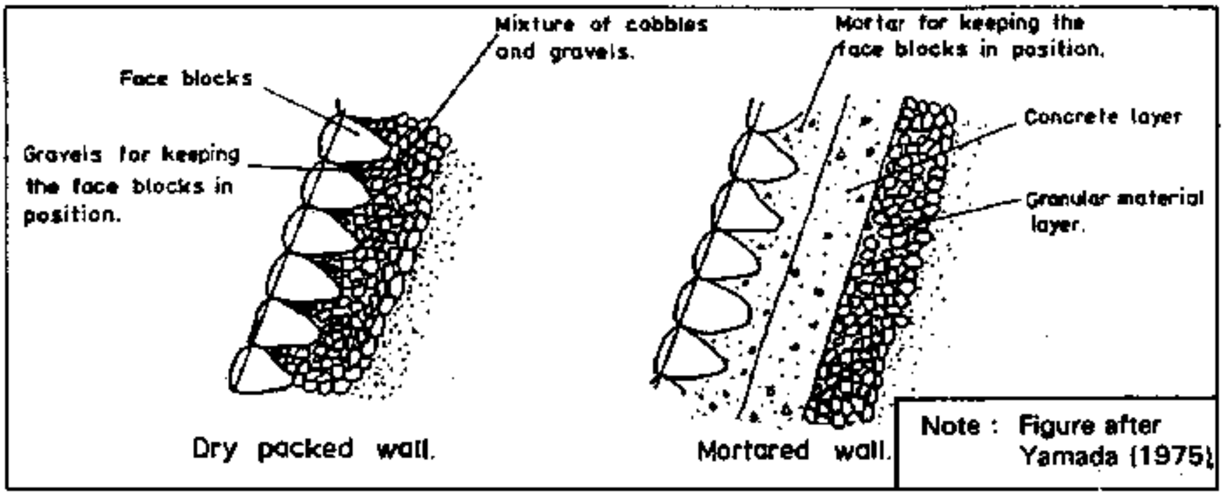


Figure 3.2 - Typical Section of Japanese Stone Masonry Retaining Walls

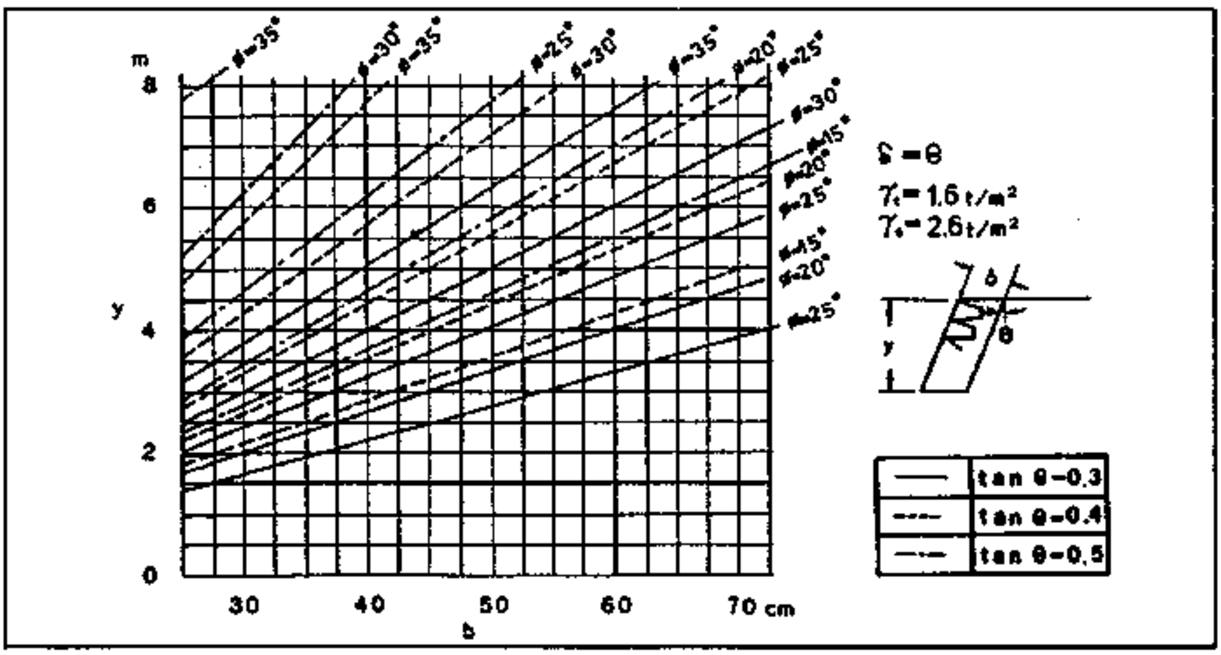
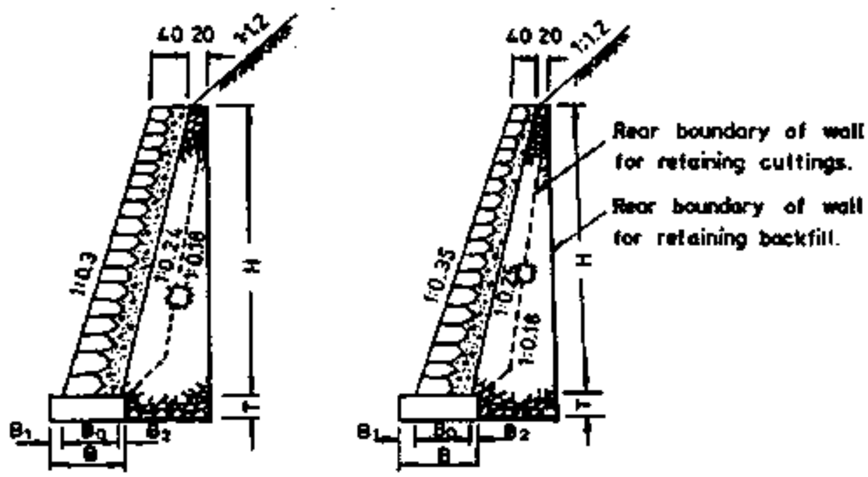


Figure 3.3 - Thickness of Japanese Stone Masonry Retaining Walls

Note : Figure after Yoshimato (1967).

Figure 3.4 - 'Arrow Feather' Bond Pattern of Face Blocks of Japanese Masonry Retaining Walls



(a)

H (m)	4.0	3.5	3.0	2.5	2.0
B (m)	0.84	0.81	0.78	0.75	0.72
B_0(m)	0.64	0.61	0.58	0.55	0.52
B_1(m)	0.15	0.15	0.15	0.15	0.15
B_2(m)	0.05	0.05	0.05	0.05	0.05
T (m)	0.30	0.30	0.30	0.30	0.30

(b)

H (m)	6.0	5.0	4.0	3.0	2.0
B (m)	1.20	1.10	1.00	0.90	0.80
B_0(m)	1.00	0.90	0.80	0.70	0.60
B_1(m)	0.15	0.15	0.15	0.15	0.15
B_2(m)	0.05	0.05	0.05	0.05	0.05
T (m)	0.30	0.30	0.30	0.30	0.30

Note : Figure after Kim (1975).

Figure 3.5 - Typical Section of Masonry Retaining Walls Recommended by Ministry of Construction, Republic of Korea

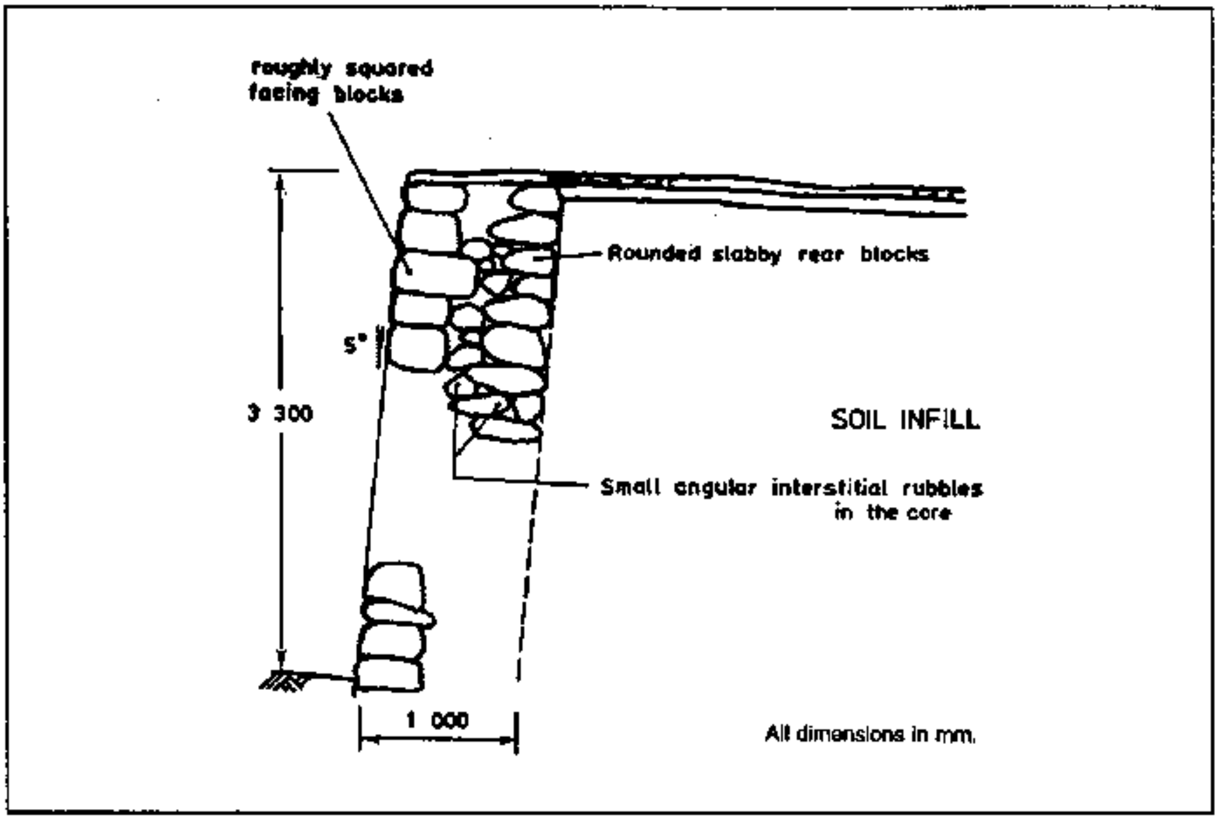


Figure 3.6 - Section of the Stone Rubble Retaining Walls at Tung Lung Fort

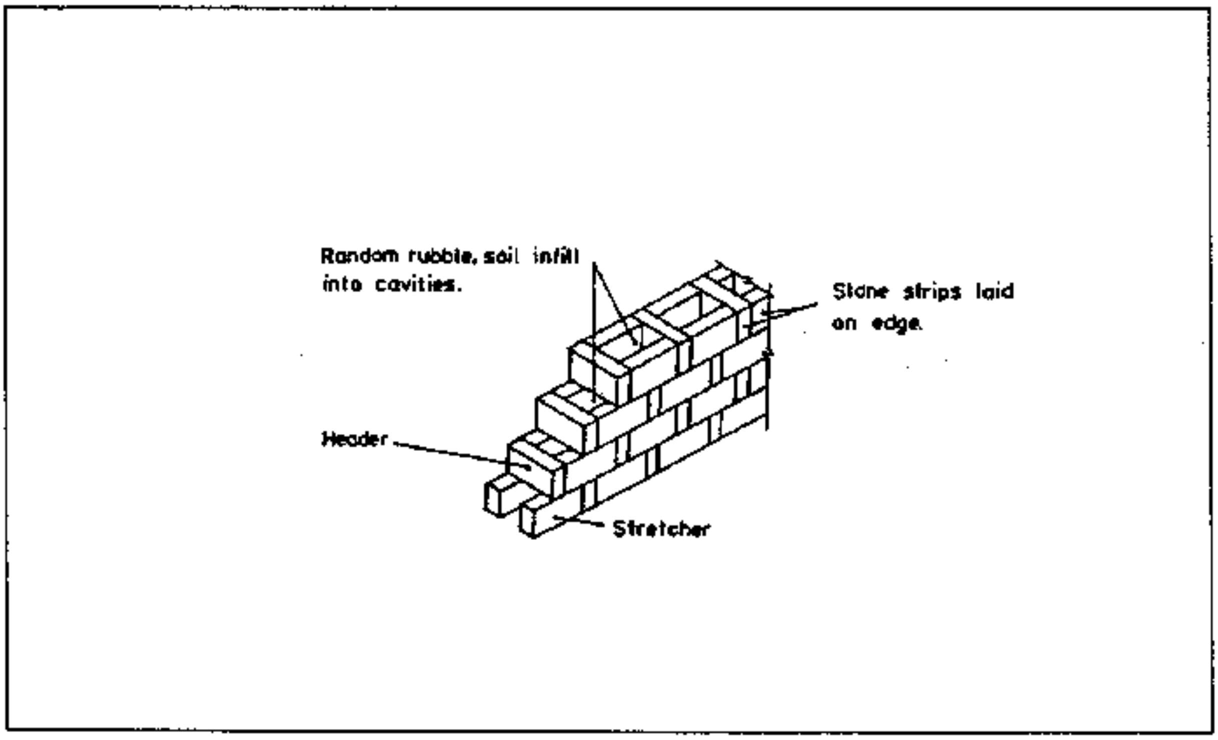


Figure 3.7 - 'Box-bonded' Masonry Wall

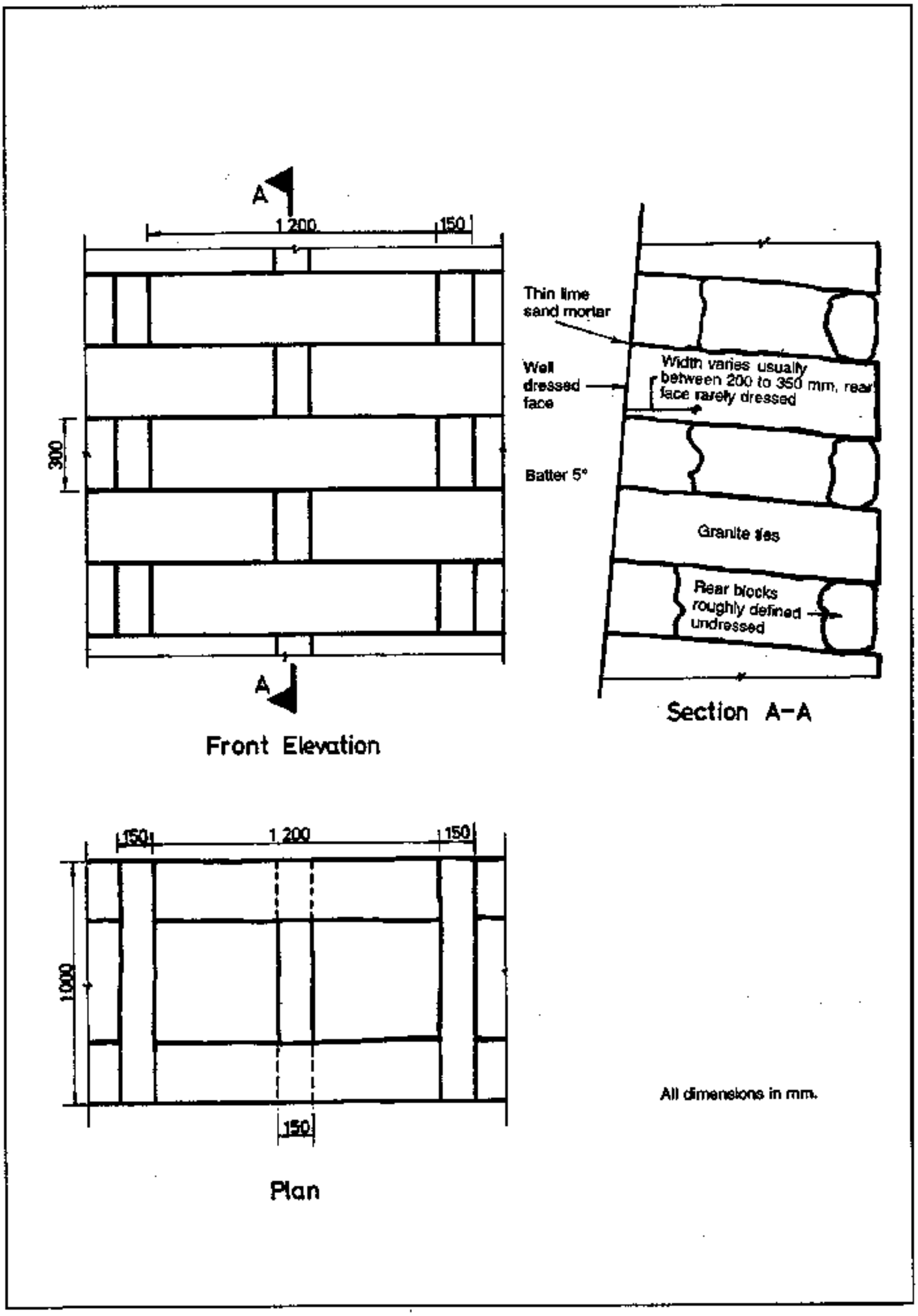


Figure 3.8 - Tied Face (Retaining) Walls

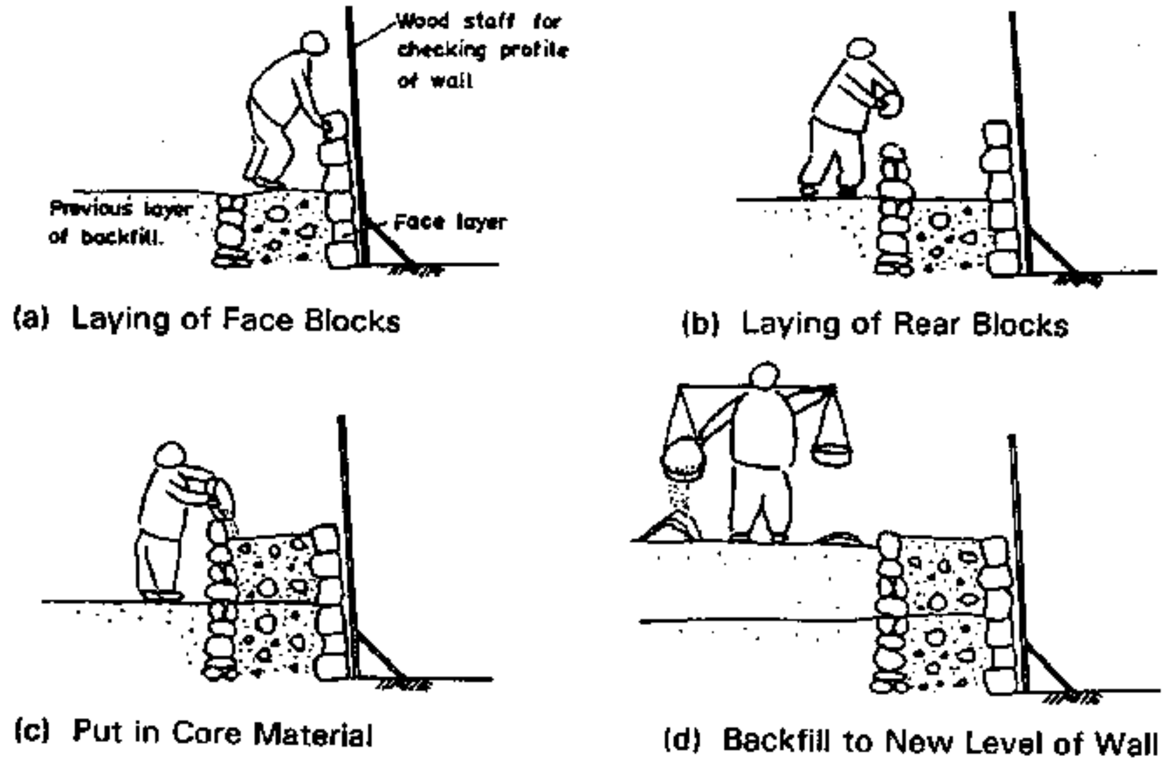


Figure 3.9 - Method of Construction of Old Stone Rubble Retaining Walls

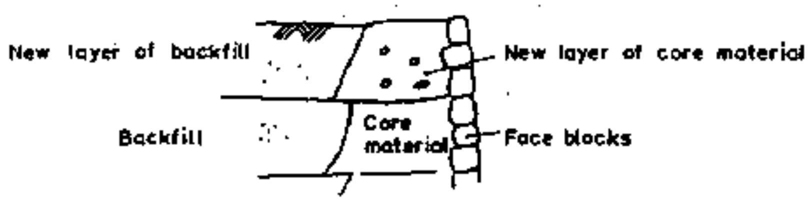


Figure 3.10 - Stone Rubble Retaining Walls without Rear Blocks

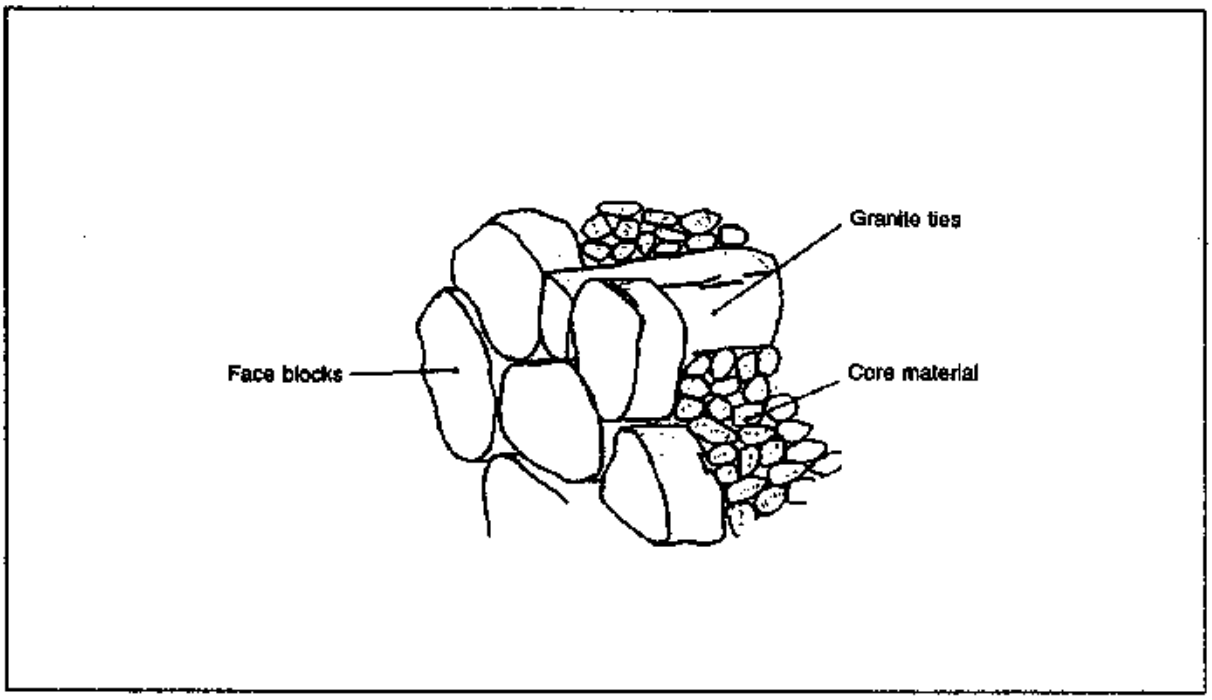


Figure 3.11 - Tied Stone Rubble Retaining Walls

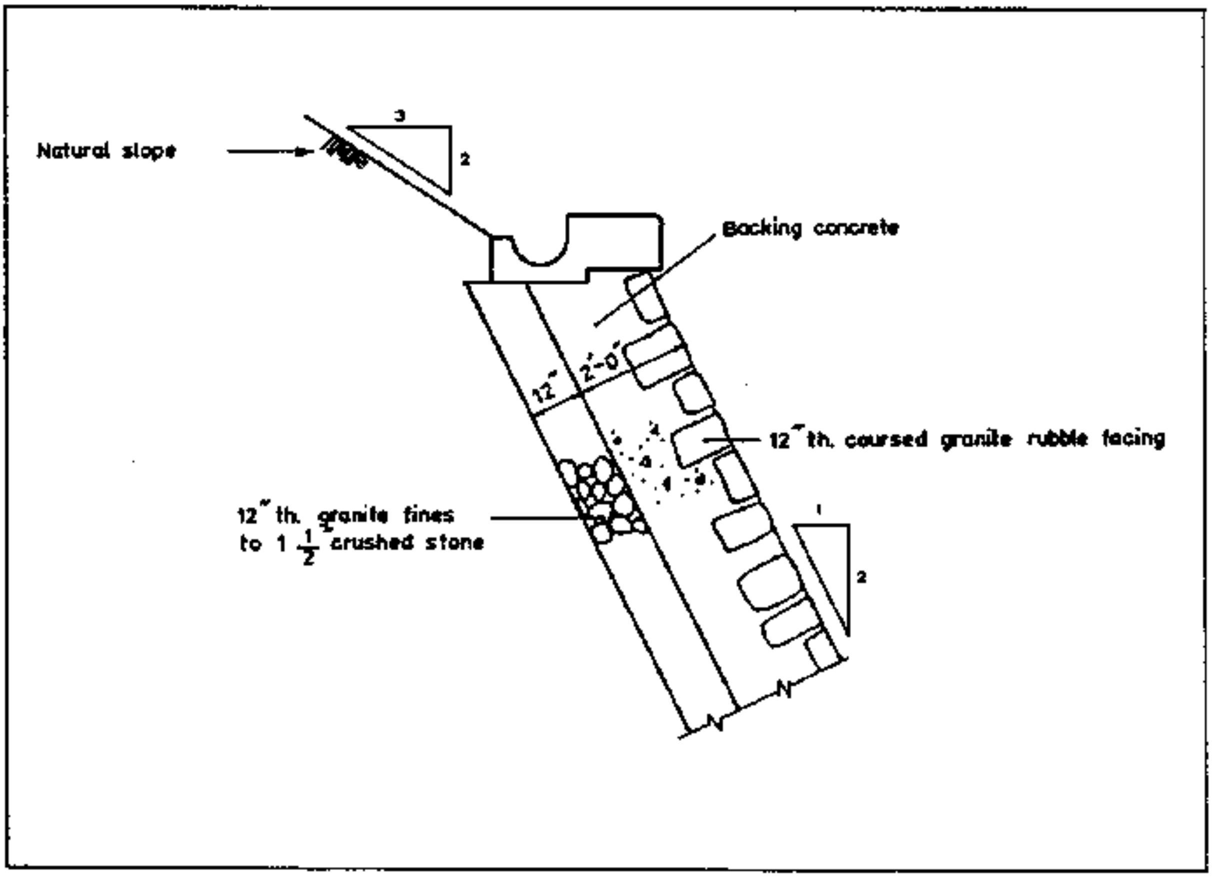


Figure 3.12 - Section of Stone Pitching at Slope 11SW-B/CR16

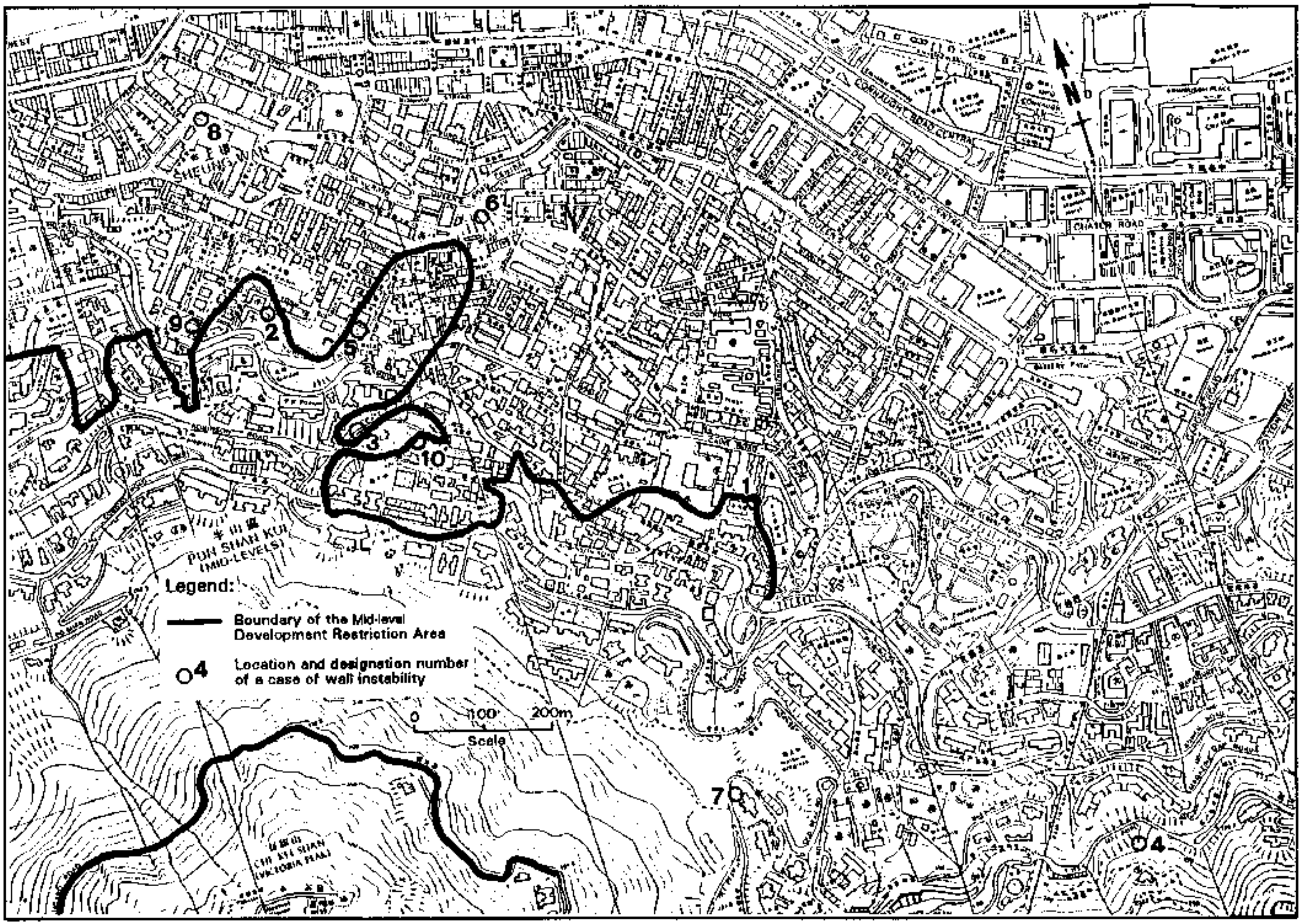
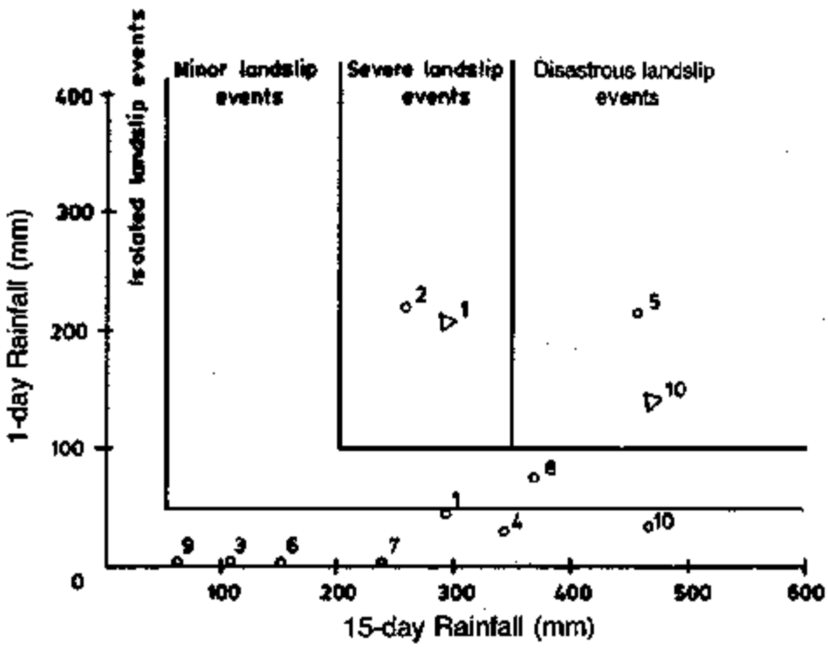


Figure 4.1 - Case Studies - Locations of the Walls



Legend :

- ∘ The 1-day rainfall corresponds to that occurred on the day when the failure of wall took place
- ▷ The 1-day rainfall corresponds to that occurred on the previous day before the failure of wall took place
- 1 Denotes case number referred to in Table 4.3

Note : Figure after Lumb (1975).

Figure 4.2 - Case Studies - Rainfall Condition at Time of Failure

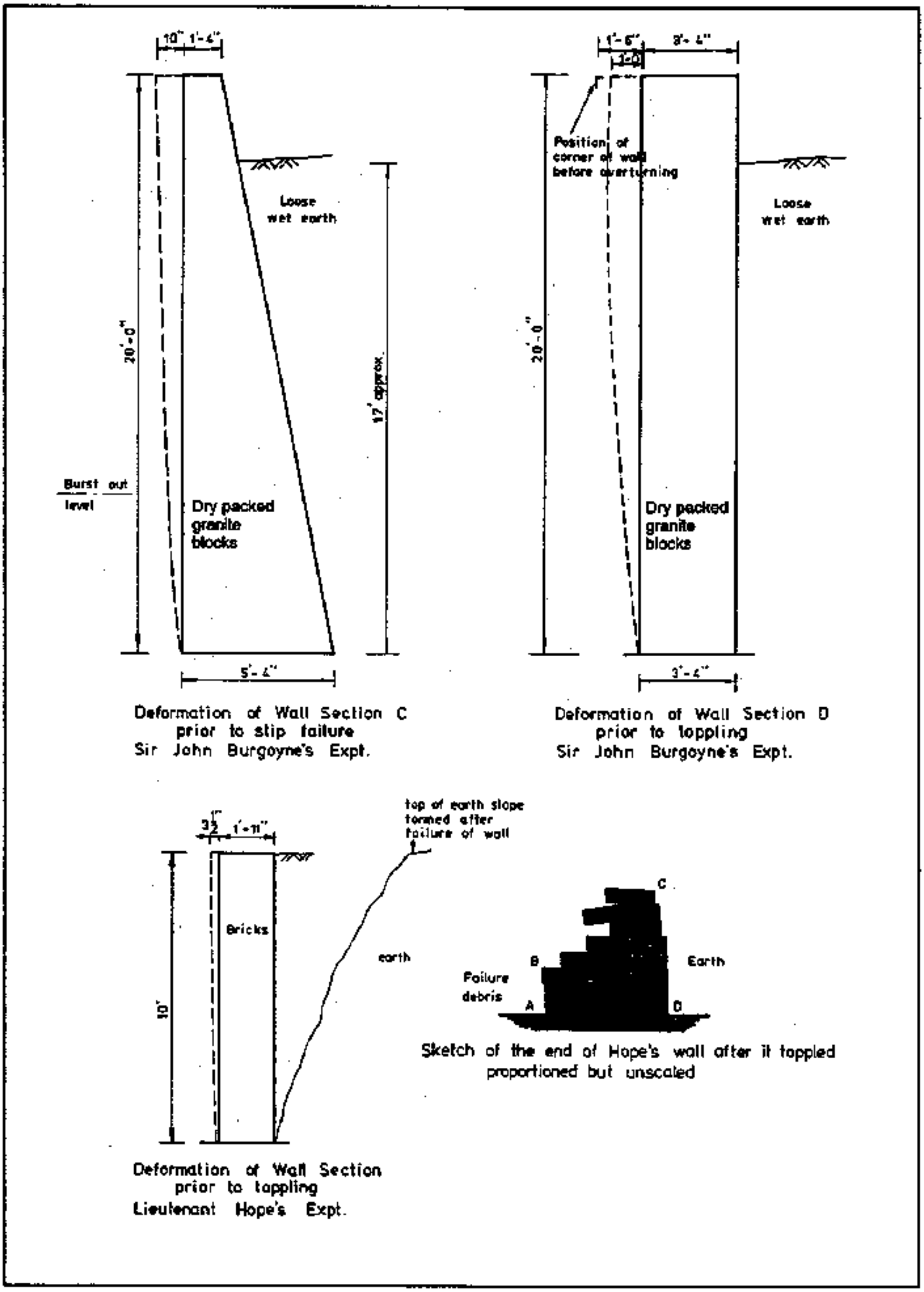


Figure 4.3 - Deformation of Masonry Retaining Walls Prior the Failures - Experiments by Burgoyne and Hope

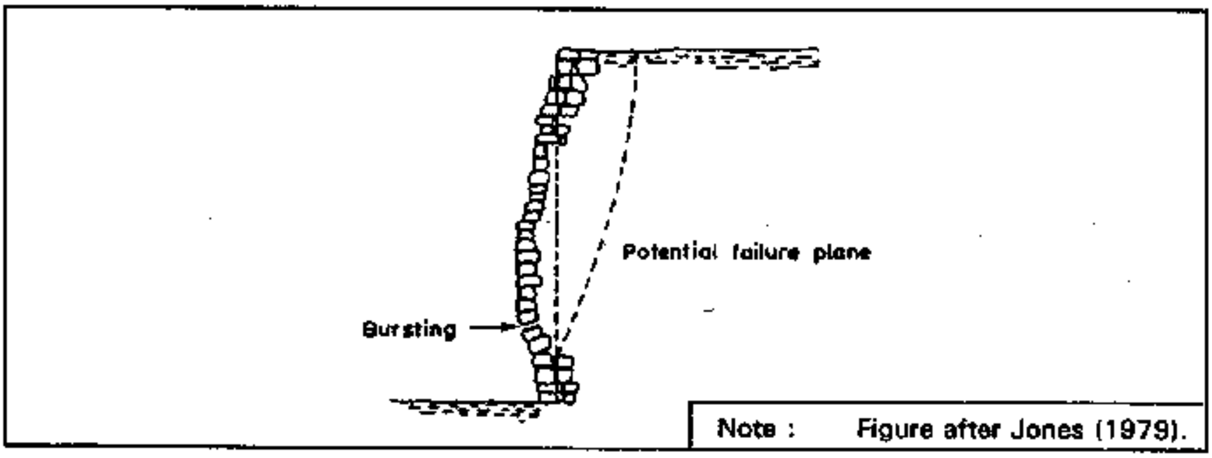


Figure 4.5 - Mode of Failure of Victorian Stone Retaining Walls

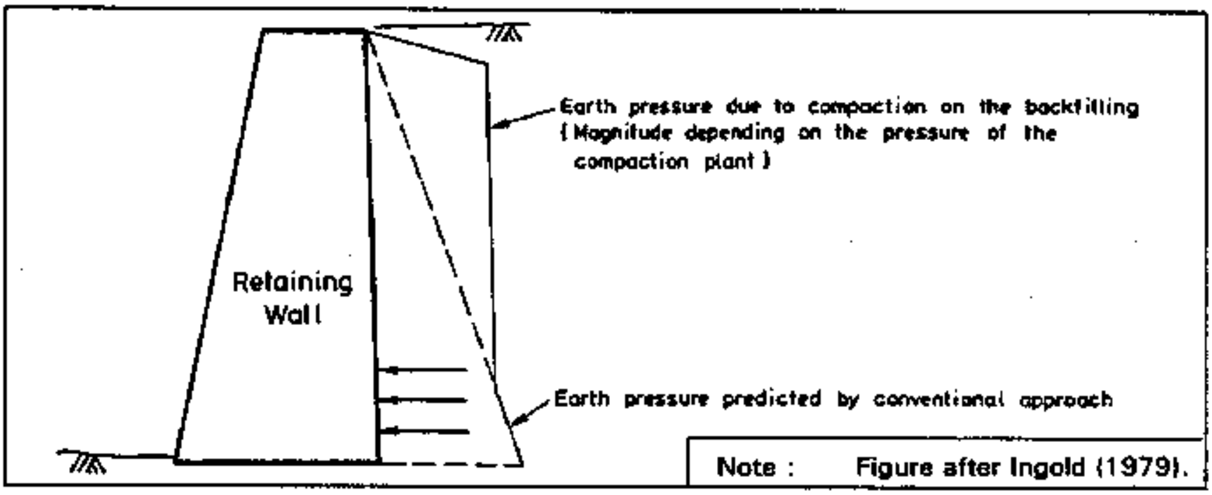


Figure 5.1 - Pressure on Retaining Wall with Compacted Backfill

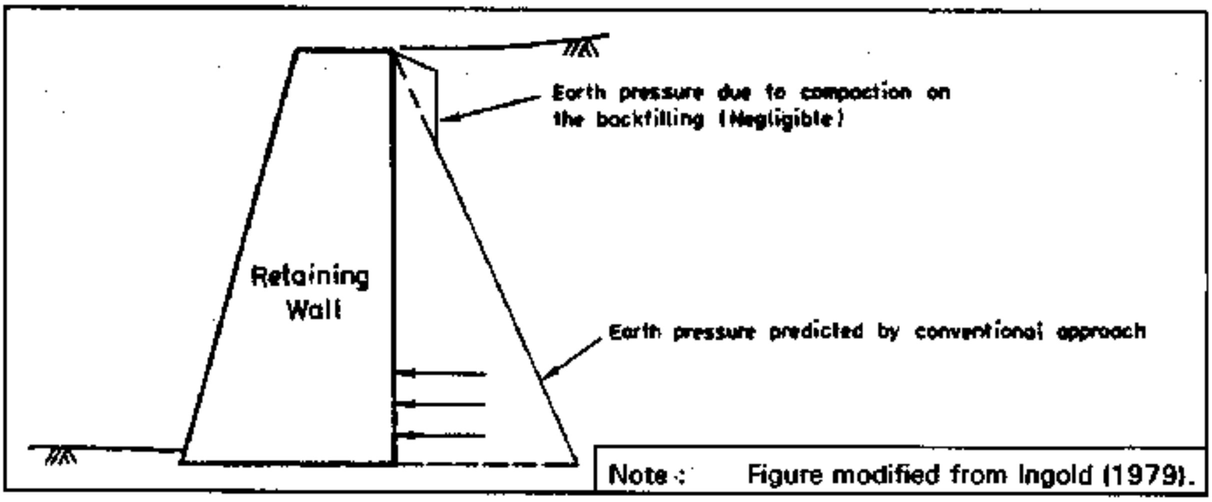
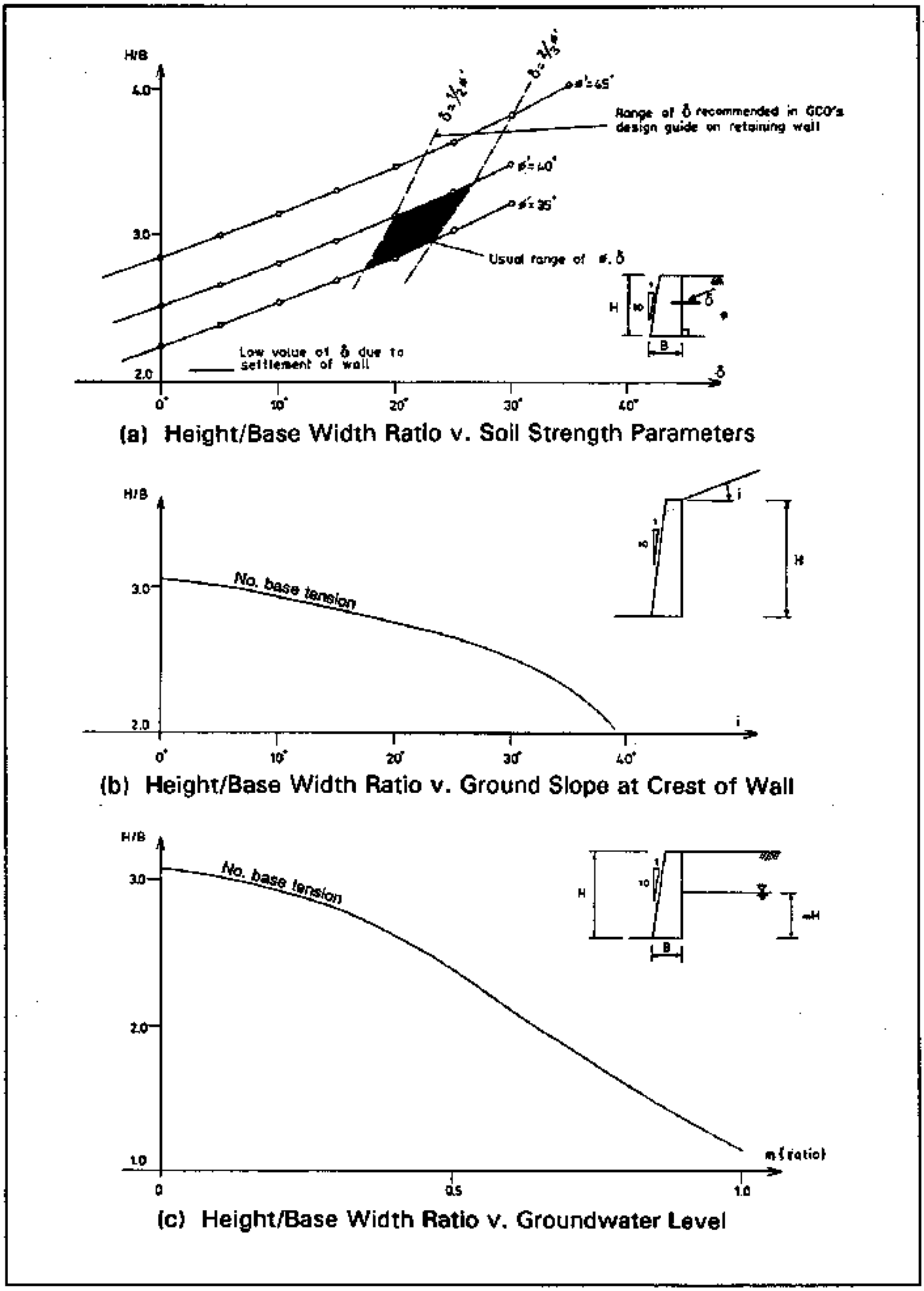


Figure 5.2 - Pressure on Old Masonry Retaining Walls



(a) Height/Base Width Ratio v. Soil Strength Parameters

(b) Height/Base Width Ratio v. Ground Slope at Crest of Wall

(c) Height/Base Width Ratio v. Groundwater Level

Figure 5.3 - Sensitivity of Height/Base-width Ratio Against Different Parameters of the Retained Ground

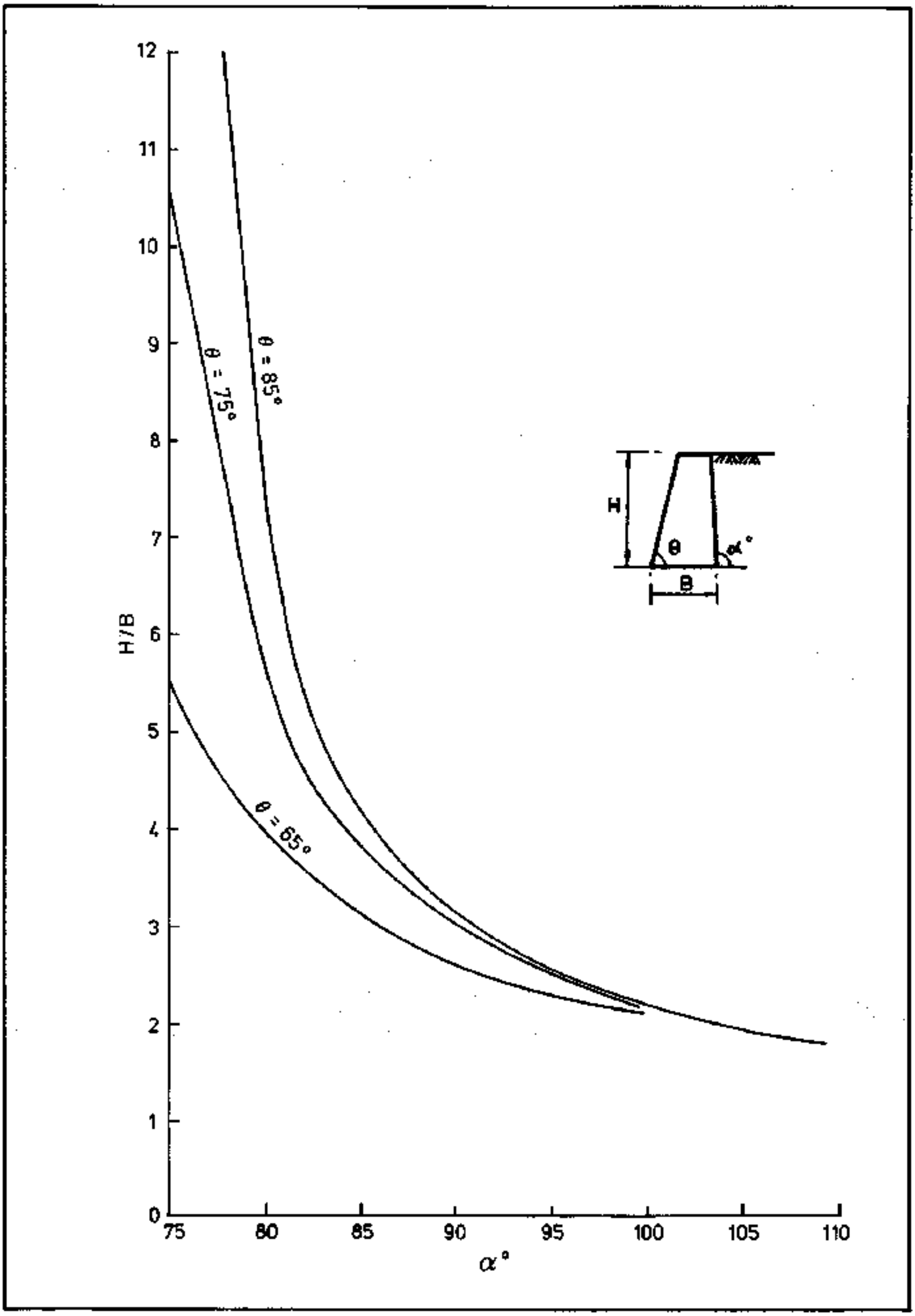


Figure 5.4 - Sensitivity of Height/Base-width Ratio Against Wall Geometries

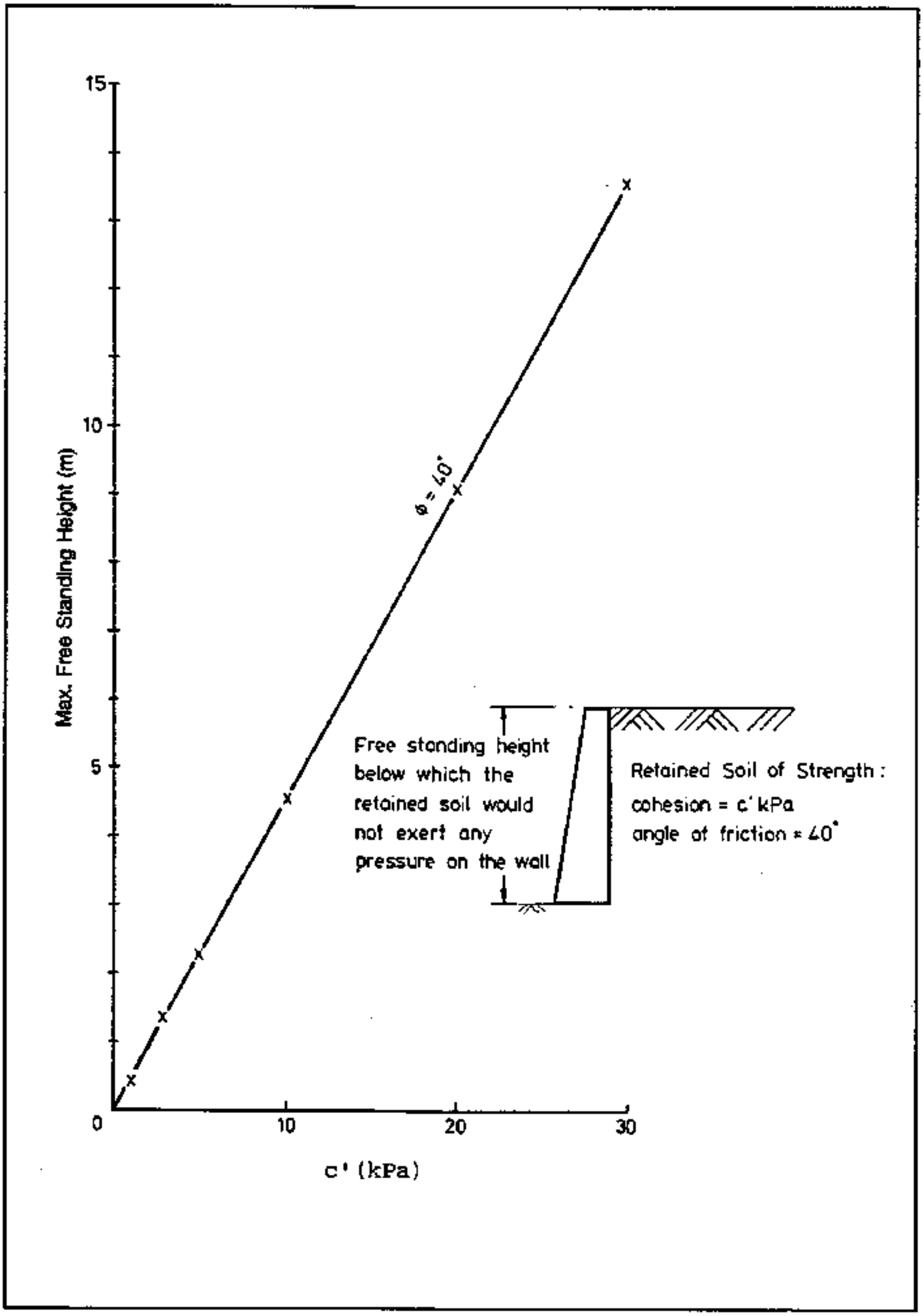
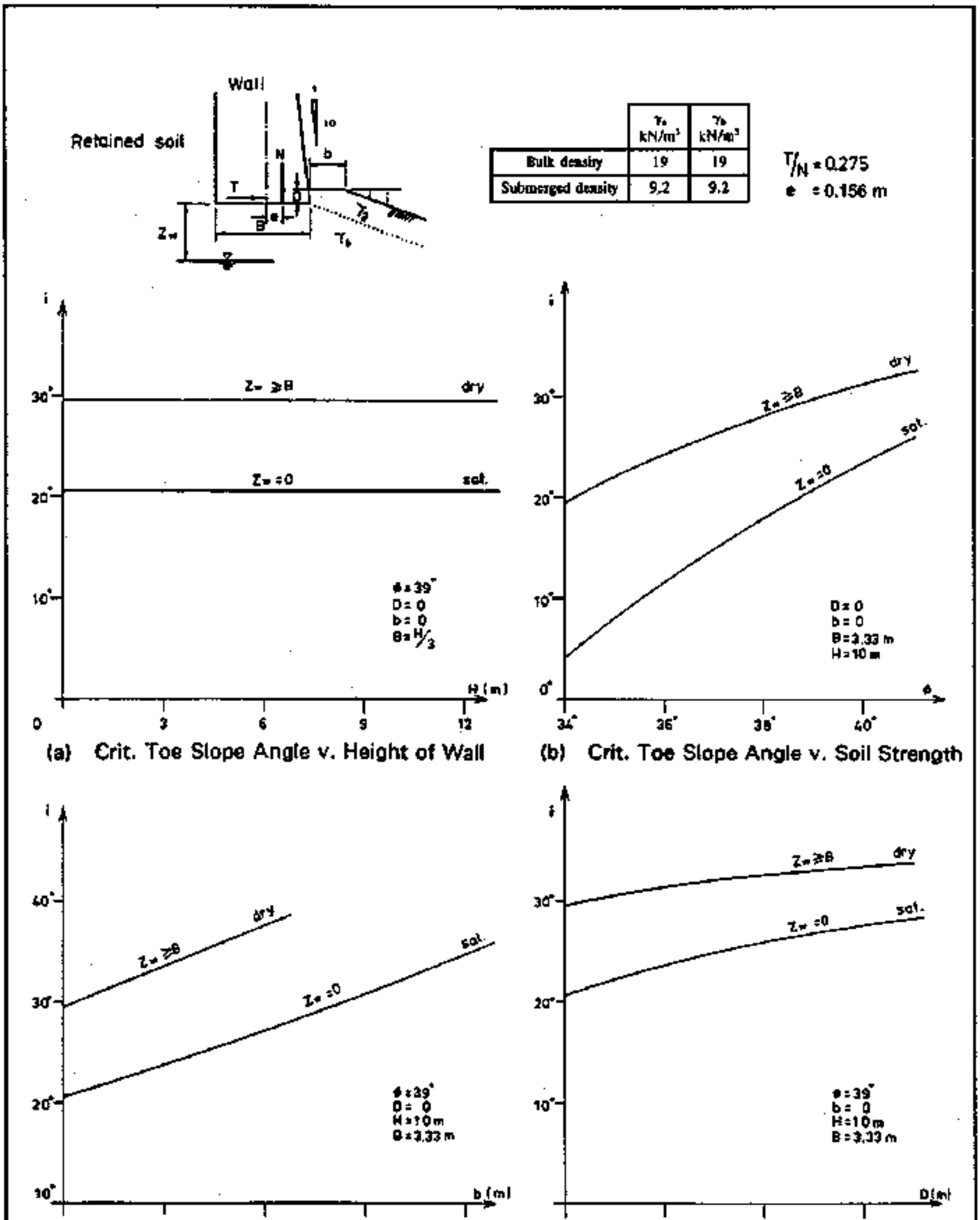


Figure 5.5 - Effect of Cohesion of Retained Soil on Stability of Retaining Walls



	γ_s kN/m³	γ_b kN/m³
Bulk density	19	19
Submerged density	9.2	9.2

(a) Crit. Toe Slope Angle v. Height of Wall

(b) Crit. Toe Slope Angle v. Soil Strength

(c) Crit. Toe Slope Angle v. Distance from Crest of Slope

(d) Crit. Toe Slope Angle v. Buried Depth of Foundation

Figure 5.6 - Sensitivity of Critical Toe Slope Angle Against Ground Conditions Variations

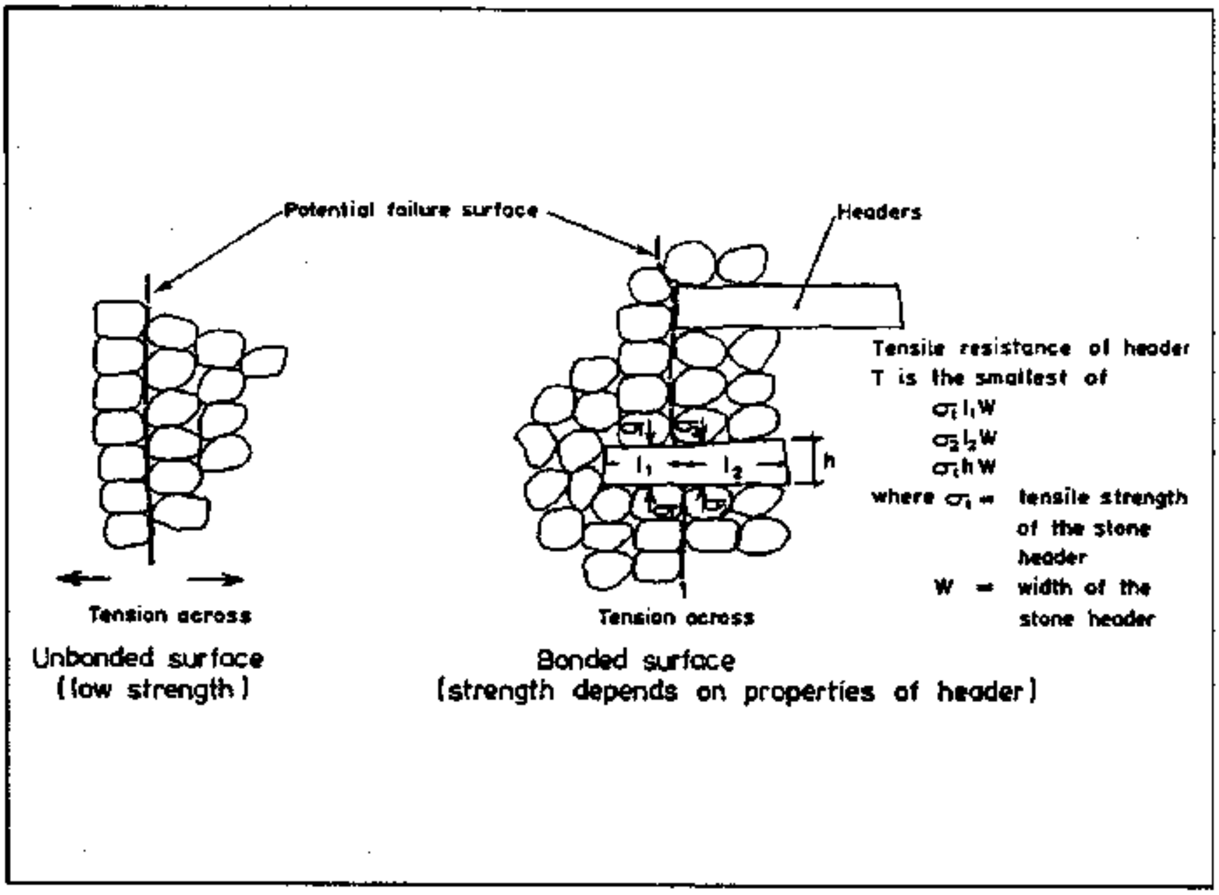


Figure 6.1 - Tensile Strength of Dry Packed Masonry

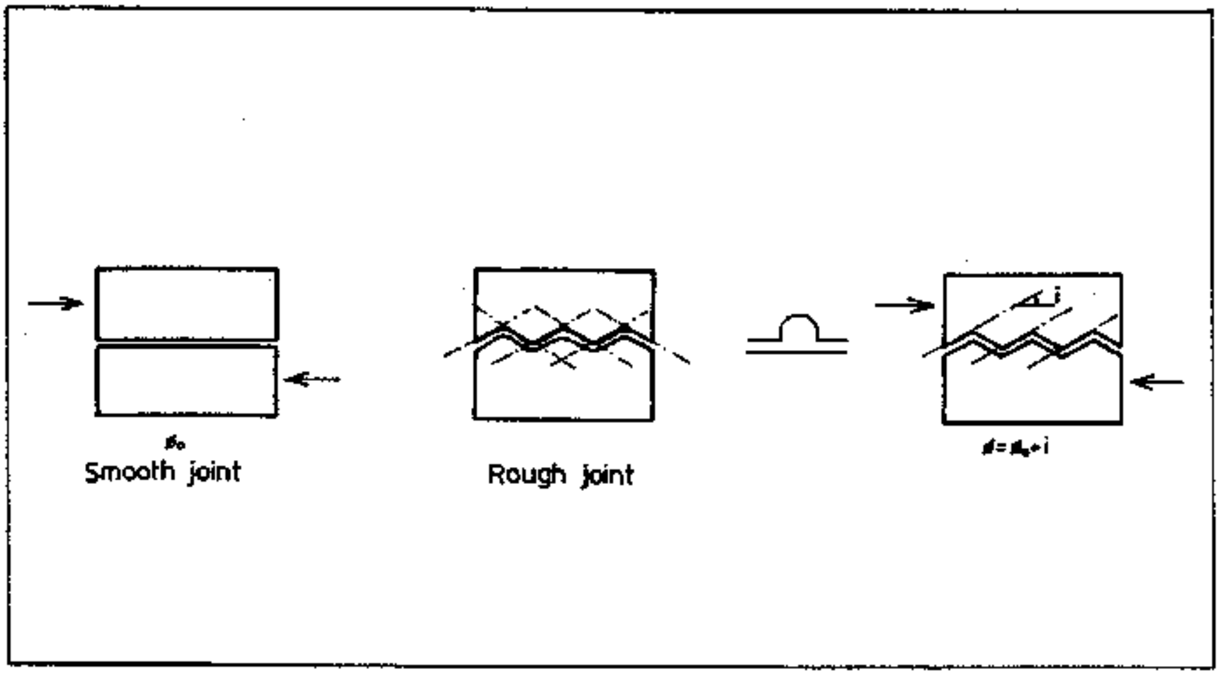


Figure 6.2 - Shear Strength of Rock Joints

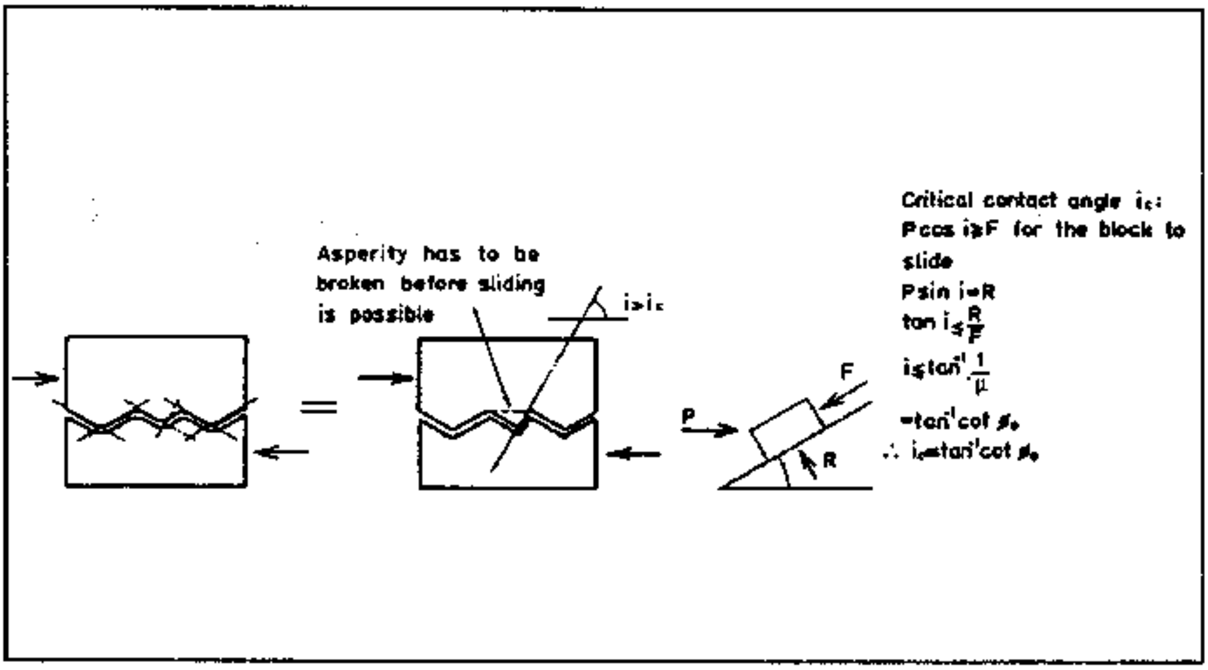
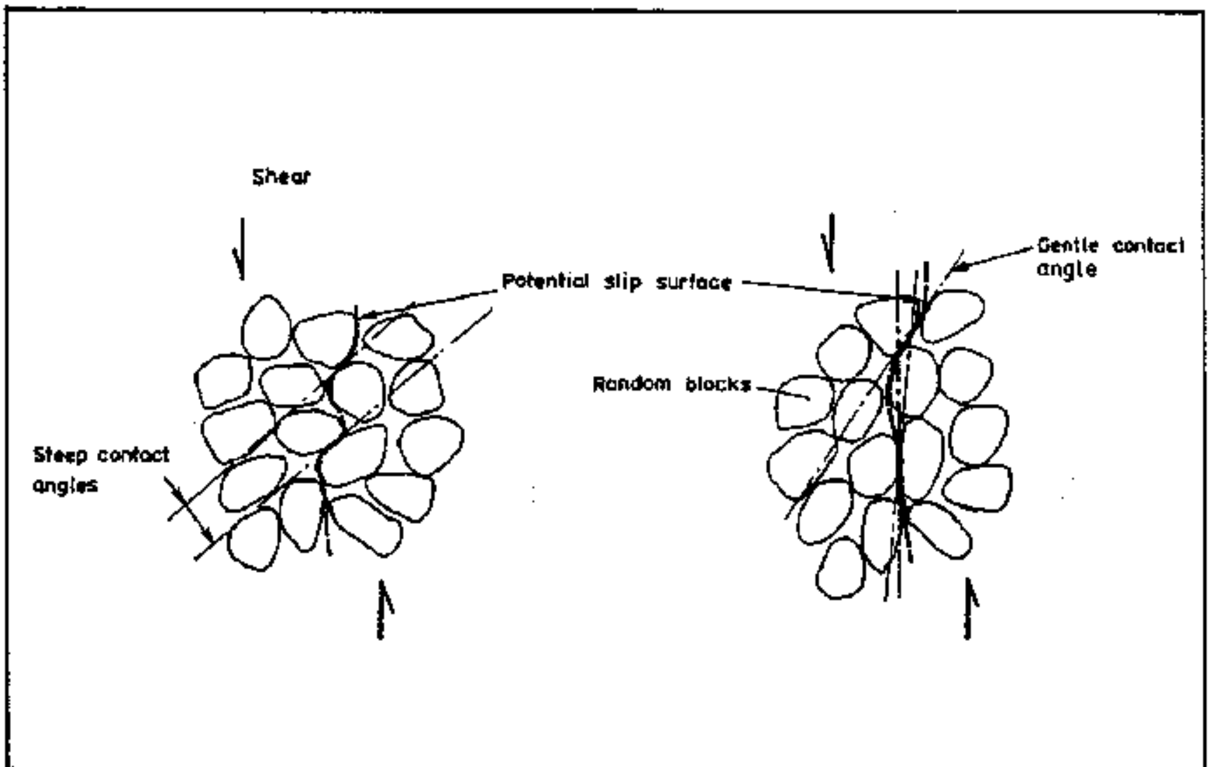
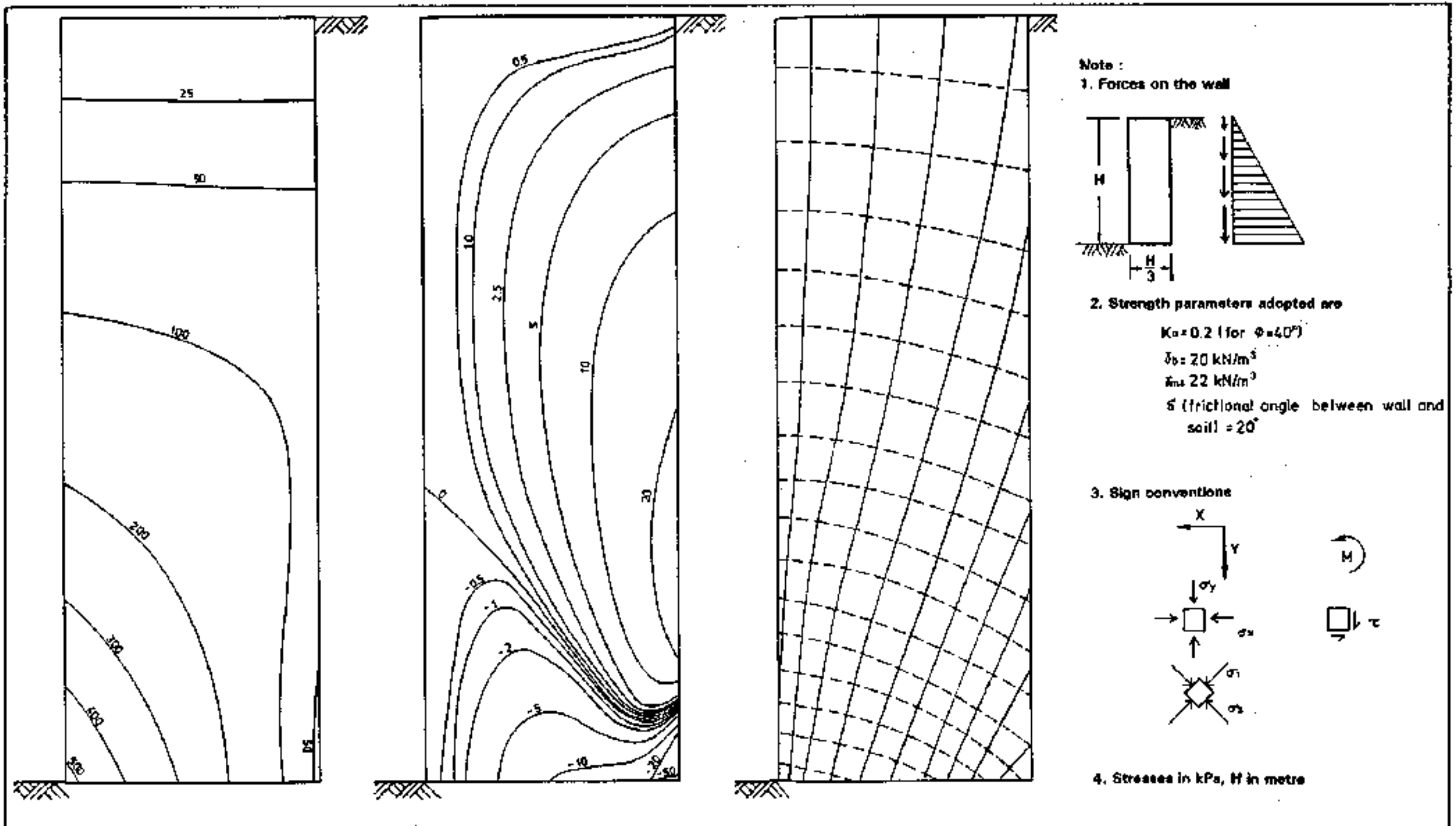


Figure 6.3 - Apparent Cohesion due to Steep Local Surface Contact
Across Irregular Joint Planes



Note : Shear movement is not possible unless the shear stress is large enough to cause reorientation of the blocks to contact at more favourable sliding angle.

Figure 6.4 - Apparent Cohesion due to Interlocking Blocks of a Masonry



(a) Magnitude of the major principal stress $\left(\frac{\sigma_1}{0.1H}\right)$ (b) Magnitude of the minor principal stress $\left(\frac{\sigma_3}{0.1H}\right)$ (c) Principal stress trajectory

Figure 6.5 - Principal Stress Distribution (No Groundwater Case)

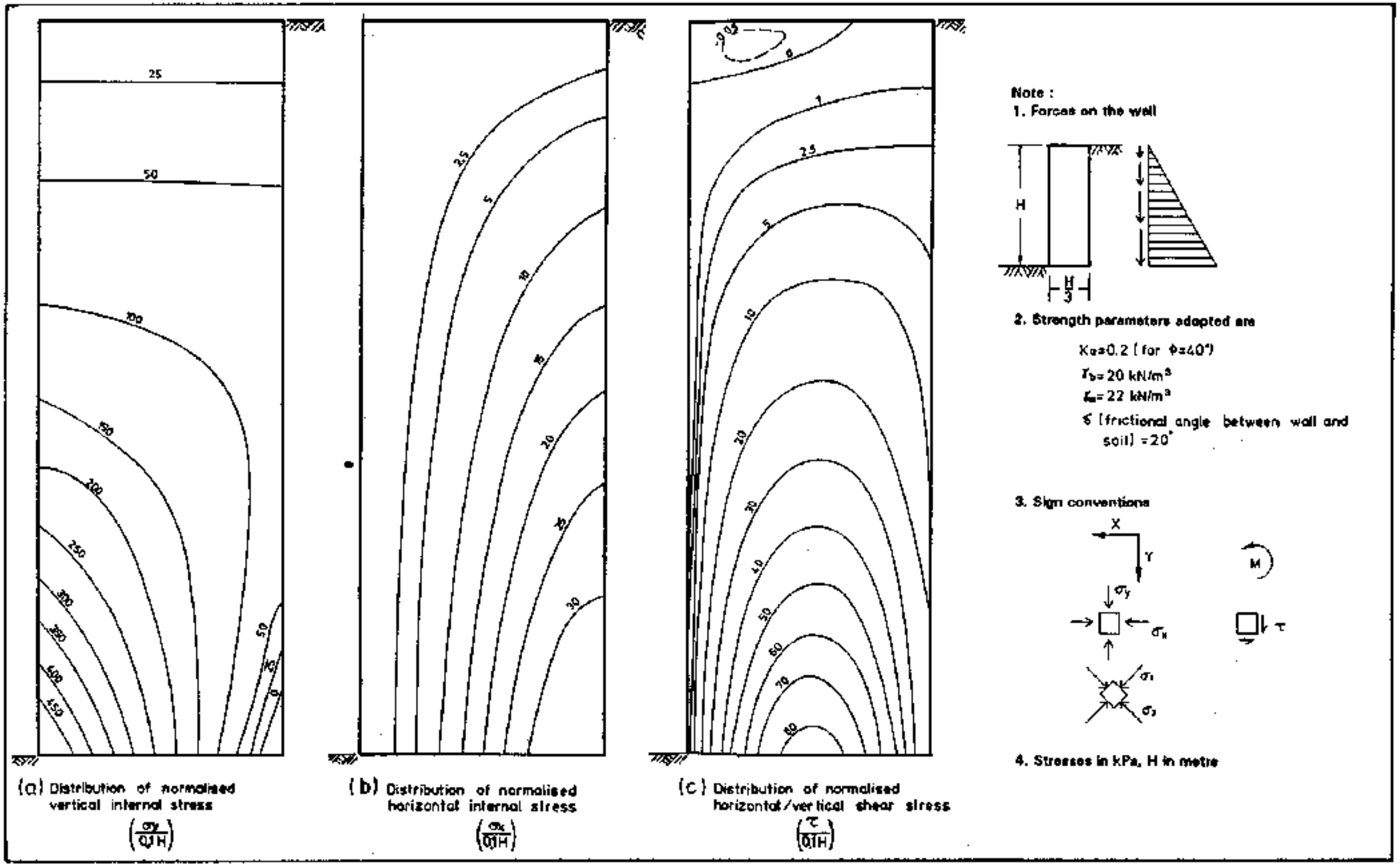


Figure 6.6 - Orthogonal Stress Distribution (No Groundwater Case)

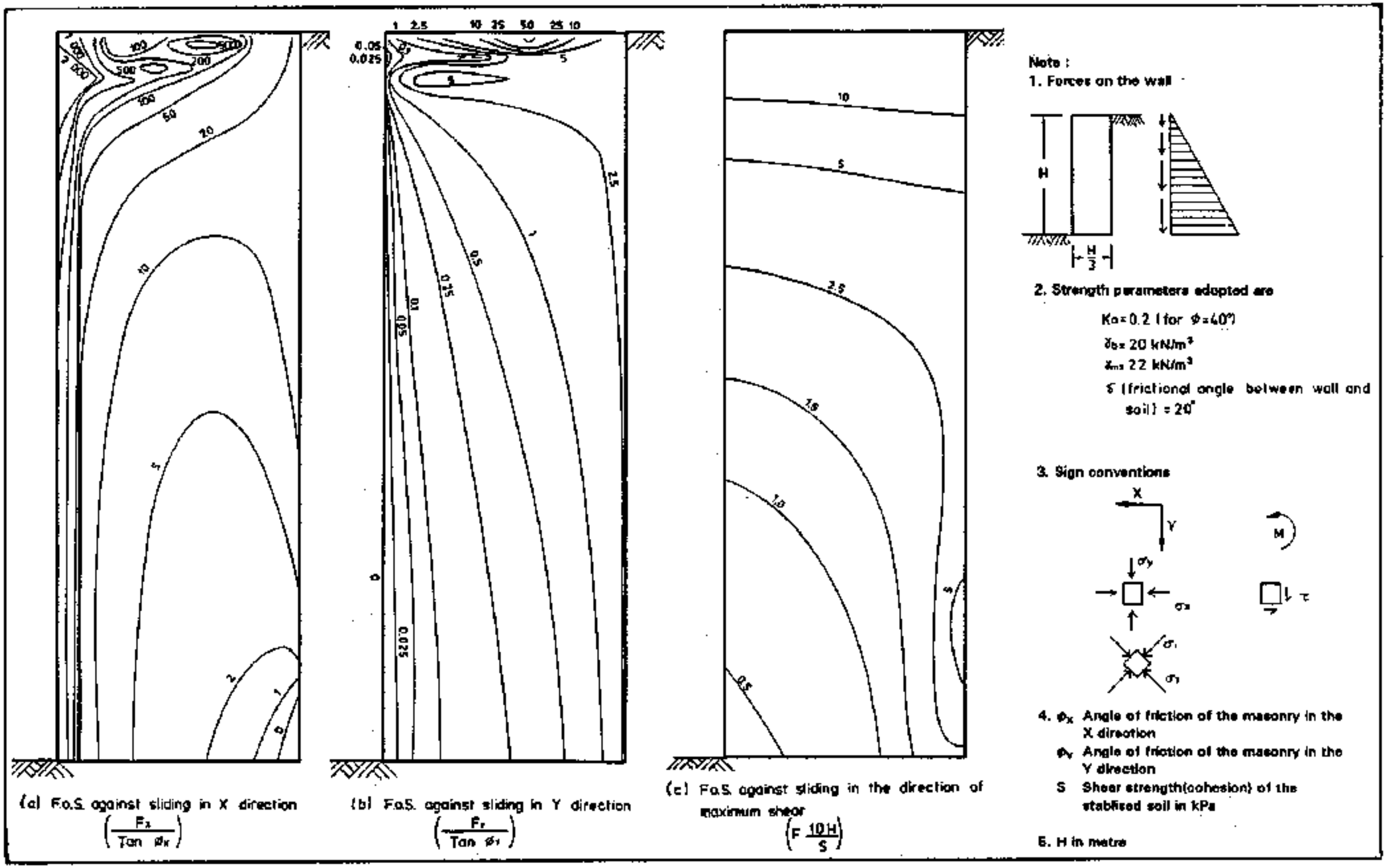


Figure 6.7 - Factors of Safety Against Internal Shearing (No Groundwater Case)

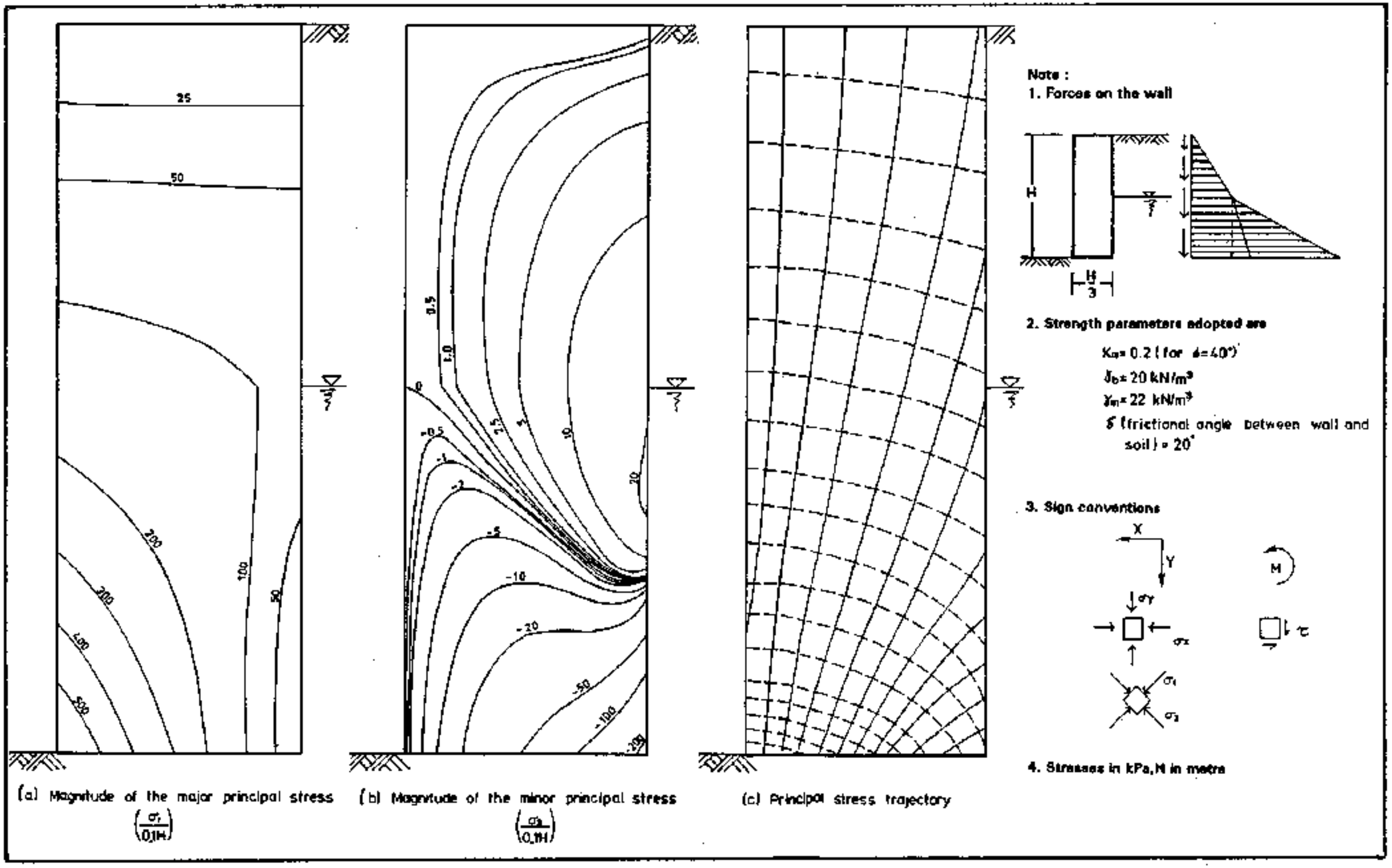


Figure 6.8 - Principal Stress Distribution (Groundwater to Half Height)

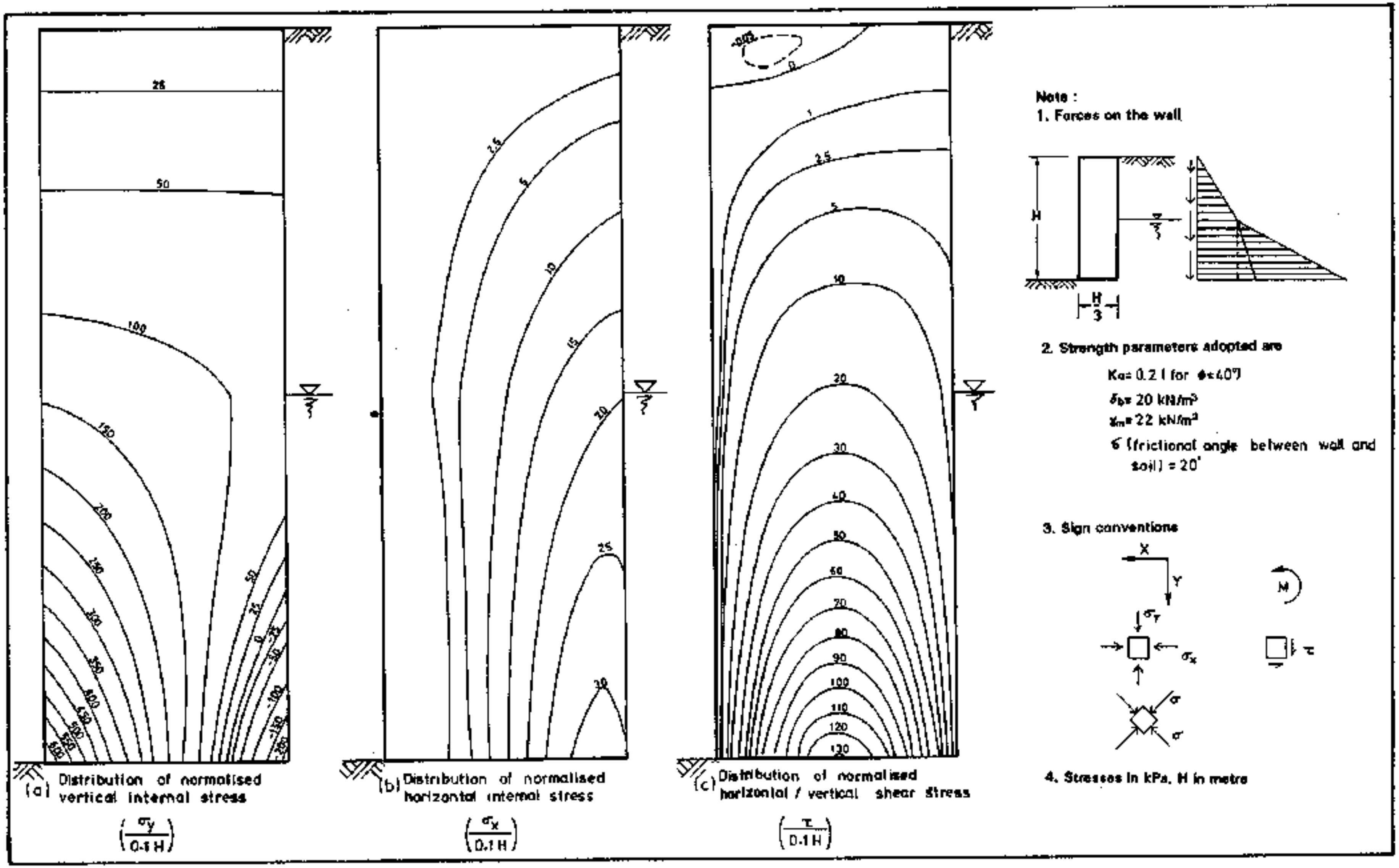


Figure 6.9 - Orthogonal Stress Distribution (Groundwater to Half Height)

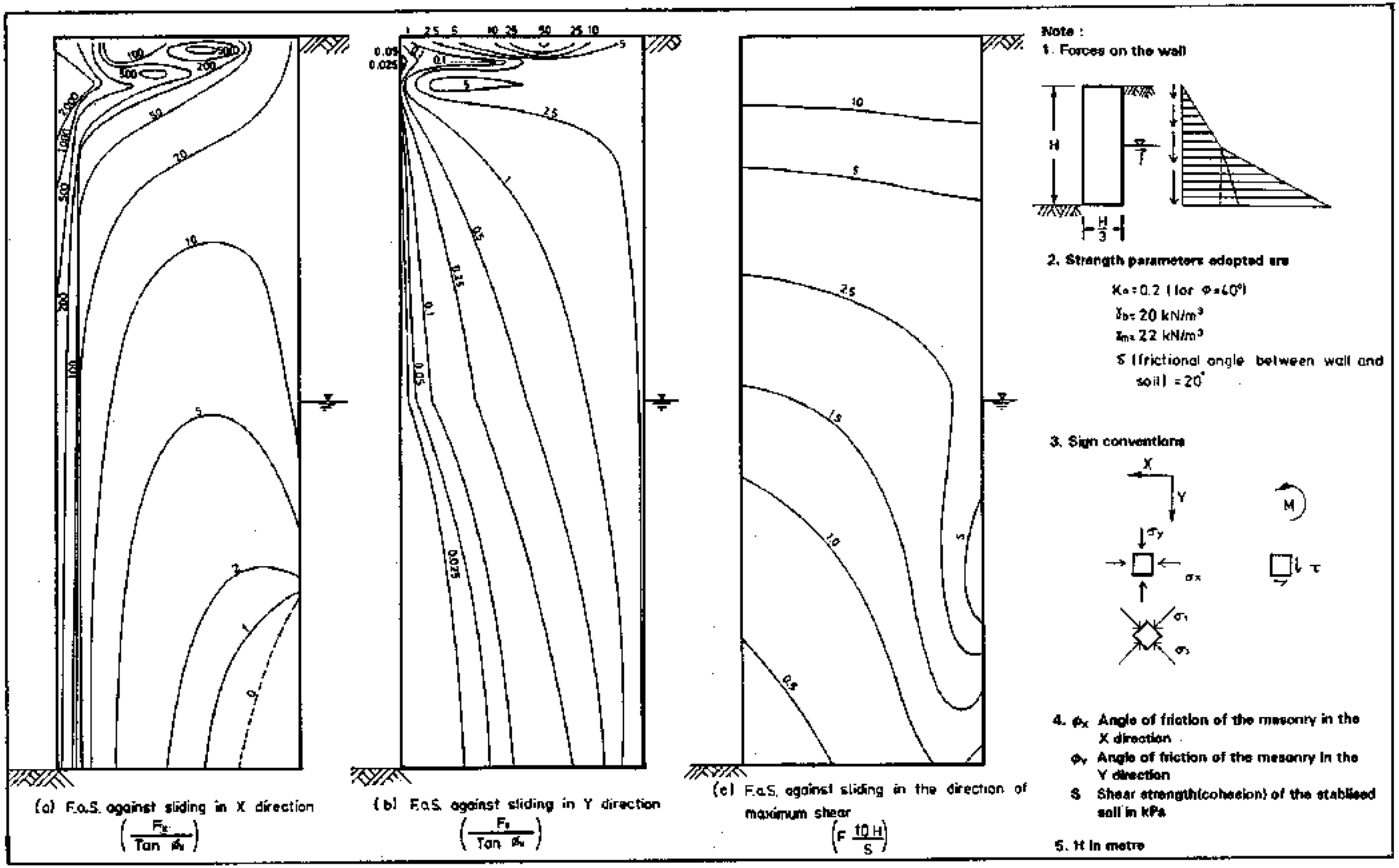


Figure 6.10 - Factors of Safety Against Internal Shearing (Groundwater to Half Height)

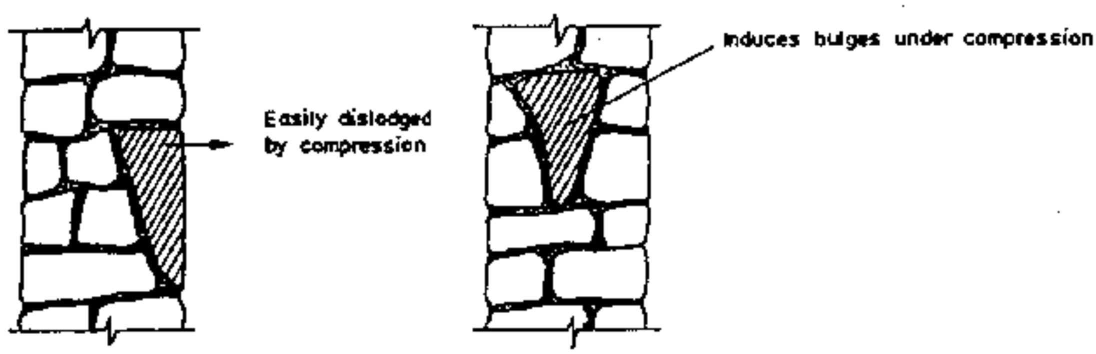


Figure 6.11 - Local Distresses due to Wrong Arrangement of Random Rubble Blocks

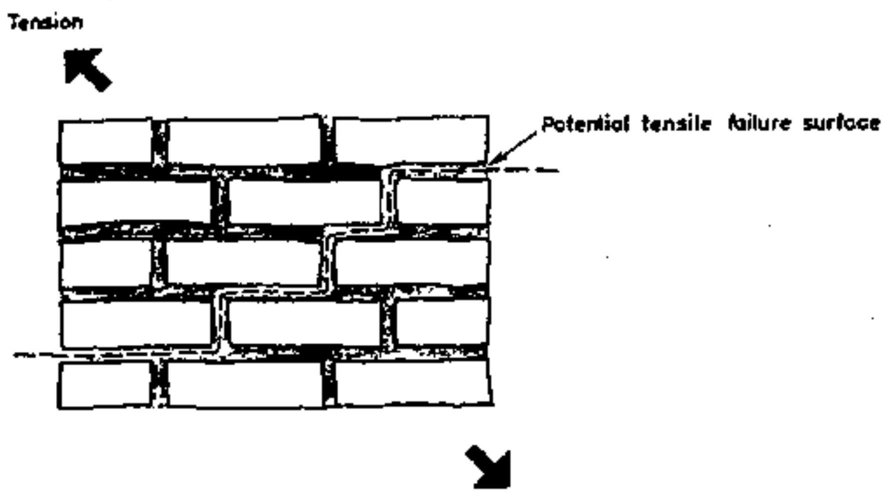


Figure 6.12 - Effect of Inclined Negative Minor Principal Stresses on Masonry

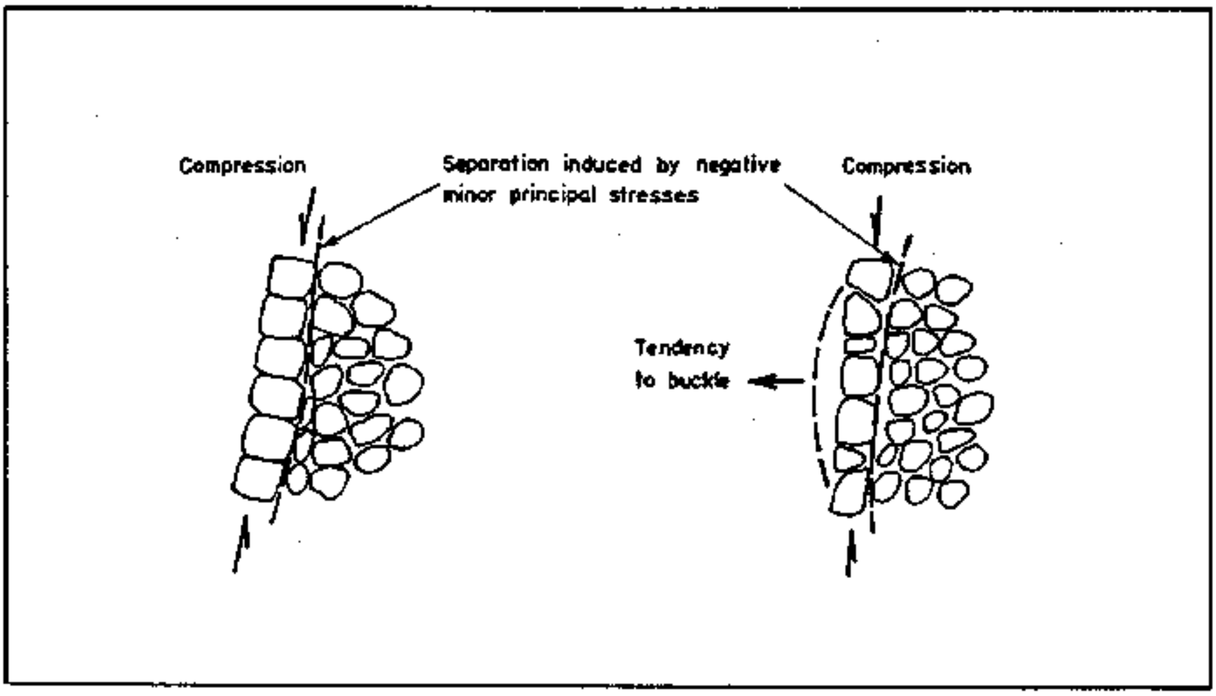


Figure 6.13 - Effect of Block Shapes on Buckling of the Face Layer of Stone Rubble Blocks in Masonry Retaining Wall

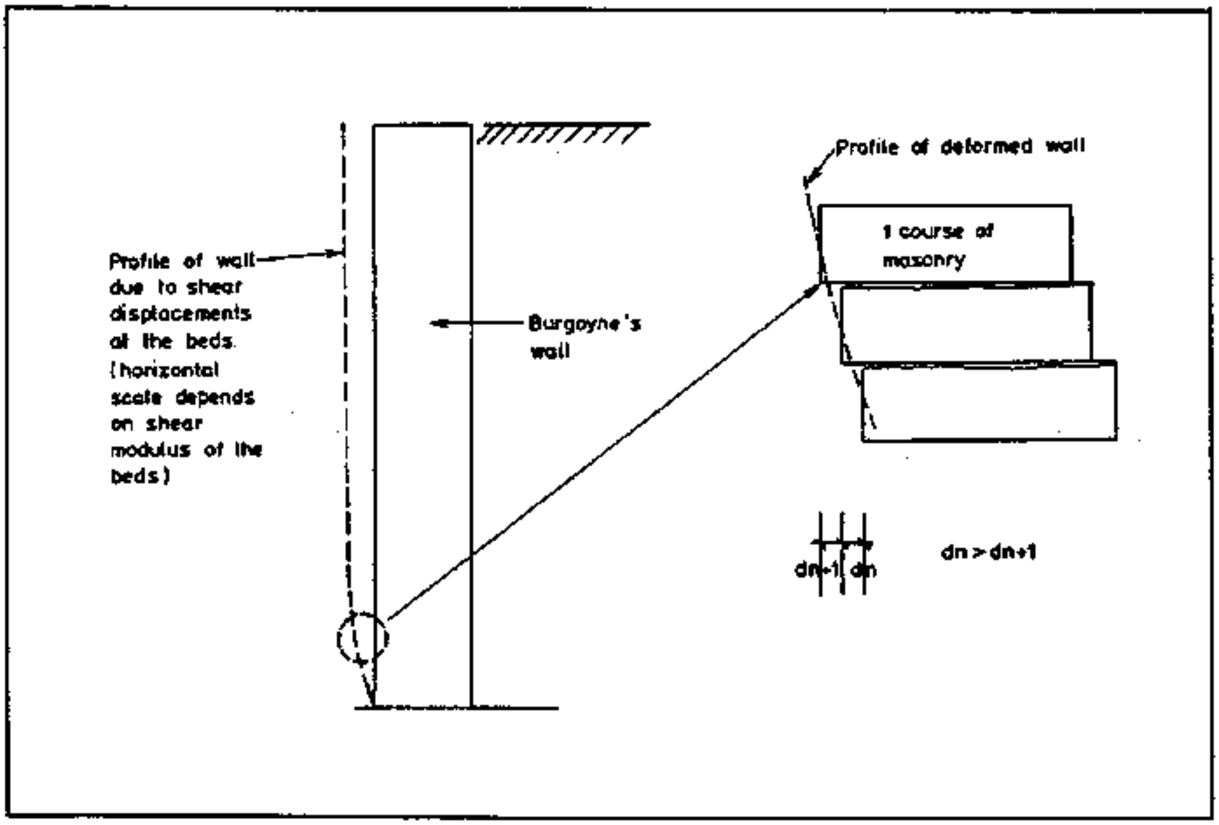


Figure 6.14 - Deformation of Masonry Retaining Wall due to Shear Displacement at the Beds

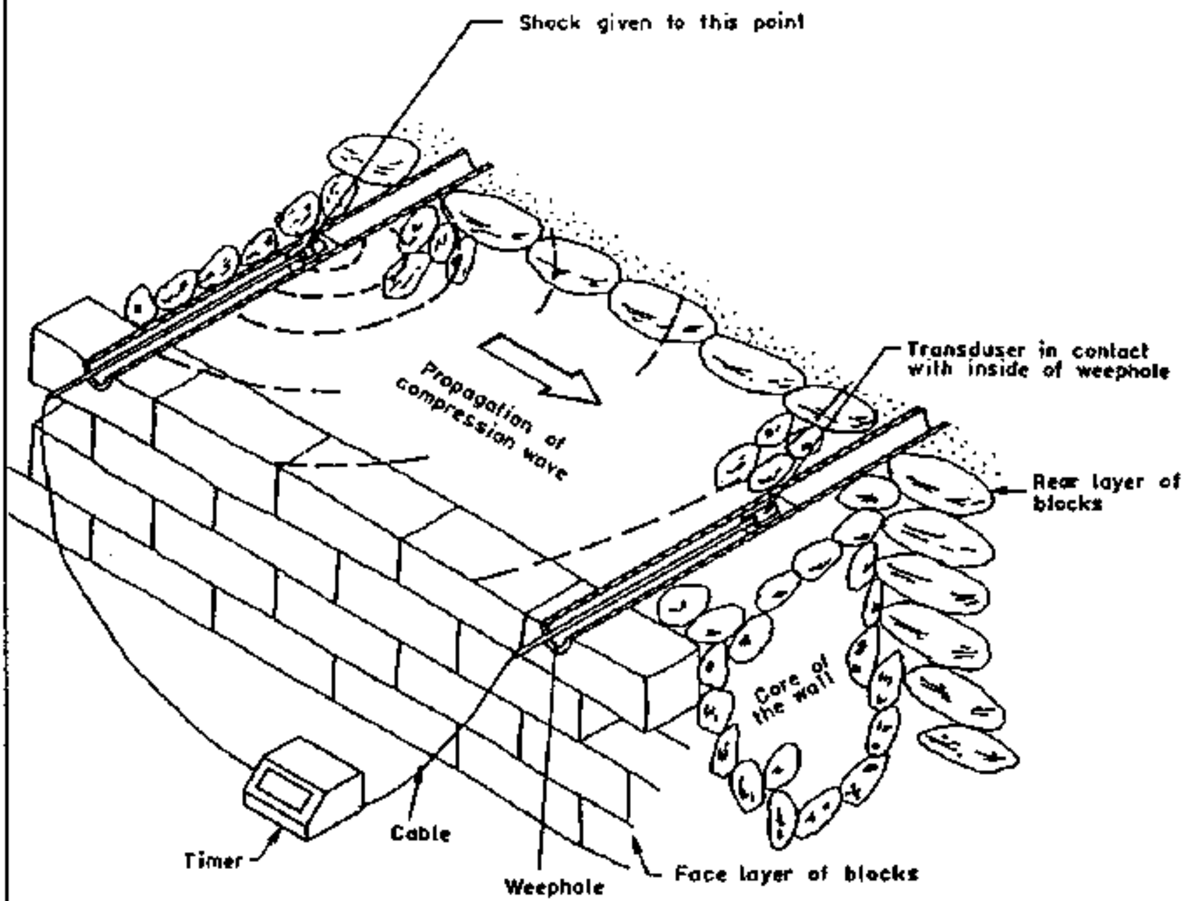
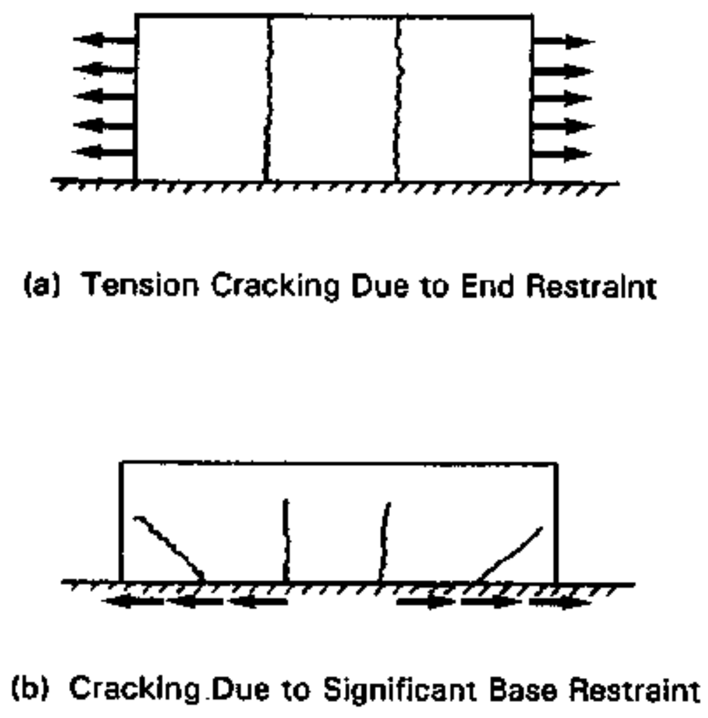


Figure 7.1 - Sectional View Illustrating the Method of Seismic Probing of Masonry Walls



Note : Figure after Huges (1973).

Figure 7.2 - Possible Crack Pattern on Walls due to Restraints Against Contraction

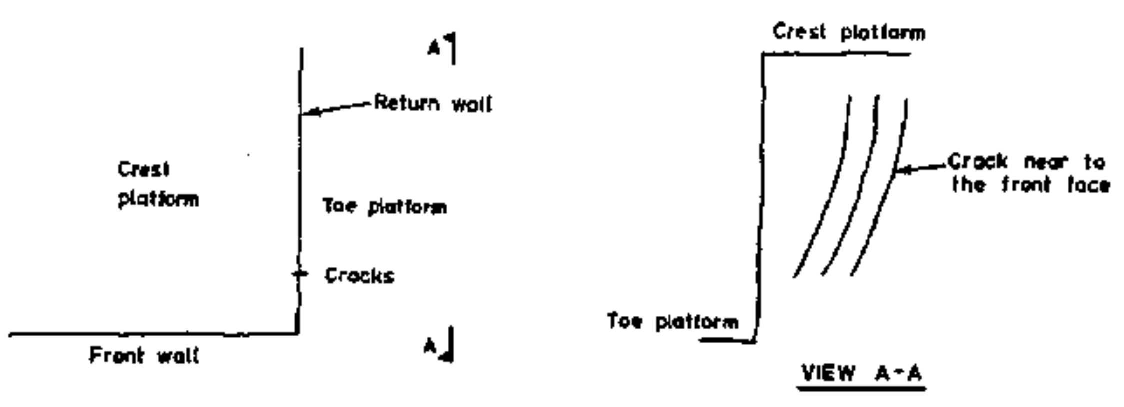


Figure 7.3 - Corner Cracks on Masonry Retaining Walls

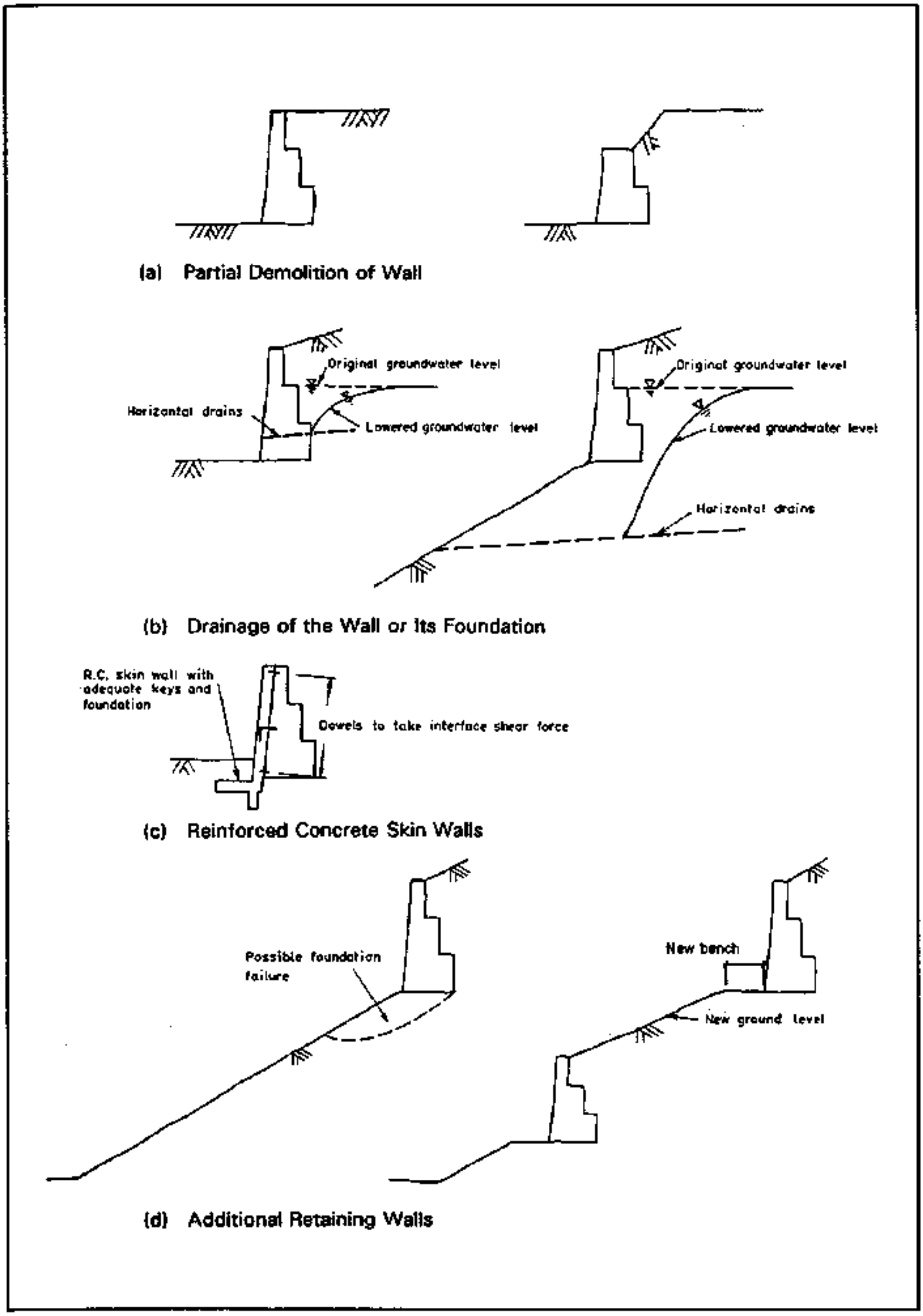


Figure 9.1 - Methods of Stabilising Old Masonry Retaining Walls

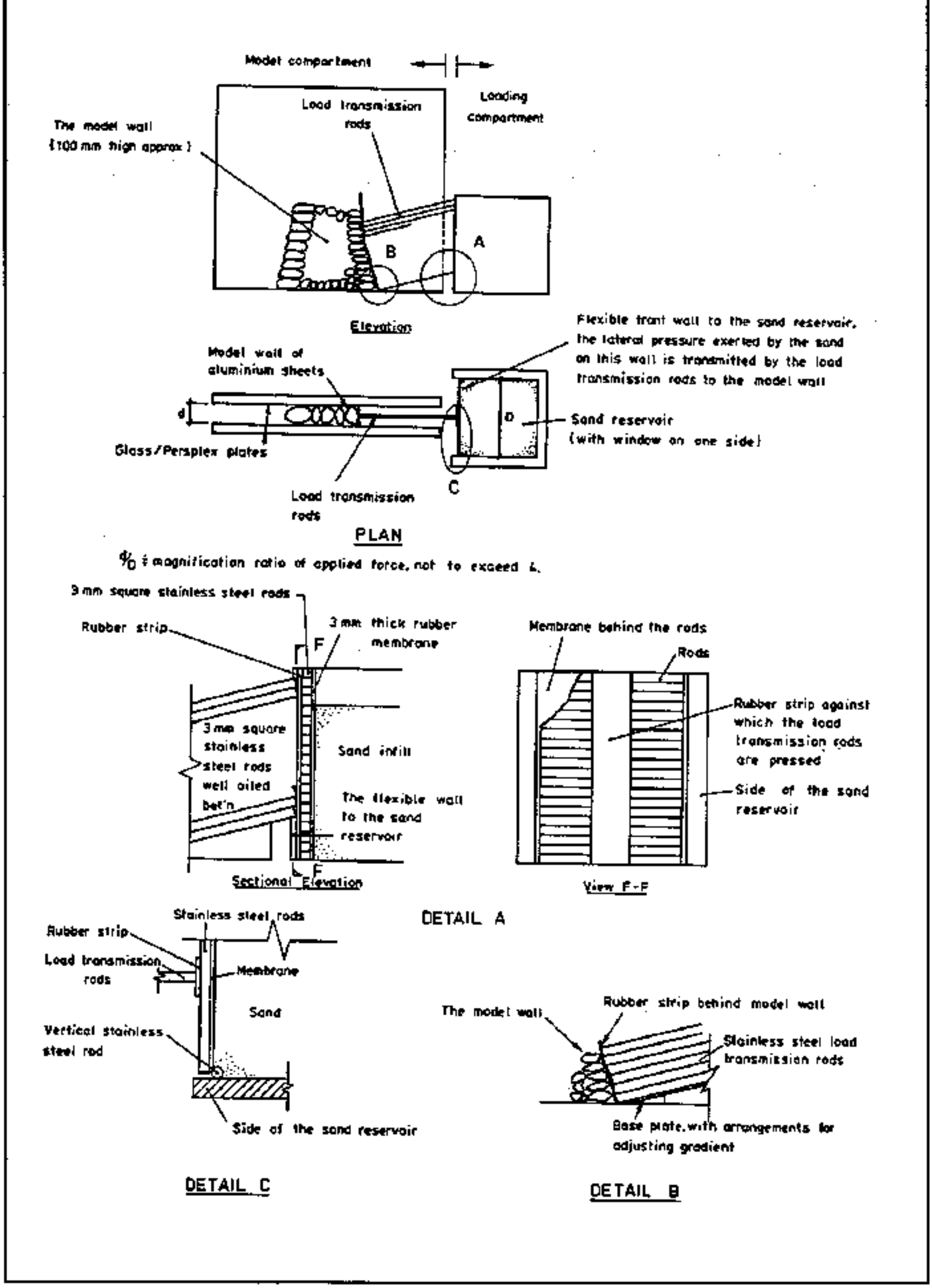


Figure 10.1 - Proposed Arrangement for Model Tests on the Failure Mechanism of Masonry Retaining Walls

APPENDIX A

REPORT ON THE STUDY OF OLD MASONRY RETAINING WALLS BY GCO (1980)

A.1 Field Inspection

The following features should be noted during field inspection :

(a) Signs of distress - See whether there is any bulging and relative movement of the wall. If tell-tales have been installed, any movement can be detected. For masonry walls, individual blocks may be displaced or the mortared joints crack. If individual block cracks, this may be due to movement of the wall or any fault during construction. In some cases, the tie beams or the walls of the structure on top of the wall may also crack.

(b) Settlement of the wall - The ground at the toe and above the wall should be inspected to see whether there is any cracking of the pavement or upheaval of the ground. Upheaval of the ground in front of the wall may indicate that the wall had rotated. Cracking of the ground generally suggests settlement of the ground. This may be confirmed by the relative vertical movement of the ground at the sides of the cracks or the copying of the wall.

(c) Sign of seepage - The locations at which water seeps out should be recorded. This may suggest where the ground water level is or whether there are drains leaking at that location. The amount of water flowing out should also be noted. Cracking of channels or pavement at the toe of the wall may allow water to infiltrate into the ground weakening the foundation of the wall.

(d) See whether there is vegetation covering the wall since it would cause serious cracking of the wall.

(e) Try to find out if there is any special structure adjacent to the wall. A highway adjacent to the wall may impose heavy loading on the wall. Vibration of the machines in a factory adjacent to the wall also imposes lateral loading on the wall.

(f) Look for consequence of failure - If a high wall is supporting a highway, which carries heavy traffic, with a lot of houses at the base of the wall, the consequence of failure of the wall is obvious very serious. On the contrary, if it is a small wall in open space supporting no important structure, the consequence of failure is low.

From the discussion with GCB, the following points are worth noting :

(a) They have done an analysis by using hypothetical wall dimensions and plotting wall height against base width for the limiting situations for sliding, overturning and shear through the wall and no tension at the base. They found that the case for no tension at the base is most critical. They also superimposed on the graph the dimensions shown on old drawings and the actual dimensions of those failed walls. They found that the constructed walls were different from those shown in the drawings and were on the unsafe side.

(b) GCB uses probing of weepholes to find the thickness of the wall. They claimed that they got good correlation with those obtained from drill-holes. Since the probe was pushed by hand, I am not in favour of this method. Binnie used pneumatic drill. In detail investigation, horizontal, vertical or incline boreholes can be used to determine wall dimensions.

(c) The most common sign of distress of these walls is bulging. For this I agree with the saying that the wall may be designed using K_a value for calculating the earth pressure. The active pressure need considerable movement in order to mobilize its full value. The wall may be constrained by the pavement at the toe preventing the wall to slide and therefore the wall bulges.

(d) We agree that traffic vibration can cause utilities breakage and affect the wall indirectly. I think the increase in surcharge load due to traffic vibration may have been accounted for in using HA and HB loadings (HA - 10 kN/m^2 and HB - 20 kN/m^2 which are already quite large).

(e) The loadings from adjacent structure may affect the retaining wall. Caissons and pile caps can carry lateral forces (mainly wind load) which in turn are transmitted to the retaining wall if they are close to the wall. I think this should be taken into account especially when piling is done adjacent to the wall. If the wall is above a 45° line drawn from the bottom of the foundation of the building, there will be no increase in lateral pressure on the wall.

(f) Leakage from water carrying services can decrease the strength of the soil. Special attention should be paid to water mains since water is under high pressure and imposes lateral pressure if the main bursts. By testing the water seeping out, the type of drain that leaks can be determined. This method is currently under study. A manometer can be inserted in the weepholes to measure the water pressure.

(g) Trees on the wall may have an anchoring effect. The increase in weight of the trees and the swelling of the trunk and roots due to the growth over the years would exert some additional surcharge loading.

A.2 Conclusion

During field inspection, any sign of movement, bulging, displacement of blocks, cracking of beams, cracking and upheaval of the ground, sign of seepage, vegetation covering, structure adjacent to the wall and the consequence of failure should be noted. Sophisticated method of finding the wall thickness of existing walls should be sorted out. The method of checking the structural serviceability of the wall should follow those set down by BOO. Consideration should also be given to the loading transmitted from adjacent structure, and the leakage of water carrying services.

APPENDIX B

EXAMPLE OF DIFFERENT TYPES OF MASONRY RETAINING WALLS

(MAINLY BASED ON BINNIE AND PARTNERS' REPORT ON PHASE 1A STUDY ON CUT SLOPES AND RETAINING WALLS, VOLUME 1, PART 1)

LIST OF PLATES

Plate B1 - Dry Packed Random Rubble Wall (11SW-A/R389)

Plate B2 - Pointed Random Rubble Wall (11SW-A/R116)

Plate B3 - Dry Packed Squared Rubble Wall (11SW-A/R109)

Plate B4 - Dry Packed Squared Rubble Wall with Horizontal Beams (11SW-A/R163)

Plate B5 - Pointed Squared Rubble Wall (11SW-A/R295)

Plate B6 - Pointed Squared Rubble Wall with Horizontal Beams (11SW-A/R194)

Plate B7 - Dressed Block Wall
(11SW-A/R46)

Plate B8 - Dressed Block Wall with
Horizontal Beams
(11SW-A/R423)

Plate B9 - Tied Face Wall
(11SW-A/R74)

Plate B10 - Tied Face Wall with
Horizontal Beams
(11SW-A/R45)

Plate B11 -
Random Rubble
Wall with
Stone Ties

Plate B12 -
Recent
Masonry
Walls

APPENDIX C

GLOSSARY OF TERMS

GLOSSARY OF TERMS

ASHLAR - See MASONRY.

ASHLAR WALL - Masonry wall which has on at least one face of the wall ashlar blocks laid with joints not wider than 12 mm.

BACKING - The use at the rear face of a wall blocks of material and/or quality different from (usually less superior than) those at the front.

BED - See JOINT.

BOND - (a) An interlocking arrangement of blocks within a wall to ensure stability. When standard format bricks are used, there are a number of standard bond patterns e.g. English cross bond, Dutch bond (Figure C1).

(b) Adhesion between mortar and stone composing a wall.

BONDER - Stone strips that penetrate two-third thickness of wall. See also HEADER.

COURSE - A continuous layer of blocks of uniform height (200 mm to 300 mm) in a wall, including the bed mortar.

Depending on whether the stone blocks in a wall are laid in such courses or not, the wall can be described as coursed, uncoursed or brought to course (Figure C2).

DRESSING - The process of fine picking and hammering the stone block faces to produce a uniform texture.

DRY STONE WALLING - A form of random rubble walling without mortar (in U.K. mostly found in the moorland areas). It is constructed of roughly dressed stones laid with a core of pise or small stones. See also MASONRY.

HEADER - Elongated stone strips laid with the longitudinal axis perpendicular to the face of the wall, to improve bonding of the wall. The American Railway Engineering Association (AREA) requires that their lengths and widths to be not less than 2½ times and 1¼ times of their thickness respectively. In Hong Kong, it is locally called TIE. The AREA does not specify that they should penetrate the entire wall unless the wall is thinner than 1 m. In BS 5390 : 1976, headers penetrating the whole wall are called through-stones. Otherwise, they are called BONDERS.

JOINTS - Thin spaces perpendicular to the wall surface between stone blocks composing the wall. In particular, a horizontal joint is also called a BED.

MASONRY - An assemblage of structural blocks so put together as to produce a well bonded solid structural element. The structural blocks may either be artificial blocks of brick, precast concrete, or natural stones. Natural stone blocks can further be classified as follows according to the different degree of efforts on squaring and dressing them.

(a) Ashlar - carefully cut and dressed blocks that can be laid with joints not more than 12 mm wide. The Chinese specification on masonry and block works requires them to have heights and widths not less than 200 mm or 1/3 of the length, whichever is the greater.

(b) Random rubble - either rough stones as they come from the quarry, usually not squared, or field stones. It is not intended to have additional dressing except as is necessary to place the stone in the structure and to knock off any edges or projections which might be detrimental to the construction.

(c) Squared rubble - stone blocks that have been worked to produce approximately planar and straight faces for bedding and jointing.

MORTAR - Mixture of sand, lime and/or cement as infill at joints and beds to ensure even contact between blocks and to provide some degree of cohesion.

BS 5628 : Part 1 : 1978 specifies four categories of mortar of different mix proportion and with 28-day compressive strength between 11.0 N/mm^2 and 1.0 N/mm^2.

POINTING - The external finish to beds and joints. It can either be put in as part of the mortar or else the mortar may be raked out for approximately 40 mm deep before the final set and be replaced by better quality cement/sand mixes. For dry-packed masonry walls, pointing may also be applied to the outside portion of the beds and joints to give a smooth surface as well as to discourage the establishment of vegetation.

POLYGONAL RUBBLE WALLING - The type of masonry wall constructed of stone hammer-pitched into irregular polygonal shapes. It may either be rough-picked or close-picked. For the former, the stones are only roughly shaped while for the latter, the face edge of the stones are more carefully formed to fit each other (Figure C3).

RIBBON POINTING - Pointing which projects proud of the face of the wall and is finished with a trowel. See also POINTING.

RUBBLE - See MASONRY.

RUBBLE WALL - Masonry walls with rubble as the main construction material. See also MASONRY.

STABILISED SOIL - Soil strengthened by the addition of lime and compaction.

SQUARING - The process of cutting or picking the sides of stone blocks to approximately flat parallel planes.

STRETCHER - Elongated stone strips laid with the longitudinal axis parallel to the strike of the wall. The AREA requirement of their dimension proportions is similar to that for header.

TIE - See HEADER.

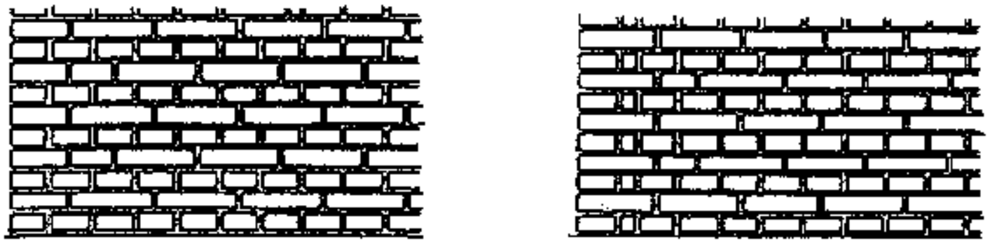
TIE COURSE - A continuous course of material penetrating the depth of the wall. It may either be a layer of concrete/stabilised soil or long stone strips laid side by side.

THROUGH-STONES - See HEADER.

UNIT - Structural blocks for building up masonry, see also MASONRY.

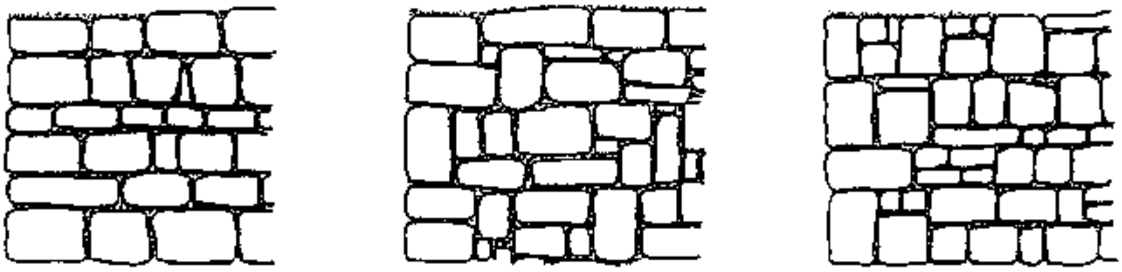
(Note : See Figures C1, C2 & C3)

LIST OF FIGURES



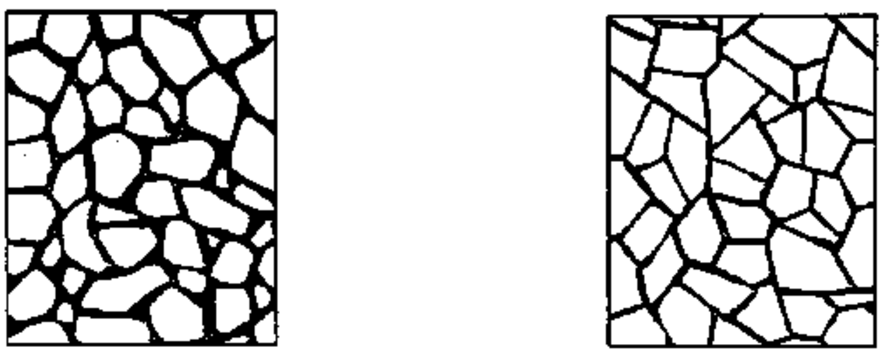
(a) Dutch Bond (b) English Bond

Figure C1 - Bond Patterns for Walls of Standard Format Bricks



(a) Coursed Stone Rubble Wall (b) Uncoursed Stone Rubble Wall (c) Stone Rubble Wall Brought to Course

Figure C2 - Stone Wall Face Patterns



(a) Rough-picked Polyonal Walling (b) Close-picked Polygonal Walling

Figure C3 - Polygonal Rubble Walls

APPENDIX D

CASE HISTORIES OF INSTABILITY OF MASONRY RETAINING WALLS IN HONG KONG

Note

The case histories are hereafter presented as an abstract of observations and comments from various parties on the instances of instability of retaining walls. Personal comments from the writer are restricted to brief interpretation on the causes of the incidents. Most of the contents of each case are taken from a limited number of sources listed out at the start of each case record. No further attempts are made to state the exact source or letter/memo reference at the end of each paragraph. The main purpose of these case history records is to familiarise the readers with common features associated with instability of masonry retaining walls, rather than for a judgement of responsibilities or the correctness of past decisions. Therefore, an effort is made not to mention the names of the involved parties as far as possible.

Case No. : 1

Location : Failure of Retaining Wall at St. Joseph Terrace

Date : 16.7.1917

Source of Information :

The Morning Post, 17.7.1917, 18.7.1925
The Hong Kong Daily Press, 17.7.1917
China Mail, 17.7.1917

Case No. 1

1. The location of the retaining wall and the layout of the site are shown in Figure D1.1

2. At the time of the failure, St. Joseph Terrace at the crest of the subject wall was utilised as the playground of St. Joseph College. The College building was on a platform higher than and immediately south of the playground.

3. At the corner of the wall stands the Mission House of the Roman Catholic Cathedral. Adjoining it and immediately in front of the retaining wall were No. 10, and 12 of Caine Road. Both these two houses were 3-storey (brick) buildings with semi-detached servant quarters at the rear.

4. Originally, there was a low retaining wall of dry packed stone rubble. The subject wall was erected on top of the older wall in about 1911. The new extension consisted of "Customary stones and clay with cement filling the interstices and binding the clays."

5. The subject wall was 4 feet thick at the bottom and 2 feet at the top. It retained approximately 50 ft of earth from the level of Caine Road.

6. Some two years before the failure, a small crack appeared at the corner within a few feet of the Mission House. It was infilled with cement. The crack reactivated sometime before the failure took place. The observations made on it were summarised in Table D1.

7. The Mission House has a narrow escape of the failure debris. The servant's quarters of no, 10 and 12 Caine Road were reduced to a heap of rubble whereas the front structure of these two buildings remained undamaged.

8. At the time of the failure, the crest platform (playground of St. Joseph's College) was being repaved. Half of the playground was covered while the remaining uncovered half was saturated to sodden mud.

9. The failure was about 75 ft wide and situated in the unpaved region.

10. About fifteen Chinese were buried. Nine were recovered alive. Of those rescued, 7 were protected from serious injuries by a broken beam which support the weight of the debris. Of those who were killed in the accident, at least three were children suffocated to death.

11. From the various evidence, it appears that the inadequate thickness of the wall was the basic cause of the failure although the saturation of the retained soil through infiltration from the unpaved surface also contributed to the final collapse.

Case No. : 2

Location : Failure of Retaining Walls at Po Hing Fong

Date : 17.7.1925

Source of Information :

The Morning Post, 18.7.1925, 20.7.25, 22.7.25, 25.7.25, 28.7.25, 29.7.25, 30.7.25, 8.8.25, 3.9.25, 5.9.25

Case No. 2

1. The failure involved three retaining walls forming the northern support of the site of the old Number 8 Police Station.

2. Figure D2.1 shows the location of the walls and the layout of the adjacent ground and a typical section of the ground.

3. There was a ledge between the upper wall and the middle wall. At the foot of the middle wall was another ledge on which ran a footpath with an iron railing on the lower side. Below this railing there was a grassy slope as far as the top of the lower wall.

4. The upper and middle walls were constructed in the year 1860 while the lower wall was constructed in 1896 to retain a cutting.

5. In front of the lower wall was No. 11 to 29, Po Hing Fong. However, the lower wall was longer than the other two walls and only No. 11 to 17 of Po Hing Fong were faced with the full height of all the three walls.

6. In 1923, redevelopment of the No. 8 Police Station was started together with widening of the Hospital Road south of the site. At the time of the incident, the trench excavation for the foundation of the new building was completed. A substantial part of it was covered with concrete for the substructure. The whole site was partly covered by the ground floor paving of the original police station.

7. The year 1925 was exceedingly wet (refer Table 4.3, Figure 4.2). The corona of a death enquiry noted that there were "5 months of heavy rainfalls before the failure". On the morning of the incident, the rainfall was particularly heavy and Caine Road, as well as Po Hing Fong, were flooded to a few inches.

8. From the description of the eyewitnesses in the death enquiry court, the failure seemed to have started with the sinking of the western end of the site. The movement gradually propagated towards the east together with outward tilting of wall. This caused the toppling of two matsheds at the extremely east edge of the crest platform (the site for the No. 8 Police Station). This series of movements was apparently caused by the yielding of the middle wall, as was according to the description of a tenant in one of the collapsed houses who happened to have witnessed the failure.

Case No. : 3

Location : Failure of Wall at 10, Castle Road, I.L. 7976

Dated : 19.6.1970

Source of Information : D204/70/H.K., 13/2943/63

Case No. 3

1. Refer Figure D3.1 for the location of the wall and the layout of the adjacent grounds.

2. The wall failed on 19.6.1970, after days of heavy rainfall (see Table 4.3). The failure, as reported by the Hong Kong Standards, was

> "After heavy rainfall (yesterday), a car and a compressor plunged from the parking space. Two water mains burst - one drinking water and the other salt water."

3. After inspections by staff of BOO, the details of the failure were given as

   (a) The retaining wall is of poor quality mass concrete.

   (b) Adjacent to the wall, on the side of the lot, sheet pile was used to support an excavation for the foundation of the new building.

   (c) Suggested main factors of failure i) poor quality of material of wall ii) recent excavation (by the Gas Co.) in Castle Road have no doubt provided easy routes for subsoil water.

4. It was further noted by BOO staff in later inspections that the extent of the collapse coincided with the extent of the sheetpiles; where the piling was in two rows, the wall, although insecure, had not fallen.

5. The unfailed sections of the wall was again brought to attention in the September 1973. The wall was found to be composed of poor quality lime stabilised soil. It crept under pressure and high groundwater regime. Consequently, it pressed against the beams and columns of the building and induced shear cracks on them. Where the wall is not supported by the structural members of the building, it bulged out (Plates D3.1 to D3.4).

6. It was finally stabilised by concrete facings with ground anchors.

Case No. : 4

Location : Failure of Retaining Wall at Thorpe Manor, 1, May Road, I.L. 2139

Dated : 2.9.1973

Source of Information : D186/78/H.K., 1,2,3/2180/72

Case No. 4

1. Figure D4.1 shows the location of the subject wall and the layout of the site.

2. The subject wall is 6.5 m high. It supported the platform on which Thorpe Manor stood. Below the wall is a 12 m high natural slope with an average gradient of 35°. North of the slope was May Road and the Grenville House between stood a steep cut slope. This cut slope was probably formed in association with the construction of the Grenville House.

3. In that area, the ground is covered by an appreciable thickness of slope wash and colluvium derived from volcanic rocks.

4. At the time of the incident, Thorpe Manor was being demolished.

5. On 2.9.73, there was heavy rainfall in Hong Kong under the influence of typhoon Ellen. In the afternoon, BOO received a report of a landslip at 1, May Road. Engineers were sent to inspect the site.

6. As the party of engineers approached the site, the second slip occurred. This was the major slip. It was described by the inspection engineers as "the sliding and overturning of a major portion of a retaining wall". The failed wall was the subject wall.

7. Plates D4.1 to D4.3 shows the failure at the day of the incident.

8. The fallen wall was described as "to have remained intact with sections weighing approximately 200 tons". These large sections nearly fell over the edge of the cut slope at the rear of Grenville House but was stopped in time by a low bund at the crest of the slope.

9. From the photographs, the wall appears to consist of stabilised soil with squared rubble facing.

10. The foundation wall of the Manor formed the rear of the failure scar.

11. The whole length of the wall fell with the exception of the east and west ends. At the east end, 3 buttresses had been constructed previously to strengthen the wall. One of them had failed with the central section of the wall while the other two were out of plumb.

12. At the crest of the east corner of the remaining section of the retaining wall, a large crack of several inches wide was observed between the face of the building and the earth. Other cracks were "also seen in many places".

13. The slip surface was found to be very wet and continued to crumble.

14. Vegetation and seepage marks were observed on the remains of the wall.

Case No. : 5

Location : Failure of Retaining Wall at Caine Lane behind U-Lam Terrace

Dated : 25.8.1976

Source of Information : H.H. C2, Aerial Photographs of the Failure

Case No. 5

1. Figure D5.1 shows the location of the wall and the layout of the site.

2. Very little is known of the wall before failure. The adjacent wall is of squared rubble facing to a stabilised soil core.

3. Groundwater level in the area was high. It caused a lot of problems in the execution of remedial works. Horizontal drains were finally installed to lower the ground water table.

4. Two sets of aerial photographs were taken of the site immediately after failure. The surface profile of the failure debris was surveyed two weeks after failure. This information is at present being interpreted by the Aerial Photograph Interpretation Unit and the Survey Section of GCO for the distribution of the debris and the deformation of ground adjacent to the failure.

Case No. : 6

Location : Unstable Retaining Wall at 3-7, Circular Pathway

Date : August 1977

Source of Information : D 167/77/H.K., 1,2,3/2558/58

Case No. 6

1. Refer Figure D6.1 for location of the wall and the layout of the adjacent ground.

2. As part of the Urban Renewal Pilot Scheme, buildings at 3-7 Circular Pathway were to be demolished in the Autumn of 1977.

3. These were pre-war brick buildings. A lane slightly wider than 1 metre was left between the retaining wall and the rear wall of the building. Brick arches were constructed between the two walls, apparently at the location of the partition walls of the buildings.

4. The retaining wall was of tied face type, with a height over 8 metres. From the geology and history of formation of such sites, it was likely that the retaining wall was constructed to support in-situ decomposed granite.

5. Demolition of the buildings commenced on 1.7.1977. Before that, a pre-demolition inspection was made by an engineer of BOO on the wall and the Pathway (3/77). It was noted then that 1, 8, 9 of the Circular Pathway had already been demolished leaving the wall in an "apparently" sound and dry condition.

6. Incidentally, no. 10 and 11 of Circular Pathway were redeveloped in the early 60's. A large diameter pumping well was installed in the courtyard of this building. This should have caused a local drawdown of groundwater.

7. Plate D6.1 shows the wall near 10, 11 Circular Pathway towards the end of the demolition work.

8. On 8.8.77, when the demolition works were substantially completed, a post-demolition inspection was made (by the same engineer of the pre-demolition inspection) on the area. A continuous crack was found on Circular Pathway adjacent to the granite blocks of the retaining wall.

9. The wall was inspected again on 9.8.77, the crack was found to have "noticeably" widened (to 6 mm wide).

10. On 10.8.77, the wall was classified as "showing signs of movement and instability in condition of prolonged rainfall".

11. It was also noted that the wall wetted up to half its height (at certain locations).

12. Arrangements for dead shoring the wall was started.

13. Inspection was again made on 22.8.77. It was found that considerable movement and change to Circular Pathway and the adjacent area had occurred since it was last inspected on 12.8.77.

14. Plates D6.2 to D6.20 show the wall and its crest on 22.8.77.

15. Based on the photographs, the pattern of the cracks on the Circular Pathway on 22.8.77 is sketched on Figure D6.1.

16. In a statement on 23.8.77, BOO described that "water from an unknown source exerted pressure on the wall which is bulging. Subsidence and crack occurred on Circular Pathway and is noticeably widening and extending".

17. Because of the critical state of the wall, the shoring work was terminated, to be replaced by the construction of a free draining embankment (6 m high approx.) at the toe.

18. No. 24A-25A of the Circular Pathway (on the wall's crest platform) was also demolished to reduce loading on the wall.

19. These measures stopped the wall from further movement.

Case No. : 7

Location : Unstable Retaining Wall at 22, Old Peak Road

Date : 11.5.1978

Source Information : D191/76/H.K.

Case No. 7

1. The location of the wall and the layout of the adjacent ground are shown in Figure D7.1.

2. The wall was a dry packed random rubble wall. The joints were not pointed. The height of the wall was between 4 and 5 m.

3. The wall was inspected by a geotechnical engineer on 11.5.78. He discovered signs of instability, i.e. "bulging, voids between blocks, and compression cracking at the face" (Plate D7.1).

4. It was not known whether these signs were new or had been there for a period of time.

5. There were signs of subsidence and cracking on the road at the crest. From the photographs (Plate D7.3 to D7.4) it can be seen that there was a newly reinstated trench on the uphill side of the road. At approximately midway between the trench and the parapet was a long continuous crack parallel to the alignment of the road. There has been some subsidence on the area between the crack and the parapet. The darker colour of newly repatched road surface could be seen.

6. Writing on the incident, the house manager of the building at the toe platform said that "The affected portion of the dry stone wall is immediately beneath an area of Old Peak Road that had been the subject of trench work and backfilling by the telephone company. The backfilling had sunk drastically and emergency surfacing had been carried out by the Highways Office.

7. The wall was later investigated and stabilised by a concrete wall constructed in front of it. In the study, the engineering consultant felt that the stability of such wall cannot be dealt with by soil mechanics principles. In the design, the masonry was treated as a skin wall without much contribution to the stability of the cutting.

8. The incident occurred at a time when rainfall was not particularly heavy. The relationship between the road and trench work and the state of distress of the wall is uncertain. The trench work, together with the compaction of new surfacing, might have induced the bulges. Alternatively, the surface subsidence and cracking might have been caused by the loose backfill to the trench. In this latter case, the crack was not a sign of instability although it drew the attention of the inspection engineer to the distressed state of the wall.

Case No. : 8

Location : Failure of the Retaining Wall at 14-16, Fat Hing Street, adjacent to 48-56, Queen's Road West

Date : 29.7.1978

Source of Information : D 26/72/H.K., 1,2,3/2101/76

Case No. 8

1. The location of the wall and the layout of the adjacent ground are shown in Figure D8.1

2. The subject wall was a tied face wall forming the northeastern support to a platform locally known as the Possession Point Chinese Recreation Ground. East of and perpendicular to the subject wall was a similar retaining wall forming the northwestern support of the same platform.

3. The buildings in front of these two walls (6-16, Fat Hing Street, 48-56, Queen's Road West) were demolished earlier as part of the Urban Renewal Pilot Scheme. Brick party walls of these buildings were partly left as buttresses at 5 m centres.

4. The northwestern wall was 8.5 m high. The northeastern wall (the subject wall) was broken up by intermediate platform into two section of walls of 3.5 m and 5 m at the top and bottom respectively.

5. Plate D8.1 and D8.2 show the wall before construction work was started on 48-56, Queen's Road West.

6. Redevelopment of 48-56, Queen's Road West was started in 1977. In the design it was planned to replace the northwestern tied face wall by screen walls. Sheet piles were driven behind and clear of the northwestern wall. However, the screen wall could not be constructed before the structural frame of the new building was completed for 8 m or higher. Consequently, the original tied face wall had to be temporarily supported for the excavation and construction of the foundation.

7. Steel raking shores were erected for this purpose (Plate D8.3, D8.4). The pile cap was substantially completed at the time of the incident. The raking shores were in position and excavation was in progress adjacent to the toe of the wall.

8. Shortly before the failure of the wall a 0.8 m deep trench was excavated on the crest platform sub-parallel to the walls. The trench was for the laying of a water pipe in association with the Urban Renewal Scheme (Plate D8.5).

9. On the day of the failure there were heavy rainfall brought by the Typhoon Agnes.

10. The wall collapsed at 11 pm. on 29.7.78. "The collapsed section is the end nearest the construction site and comprises a 8 m section of the 30 m wall". The debris of the failure pressed against a weakly supported mild steel waling of the work site at 48-56, Queen's Road West and caused it to deflect laterally (Plate D8.6 and D8.7).

11. Figure D8.2 shows the location of the failure and the construction sit eat Queen's Road West. The Figure was composed from a pre-construction survey record of the site (in 1/2101/76), the sketch attached to the incident report (58 in 2/2101/76) and the building contractor's sketch and photographs of the failure (52 in 3/2101/76).

12. The debris of the failure was described as "an extensive amount of rubble across the toe playground but very little soil from behind the wall had slipped. The retained soil behind the wall appeared to be D.G. in good condition standing almost vertically". In other words, the failure of the wall was not caused by the weakness of the soil behind.

13. Writing on the cause of the failure, Water Supply Dept mentioned that the sheet pile of 48-56, Queen's Road West had been driven through the northeastern wall which later failed. It is not known whether it was the case or not. In the photographs it appears to be true. However, it would be very difficult to drive sheet pile through tied face wall. If the sheet piles were really driven through the wall then it should have weakened the wall.

14. The immediate cause of the failure according to the inspection engineer from GCB was a build-up of water pressure behind the wall.

15. It appears that the presence of the trench on the periphery of the crest platform no doubt contributed to this rise in water pressure.

Case No. : 9

Location : Failure of the Retaining Wall at 1-10 Wing Wa Terrace

Case No. 9

1. The location of the wall, the layout of the site and the activities on the site at the time of the incident are shown in Figure D9.1.

2. The subject wall was the north support to the platform known as Wing Wa Terrace. In front of the wall were 1-13 Rutter Street.

3. The wall was a 9 metre high dry packed random rubble wall. It has an average batter of 83° (Plate D9.1).

4. In the winter of 1974, crude monitoring systems were established on the wall when settlement at the crest platform and heavy seepage at the toe of the wall aroused concern over its stability.

5. No movement was detected in May, 1975.

6. Binnie and Partners inspected the wall in 1978 in association with the Caine Road Area Study. It was described as in a critical state of instability. The signs of distress as described in a letter report to the P.W.D. were :-

   (a) a bulge in the wall behind 7-8 Rutter Street;

   (b) steepening of the wall from 83° to near vertical behind 1-3 Rutter Street;

   (c) failure of a strut cast from 4-5 Rutter Street to the wall; this strut could have failed because of high compressive forces or by rusting of the reinforcement;

   (d) in several places evidence of relative movement between masonry blocks;

   (e) broken steps behind 3-4 Rutter Street; the damage may be caused by compression or by settlement induced because of erosion of the underlying material.

7. In reaction, BOO issued a notification to the owners of the houses in Wing Wa Terrace requesting them to carry out preventive works on the wall. The owners employed a geotechnical consultant to study the stability of the retaining wall.

8. The section of the wall was determined by two vertical drill holes, one horizontal drill hole and some inspection pits. The soil parameters were taken as $c' = 8.06$ kPa, $\phi' = 37.5°$. Factors of safety against sliding and overturning were calculated as 1.28 and 1.61 respectively.

9. The remedial works recommended included sheet piling at the toe to improve sliding resistance, 6 m long horizontal drains at 3 m centres at the bottom of the wall to lower ground water and concrete counterweight at the crest to improve stability against overturning, (Figure D9.2).

10. The stabilisation works started in Sept. 1978. At the time, the buildings at 1-12 Rutter Street were being demolished for redevelopment. The demolition was substantially

completed except at the previous 4-5 Rutter Street. At this location, the retaining wall was supported by some concrete struts thrusting against the old buildings. Therefore, the buildings have to be demolished in stages with allowances for shoring the wall.

11. The horizontal drains were first installed. A total of 12 drains were installed. Constant flows were observed from them.

12. The contractor then proceeded to install sheetpiles. Difficulties in pile-driving were reported. The vibration caused dropping off of pointings from the wall. The contractor inspected the crest platform (Wing Wa Terrace) after two piles were driven. A ¾" wide crack between the pavement and the side of the buildings extending for half the length of the wall was discovered.

13. Sheet pile driving was continued after provisions for shoring up the whole wall were made. After another two piles were driven, a bulge developed at 4 m below the crest near the east end of 2, Wing Wa Terrace. The driving operation was stopped.

14. On 10.11.78, an inspection engineer reported that the sheet piling had been stopped and other than those already driven, the sheet piles were removed off site.

15. The wall failed at 1 a.m. on 13.11.78, at a time when the weather had been dry for a long period of time.

16. The failure was described by the inspection engineer as,

> "There was an extensive amount of rubble and soil across the empty site at 1-2, Rutter Street, and also an amount of soil from behind the wall had slipped with the wall. The material behind the wall appeared to be fill and part of the foundation supporting the building was exposed. There were also water discharging from one broken pipe and the ground under the floor. There was a new vertical crack on the parapet of the wall supporting 3-4, Wing Wa Terrace and a glass tell-tale placed across an old crack on this portion of the wall was cracked."

17. There were some photos of the failure (Plate D9.2 to D9.5). A sketch of the failure was also available from the geotechnical consultant (Figure D9.3) of the lot owner.

18. After the failure it was recognised that the structure of the wall was unstable and needed strengthening. The stabilisation measures were modified to a thick skin-wall properly dowelled to the rubble surface.

19. There is no doubt that the driving of the sheet piles was the immediate cause of the failure. The vibration might have damaged the structure of the wall. Sometimes this type of walls was provided with a spread footing. Being driven too close to the toe of the wall, the sheet piles might have disturbed the footing and caused the failure.

20. It was also suggested that the vibration might have cracked a sewer behind the wall. The resulting leakage caused a local rise in groundwater level and reduced the stability of the wall.

21. Another worthnoting point of this failure is that even when left undisturbed, a newly formed bulge may develop into a total failure over a period of time (more than 3 days).

Case No. 10

1. Figure D10.1 shows the location of the wall and the layout of the site.

2. The wall supports a platform on the far side of which the Recreation Club stands. Immediately adjacent to the wall on the crest platform are the car parking spaces (unpaved) and a tennis court (paved) which is at the western side of the platform. A staircase on the central part of the wall connects the crest platform with the toe platform.

3. The wall is a 3.5 m high dry packed random rubble wall with a surface batter of 10°. There are a number of serious bulges at and near to the staircase.

4. The failure occurred on 3.8.79. There had been heavy rainfall under the influence of Typhoon Hope (see Table 4.3, Figure 4.2). A 5 m portion of the wall in the west end near the tennis court failed. This section of the wall did not bulge particularly seriously before its failure.

5. Very little is known about other aspects of the failure. A photograph of the failure is available in GCB in the retaining wall inspection cards (wall no. W19).

LIST OF TABLES

LIST OF FIGURES

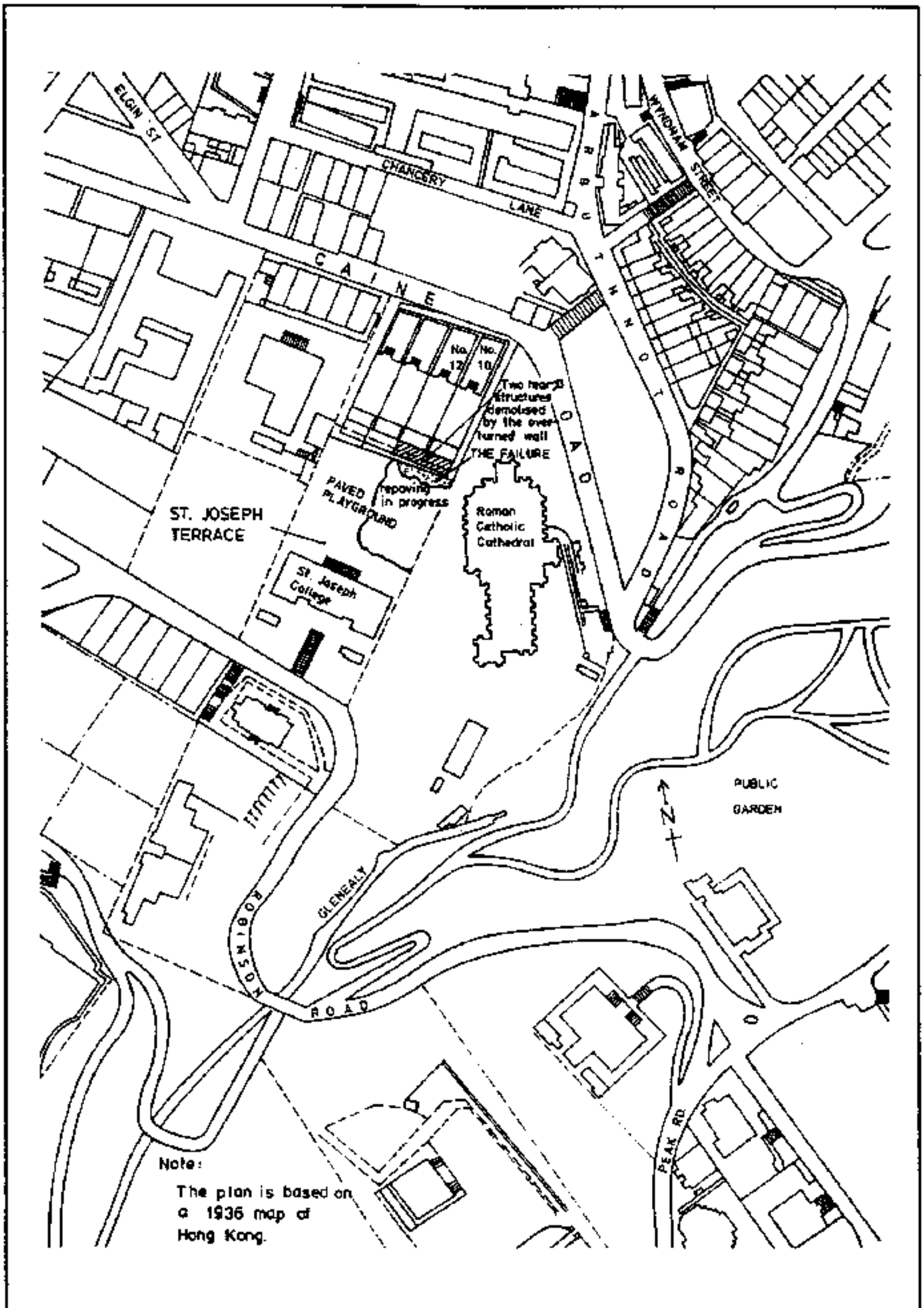


Figure D1.1 - Retaining Wall at St. Joseph Terrace - Site Plan

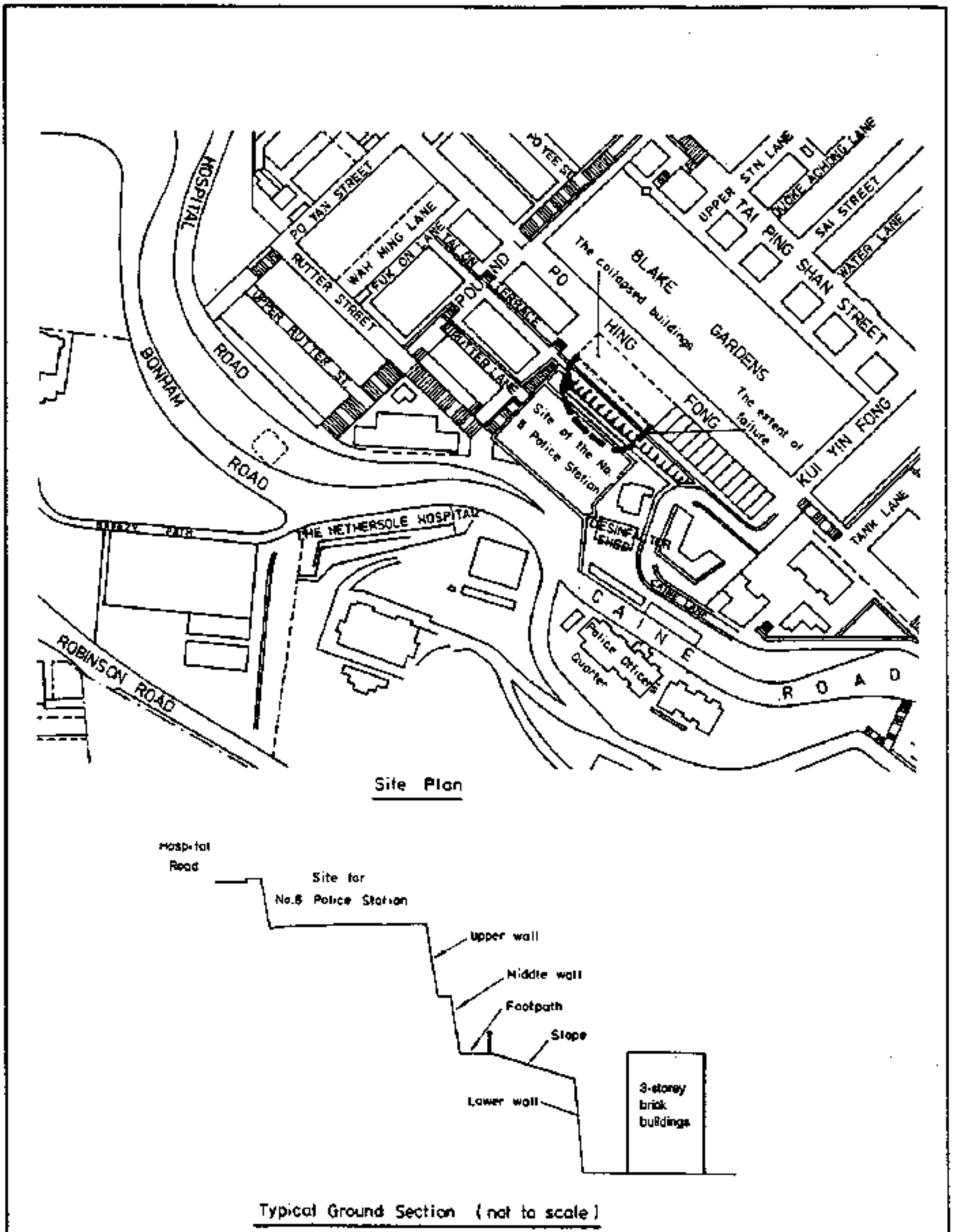


Figure D2.1 - Retaining Wall at Po Hing Terrace - Site Plan and Typical Sections

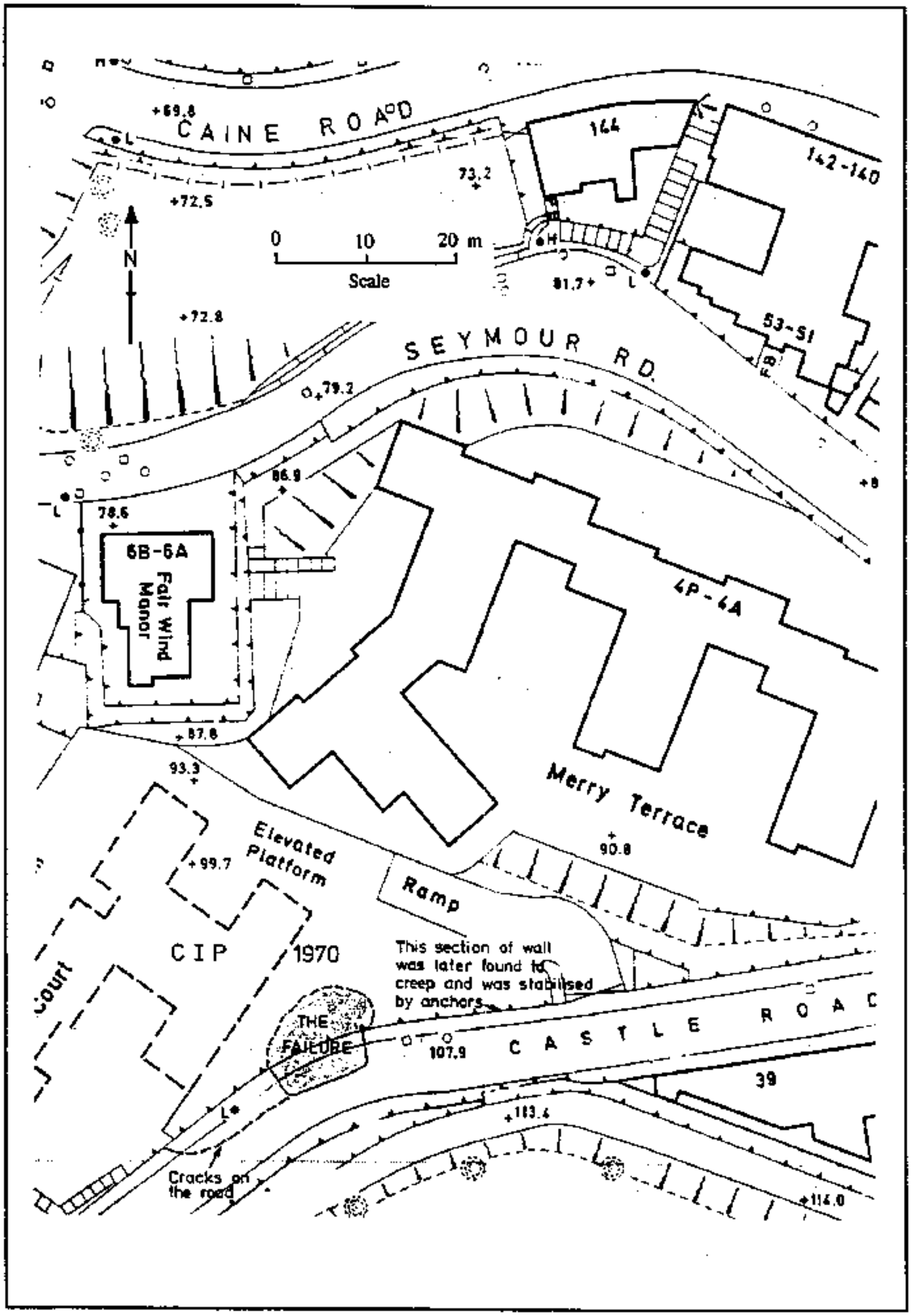


Figure D3.1 - Retaining Wall at 10, Castle Road - Site Plan

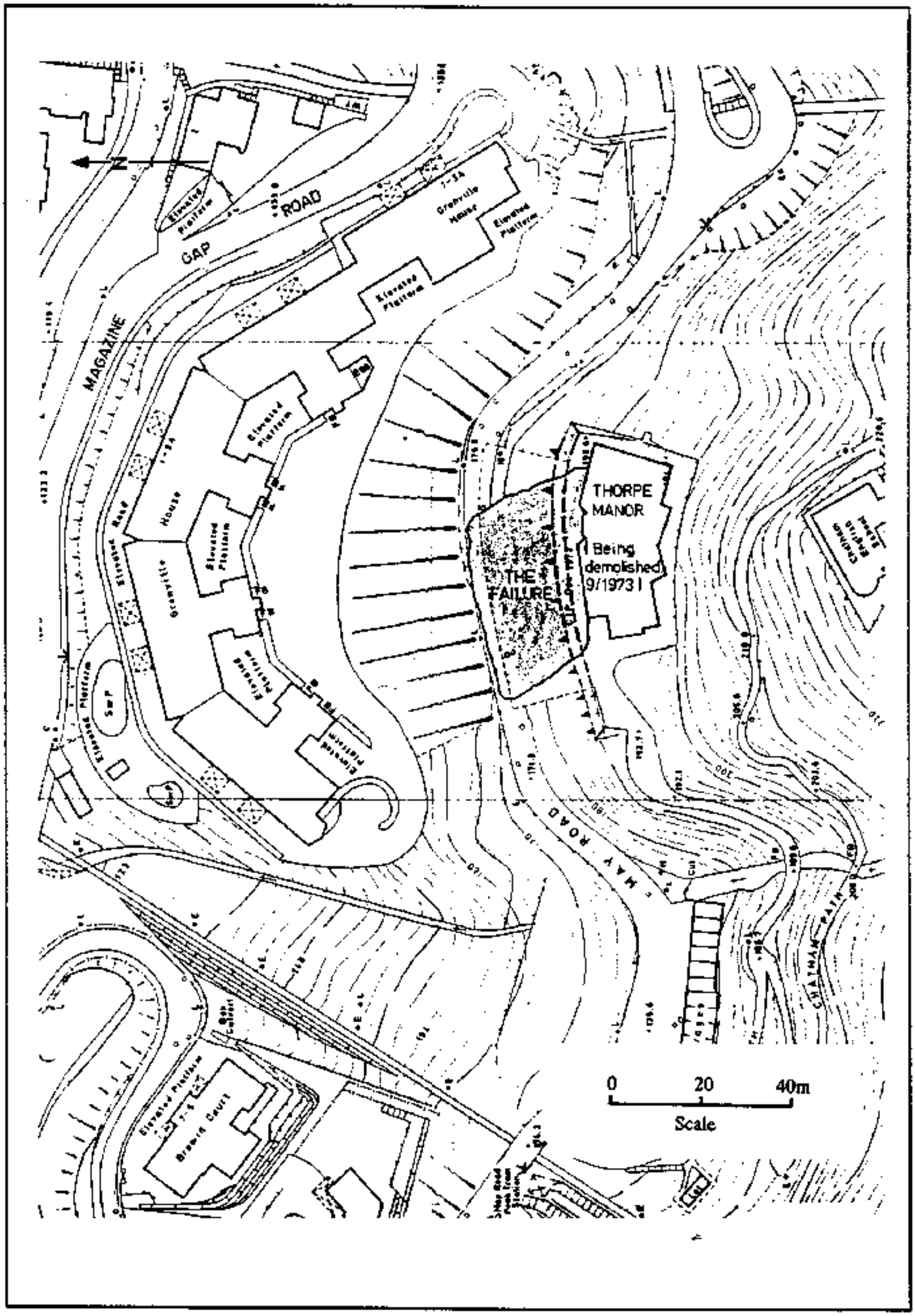


Figure D4.1 - Retaining Wall at 1, May Road - Site Plan

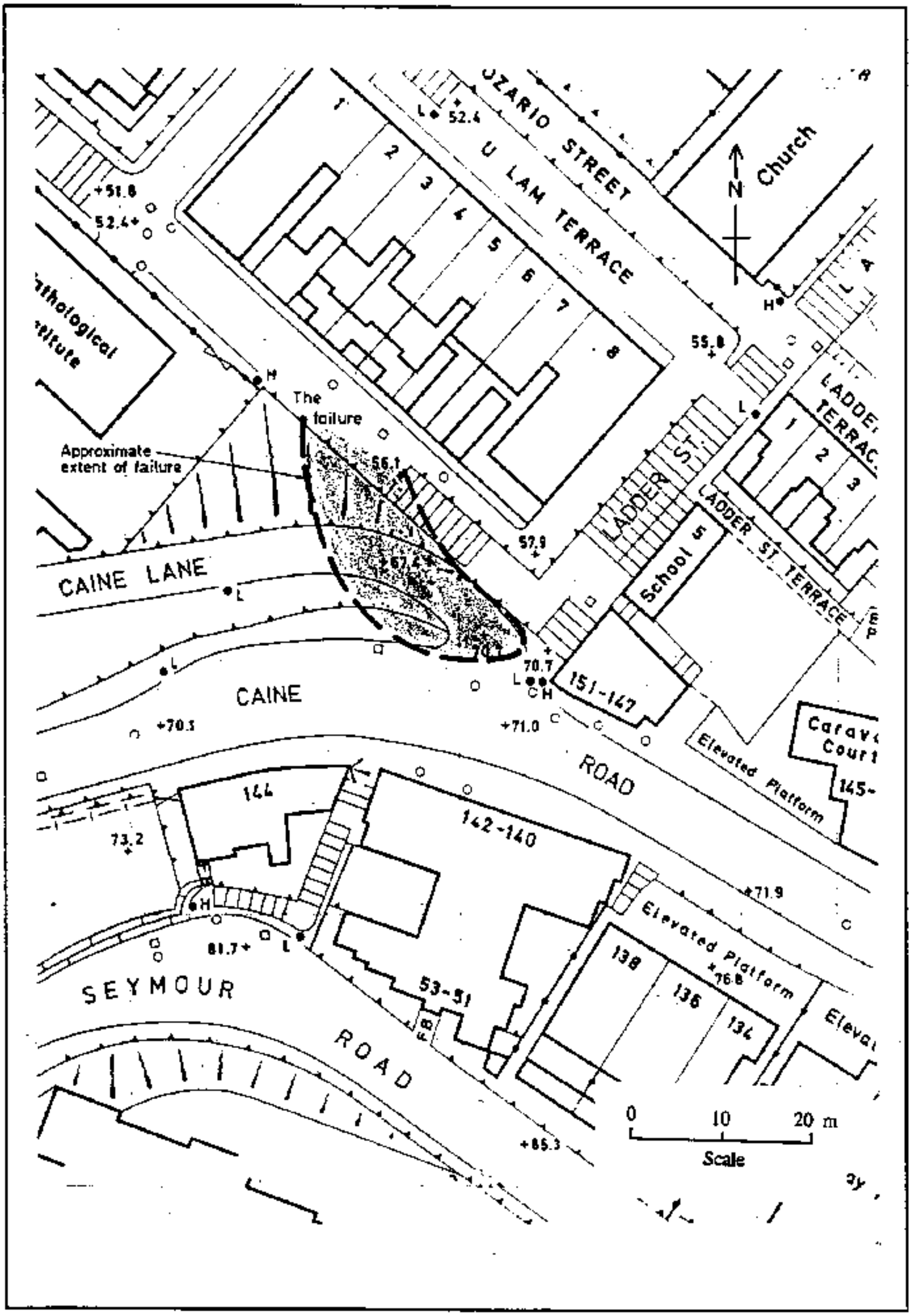


Figure D5.1 - Retaining Wall at Caine Lane - Site Plan

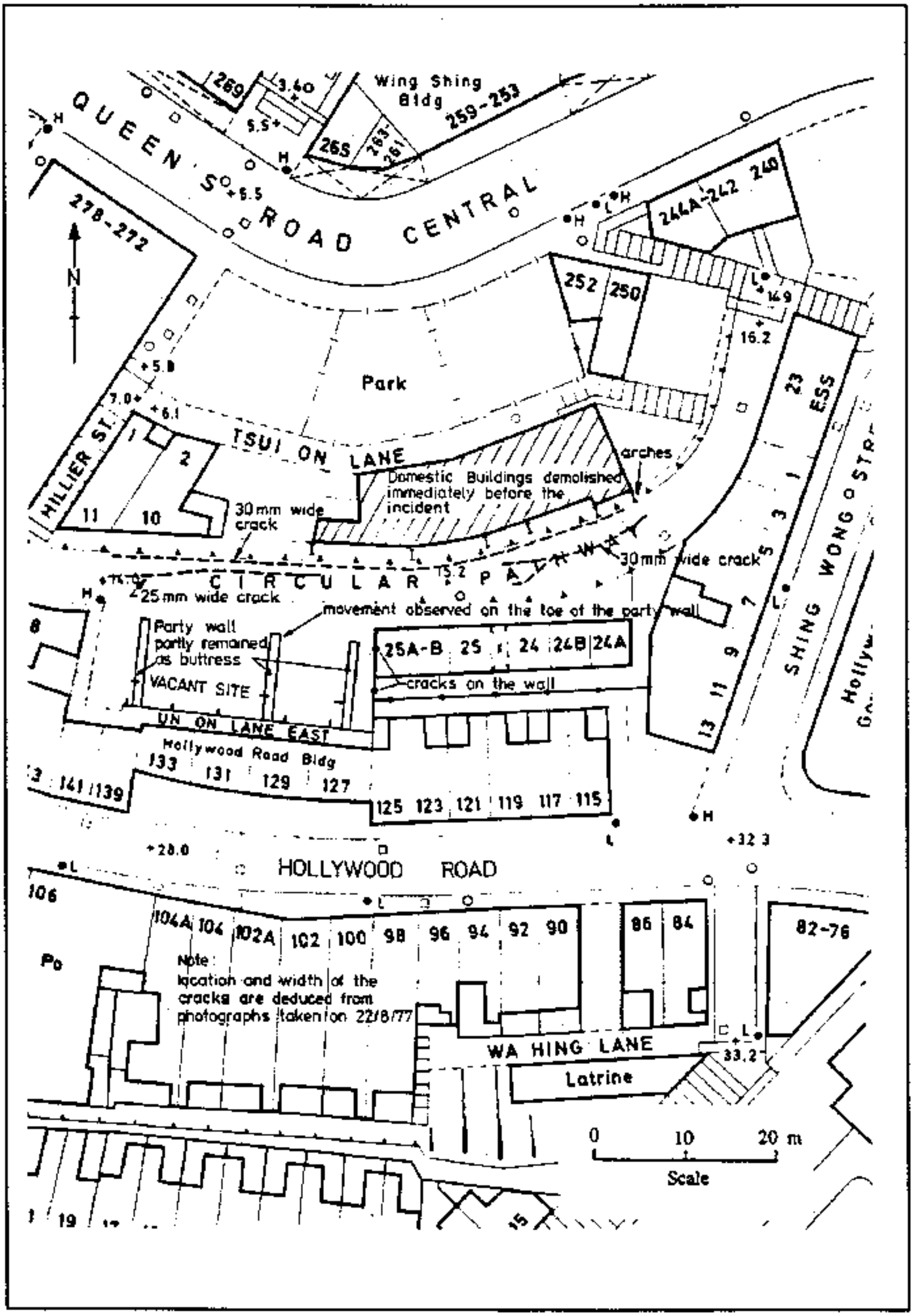


Figure D6.1 - Retaining Wall at Circular Pathway - Site Plan

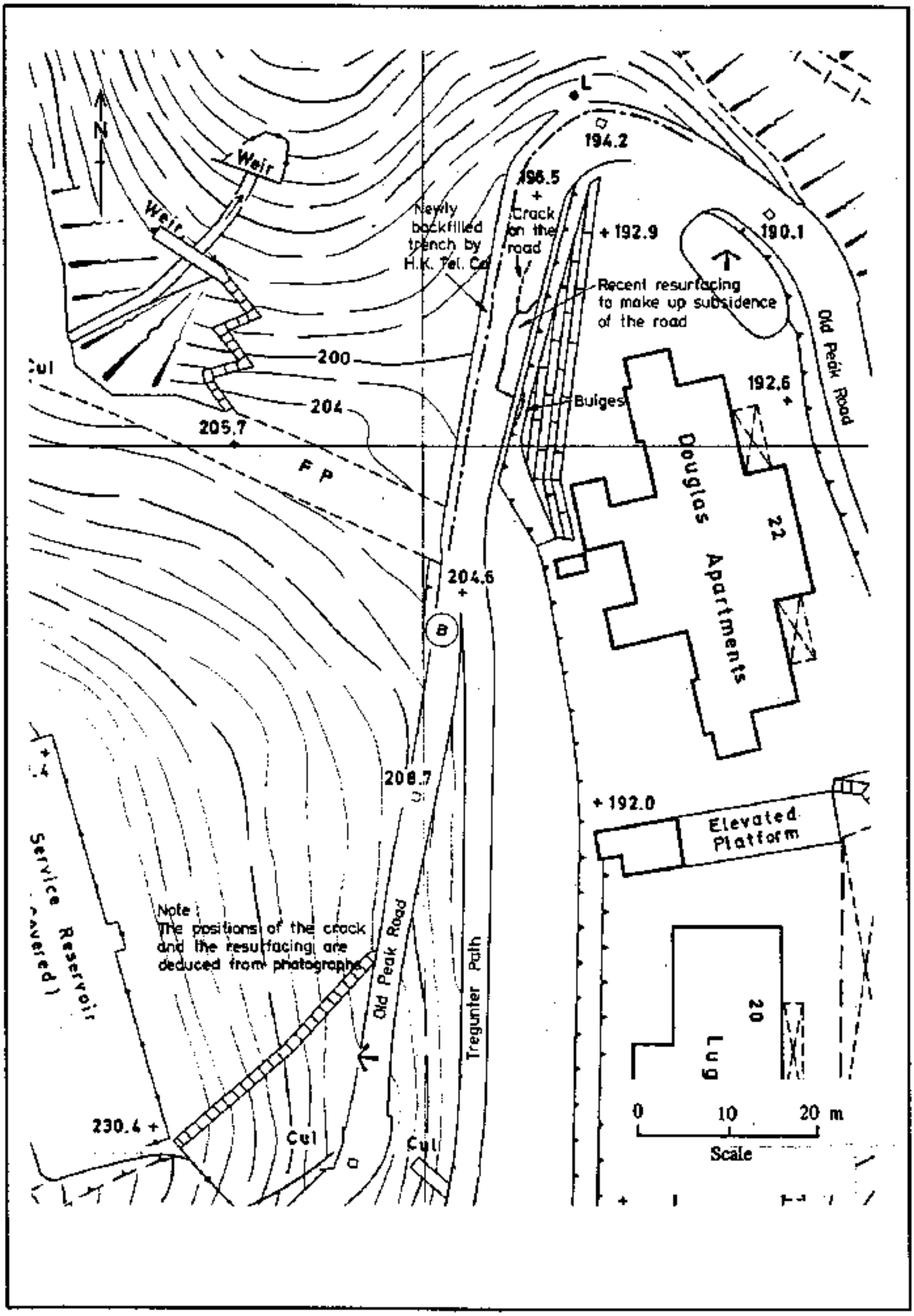


Figure D7.1 - Retaining Wall at 22, Old Peak Road - Site Plan

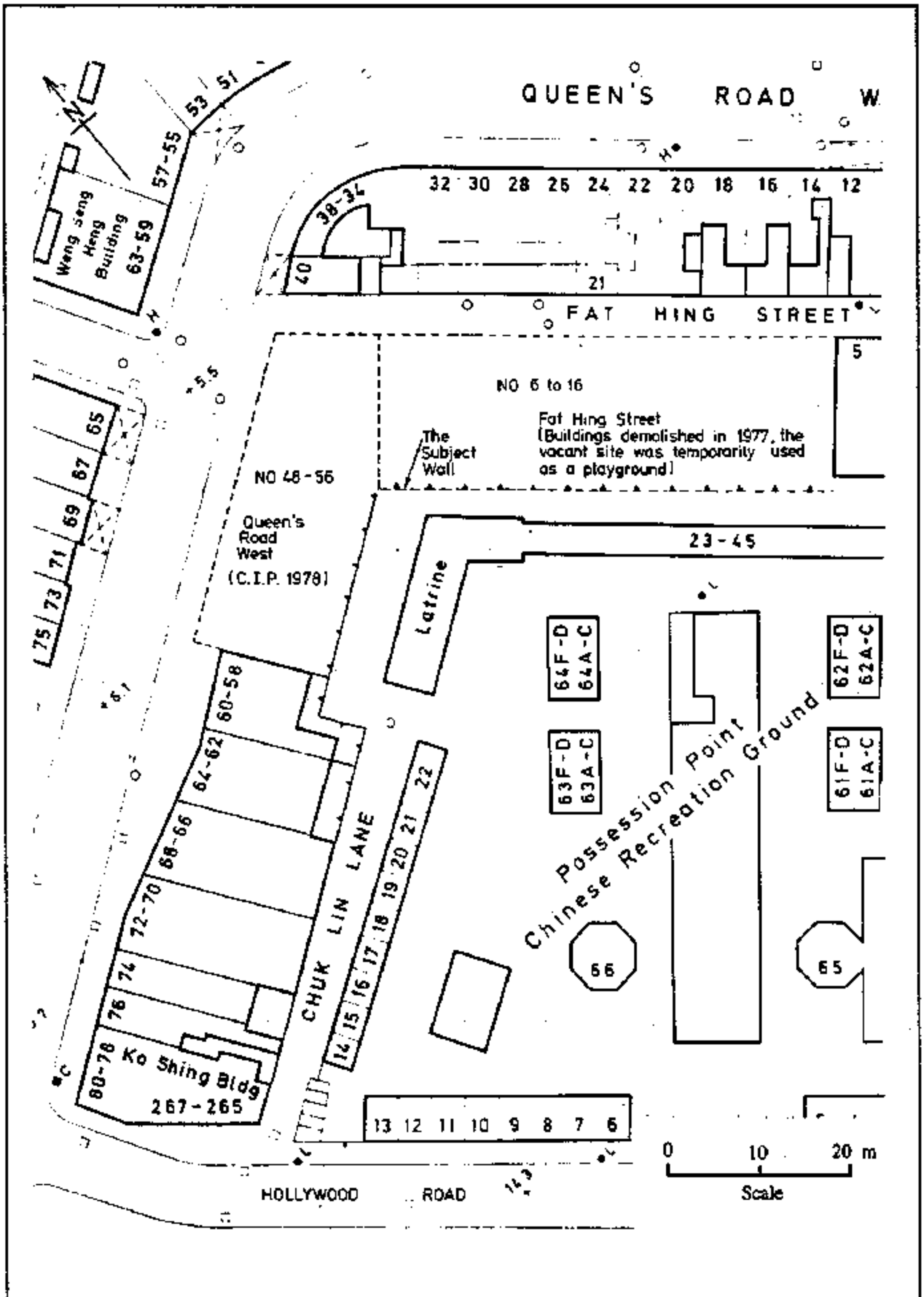


Figure D8.1 - Retaining Wall at Fat Hing Street - Site Plan

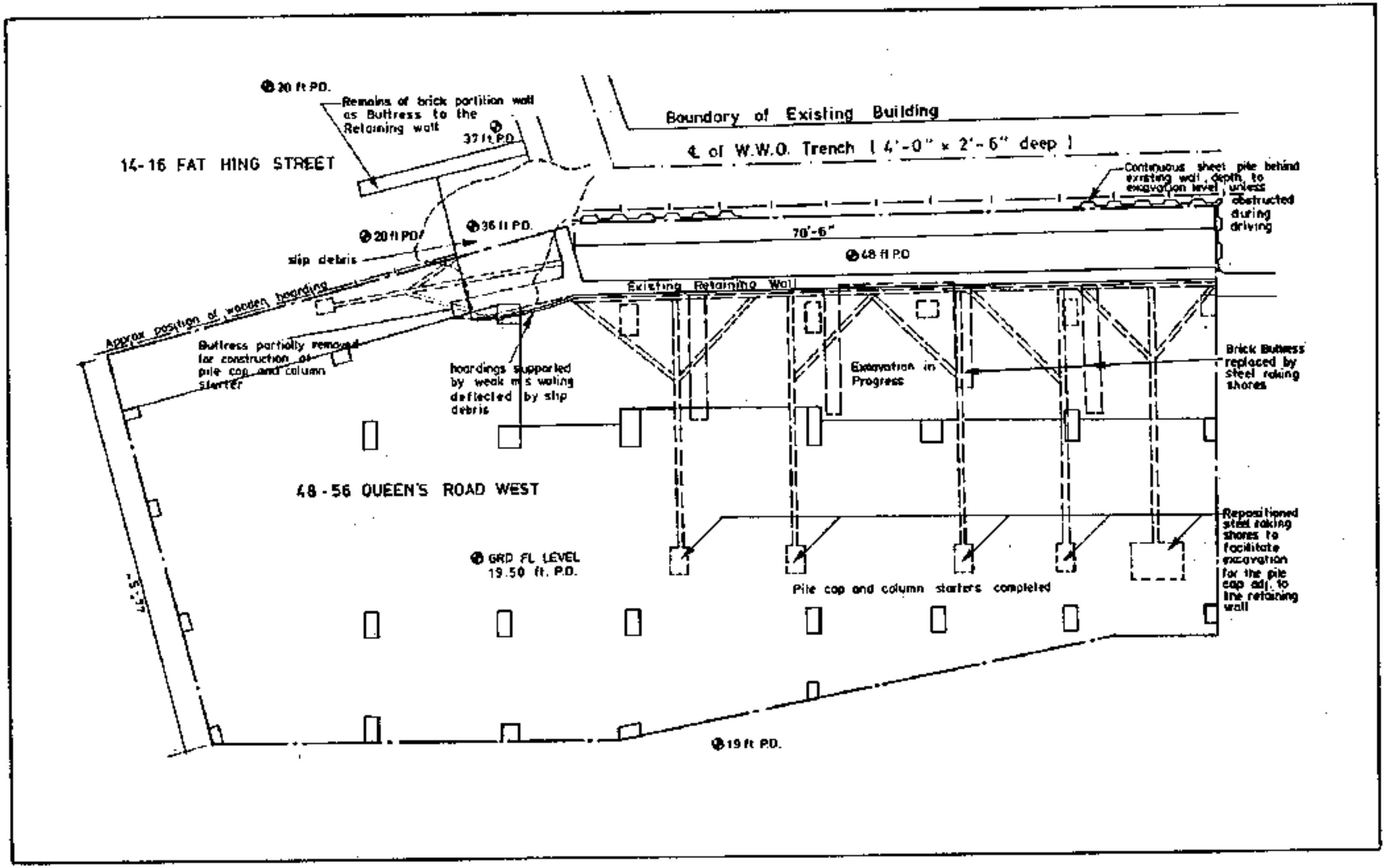


Figure D8.2 - Retaining Wall at Fat Hing Street - Approximate Extent of the Failed Wall and the Temporary Shorings on the Adjacent Site

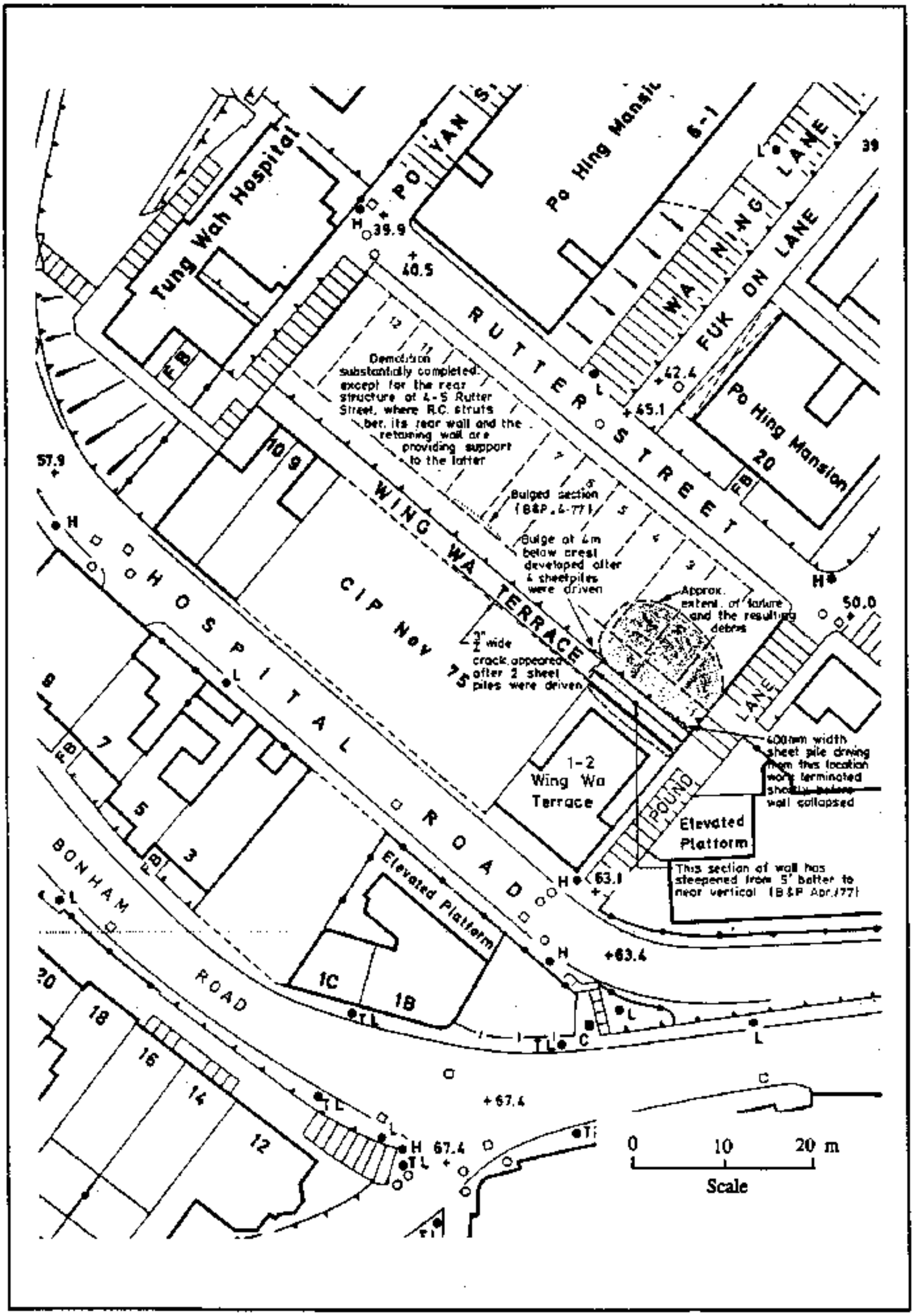
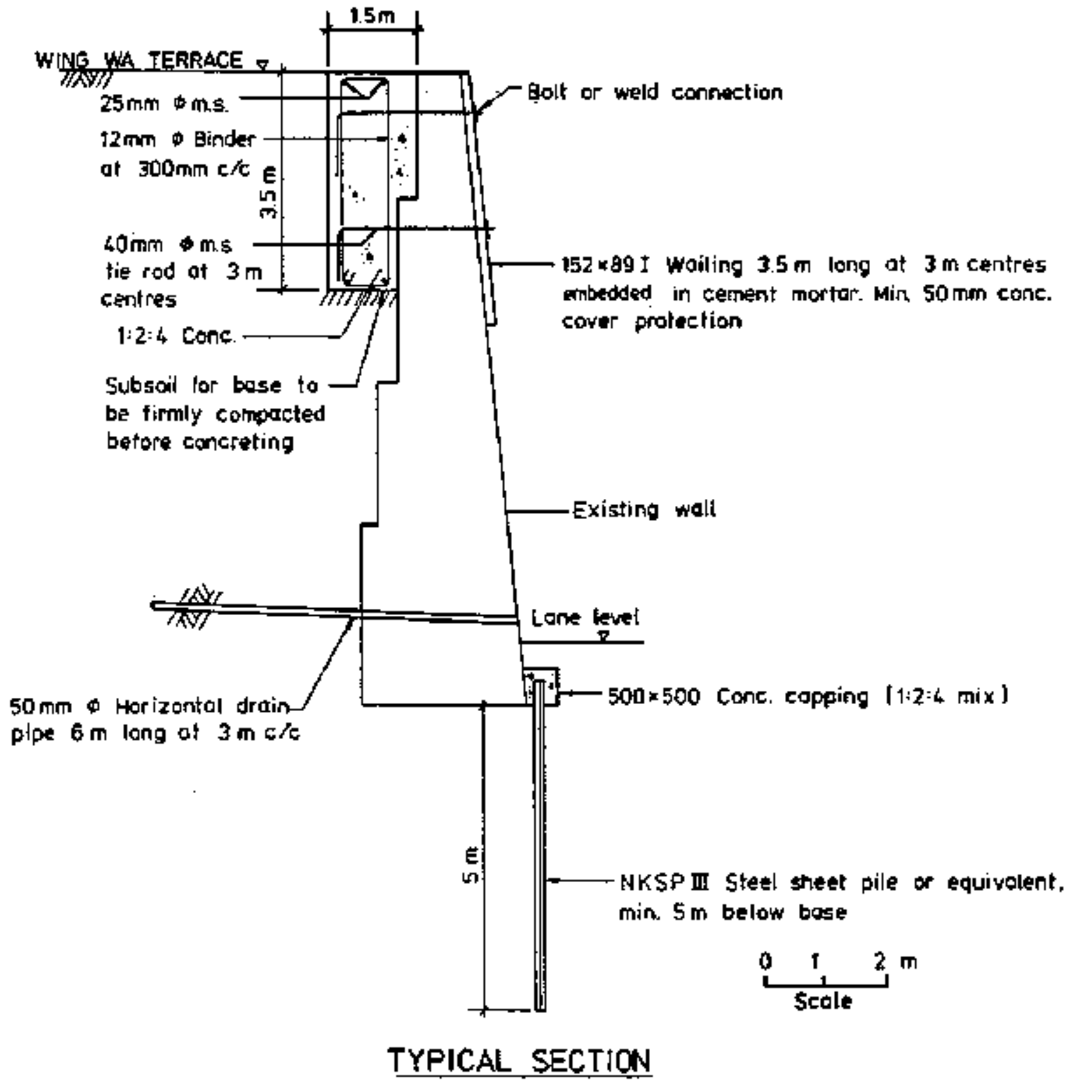


Figure D9.1 - Retaining Wall at Wing Wa Terrace - Site Plan



Notes:

1. Excavation to the conc. backing should be in stages of no longer than 6m at one time.
2. Adequate shoring and strutting should be provided to the excavation as necessary.
3. The excavated pit should be guarded against ponding of rain water and surface water.
4. Glass tell-tales should be installed on the face of the wall at positions to be instructed by the Engineer before driving sheet piles and to be monitored during the period of piling.
5. The horizontal drain holes should be installed first before the piling operation begins.
6. Any soil backfilling should be firmly compacted or replaced by a laterite material as to be instructed.

Figure D9.2 - Retaining Wall at Wing Wa Terrace - Proposed Stabilisation

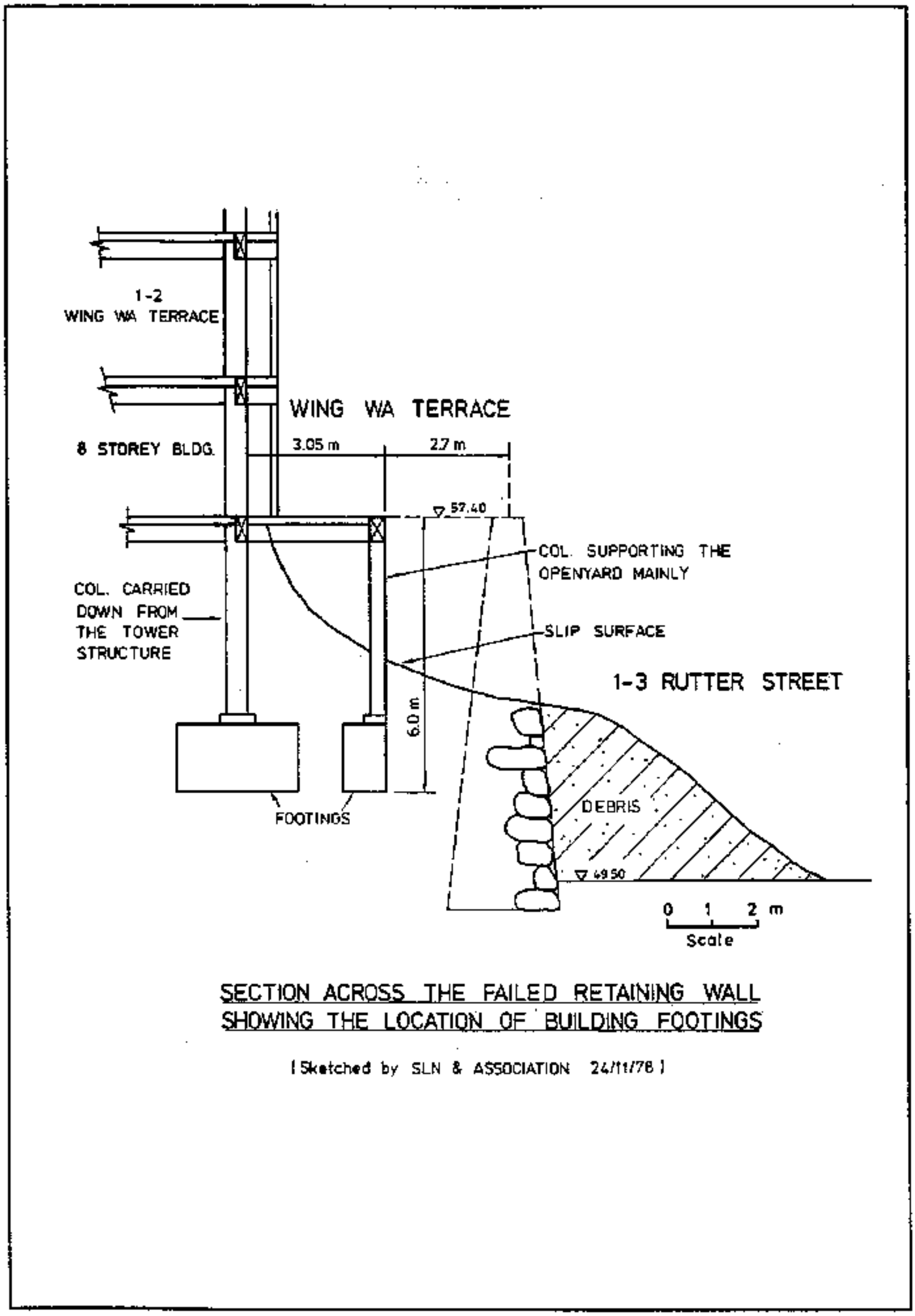


Figure D9.3 - Retaining Wall at Wing Wa Terrace - Section of the Failure

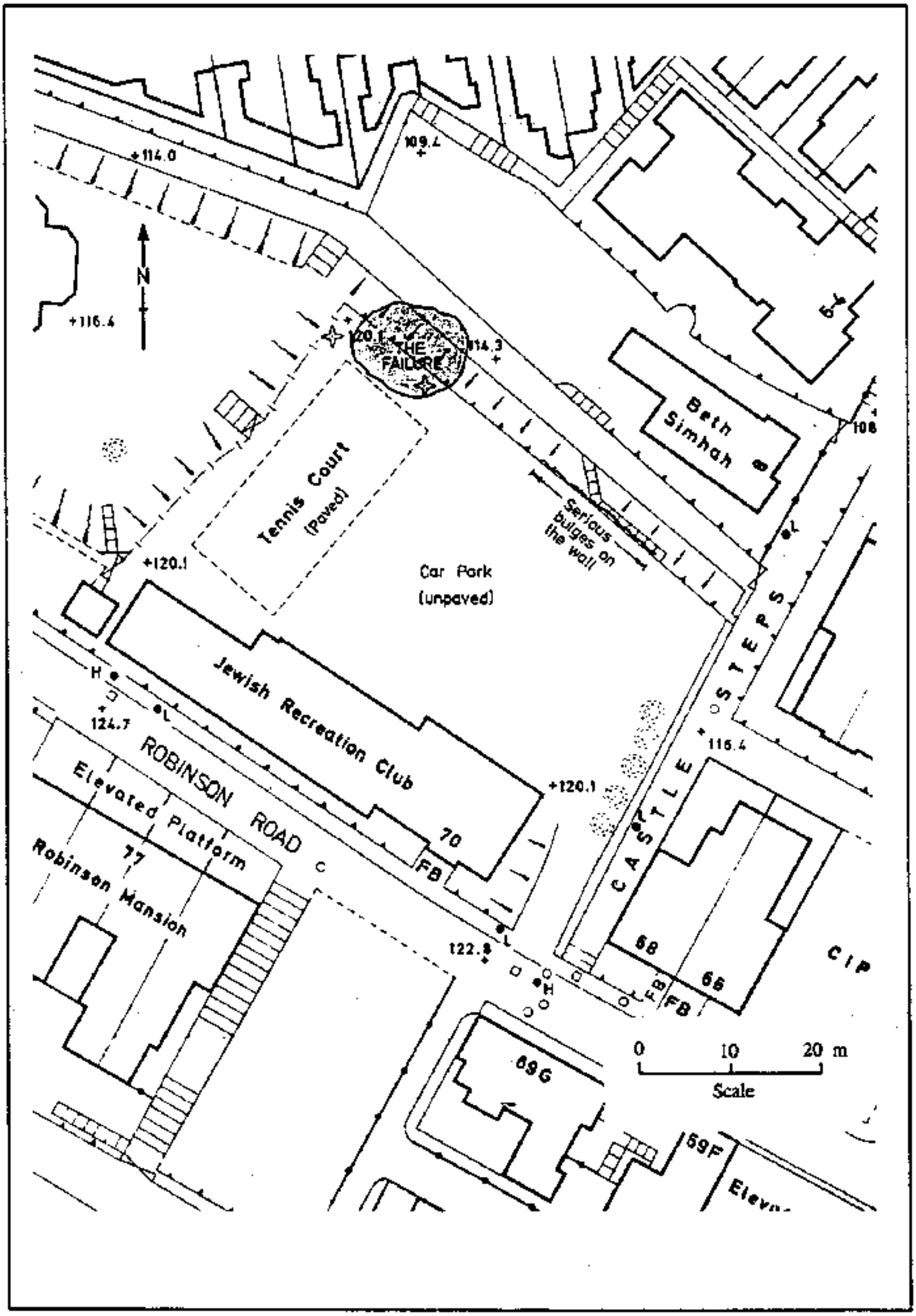
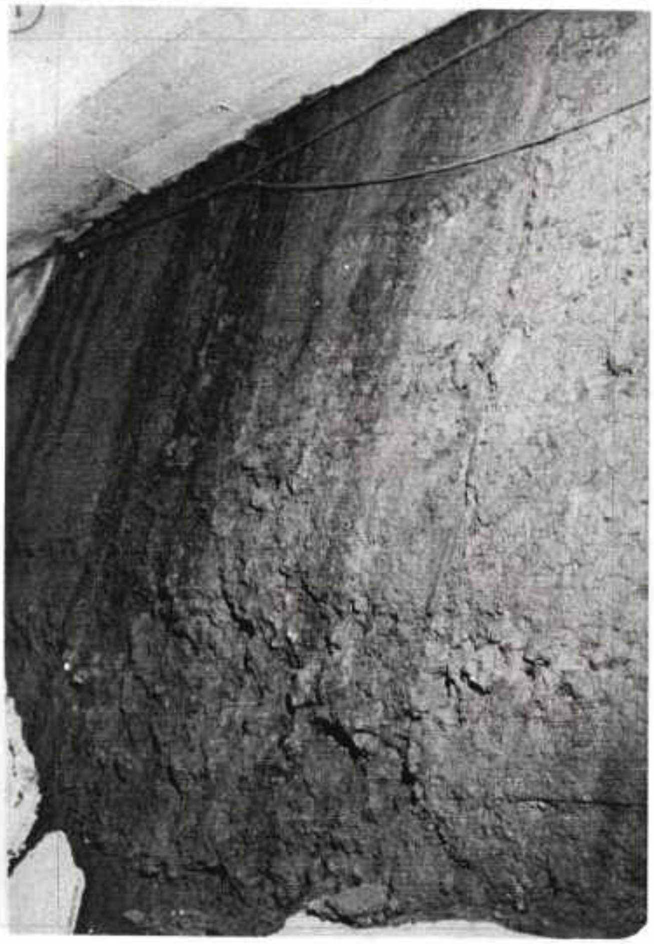


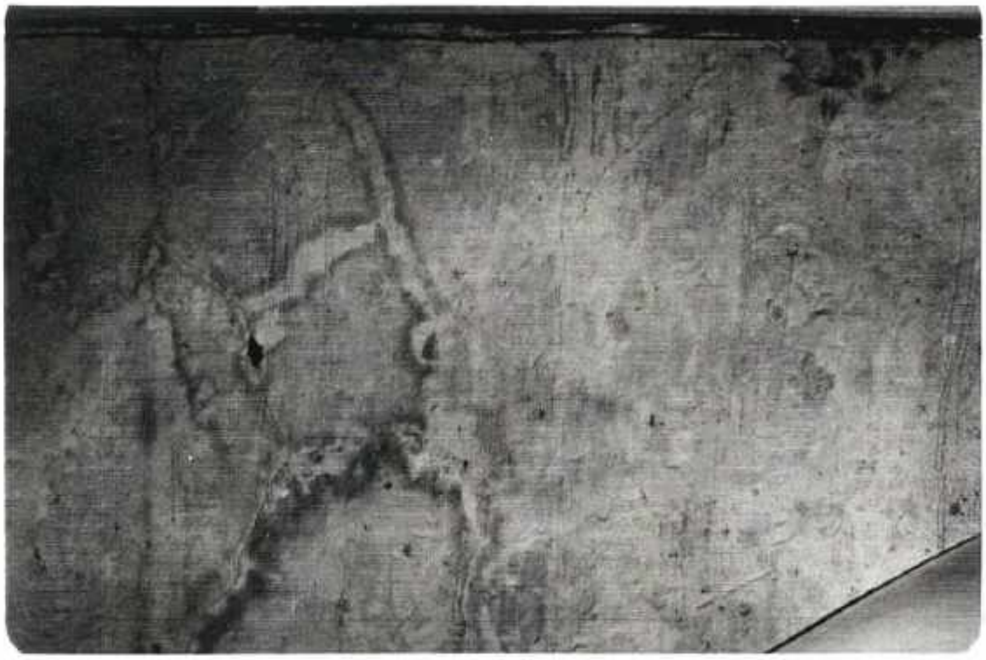
Figure D10.1 - Retaining Wall at the Jewish Recreation Club - Site Plan

LIST OF PLATES

Negative No. R/W 81.12.18

Plate D3.1 -
Retaining Wall at Castle Road -
Poor Quality Material of the Wall



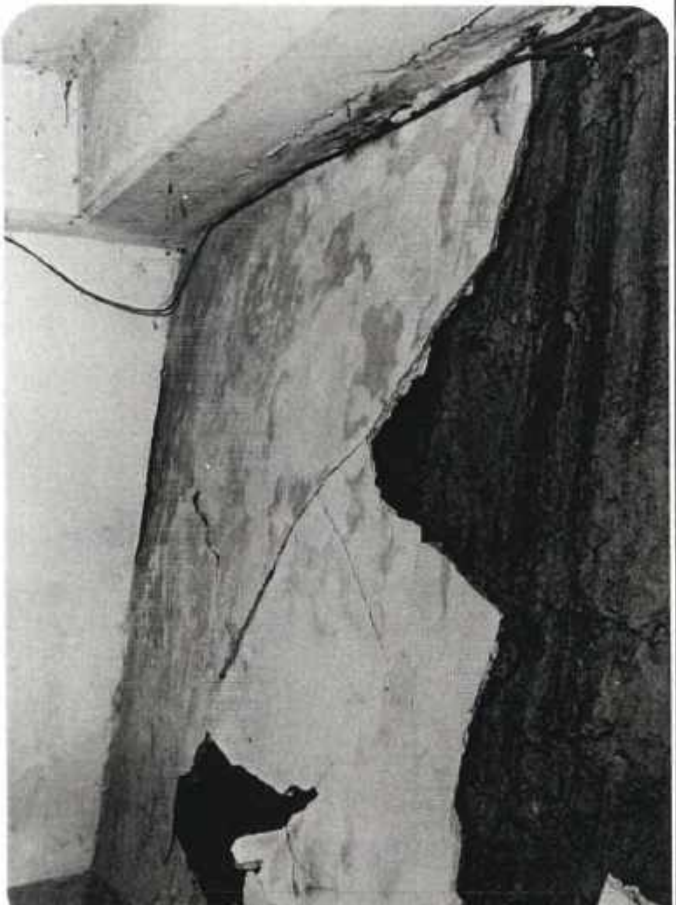


Negative No. R/W 81.12.15

Plate D3.2 - Retaining Wall at Castle Road - Cracks Induced on the Surface Finish by the Bulging of the Wall

Negative No. R/W 81.12.16

Plate D3.3 - Retaining Wall at Castle Road - Bulging of the Wall Caused Failure of the Plaster Layer



Negative No. R/W 81.12.17

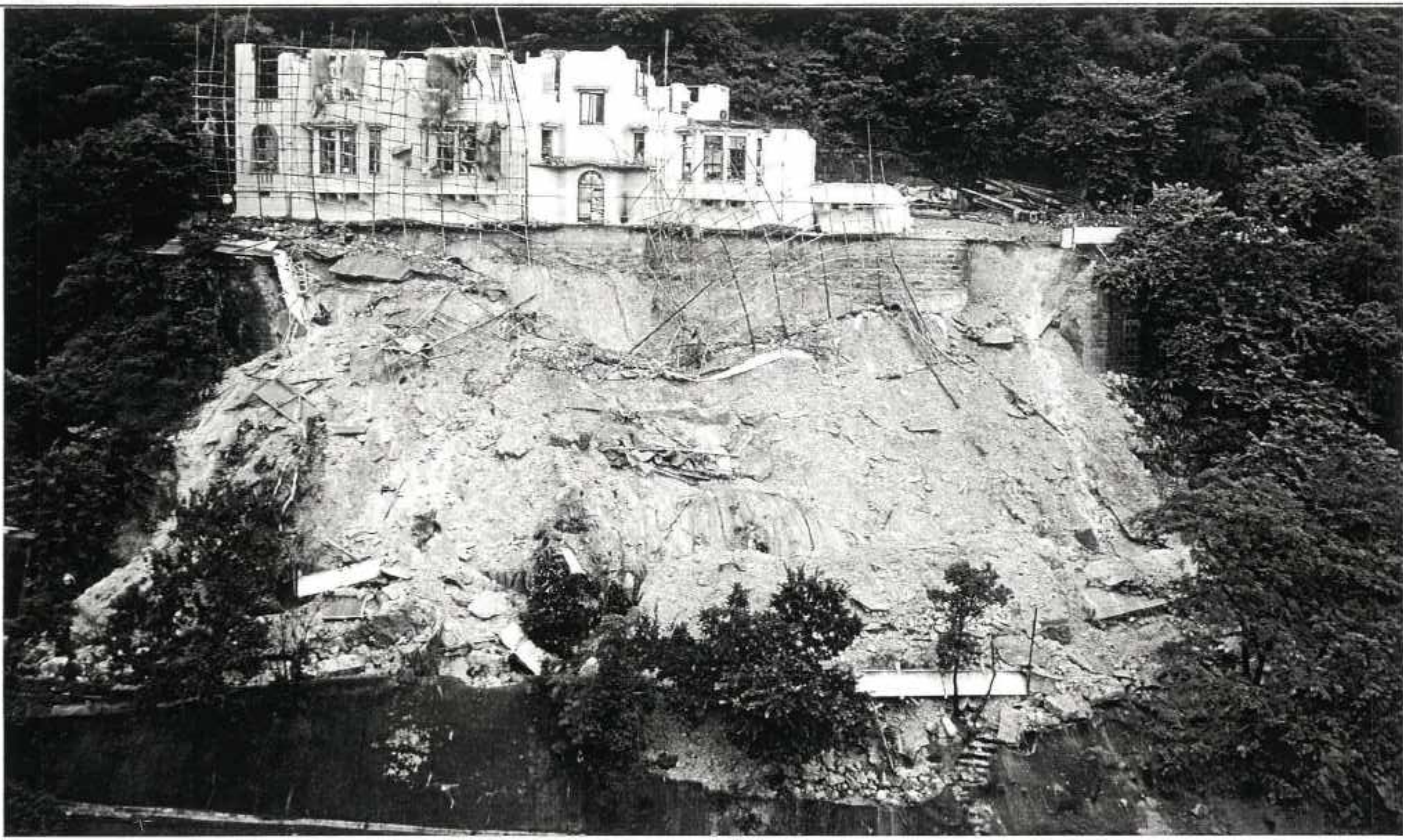
Plate D3.4 -
Retaining Wall at Castle Road -
Shear Cracks Caused by the
Pressure from the Yielding Retaining
Wall

Negative No. 7/98/73

Plate D4.1 - Retaining Wall at May Road - Photo of the Failure Taken from Grenville House (I)

Negative No. 8/98/73

Plate D4.2 - Retaining Wall at May Road - the Failure (II)

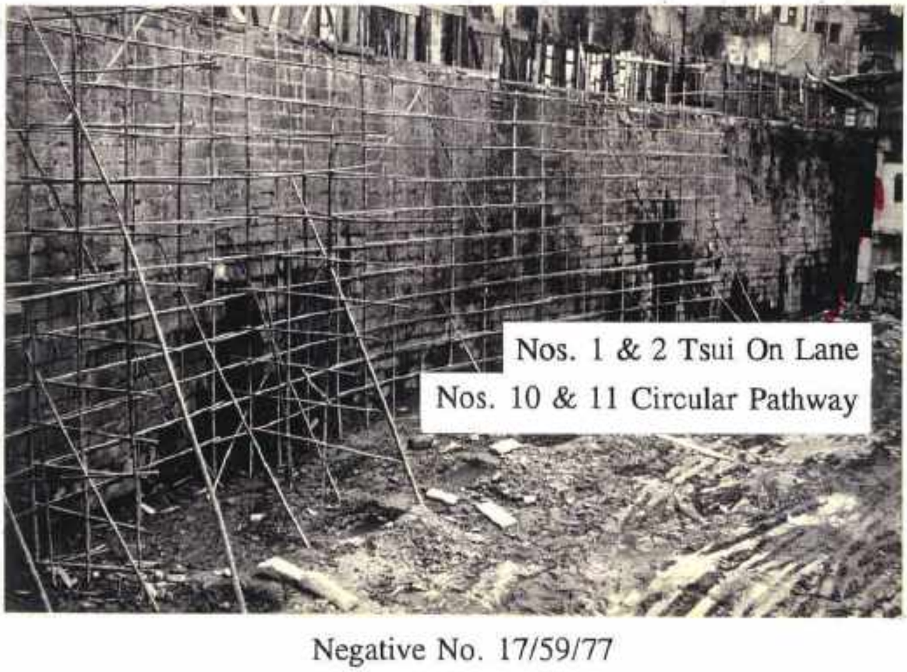


Negative No. 3/98/73

Plate D4.3 - Retaining Wall at May Road - the Failure (III)

Negative No. R/W 81.09.01

Plate D6.1 - Retaining Wall at Circular Pathway - the Wall at the End of the Demolition of the Houses on the Toe Platform



Negative No. 17/59/77

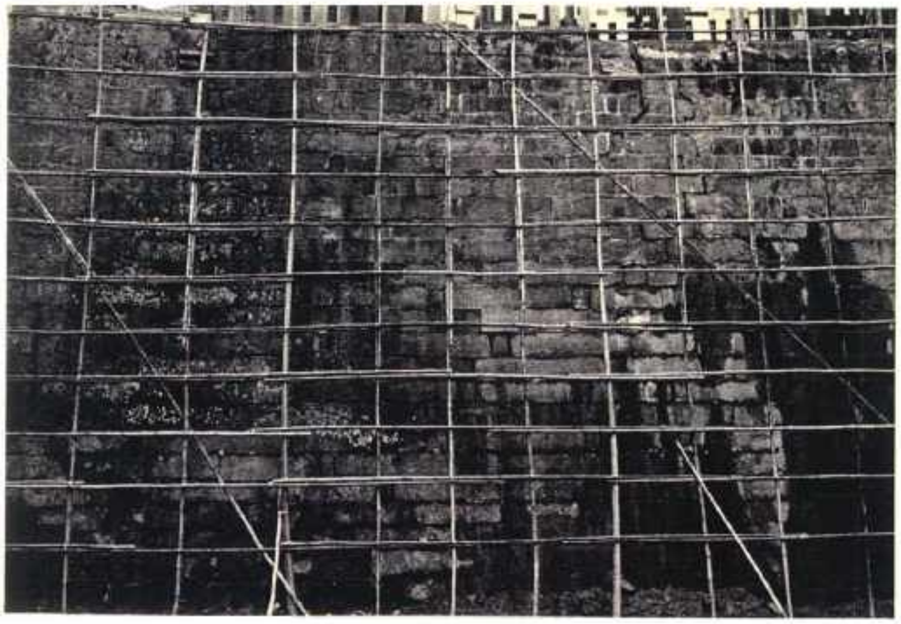
Plate D6.2 - Retaining Wall at Circular Pathway - Elevation of the Eastern Portion of the Wall

Negative No. 19/59/77

Plate D6.3 - Retaining Wall at Circular Pathway - Elevation of the Western Portion of the Retaining Wall

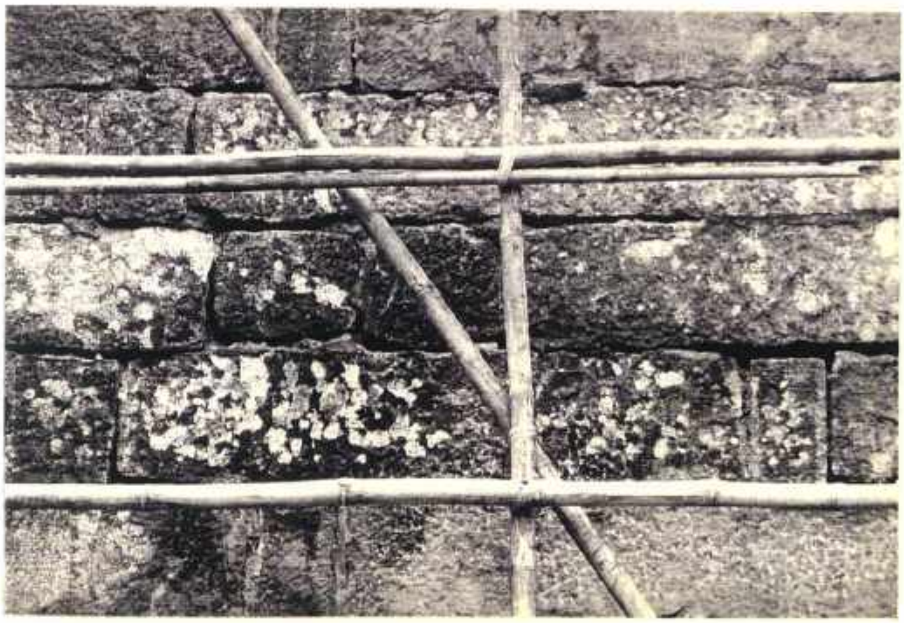
Negative No. 20/59/77

Plate D6.4 - Retaining Wall at Circular Pathway - Front Elevation



Negative No. 18/59/77

Plate D6.5 - Retaining Wall at Circular Pathway - Details of the Front Elevation (I)

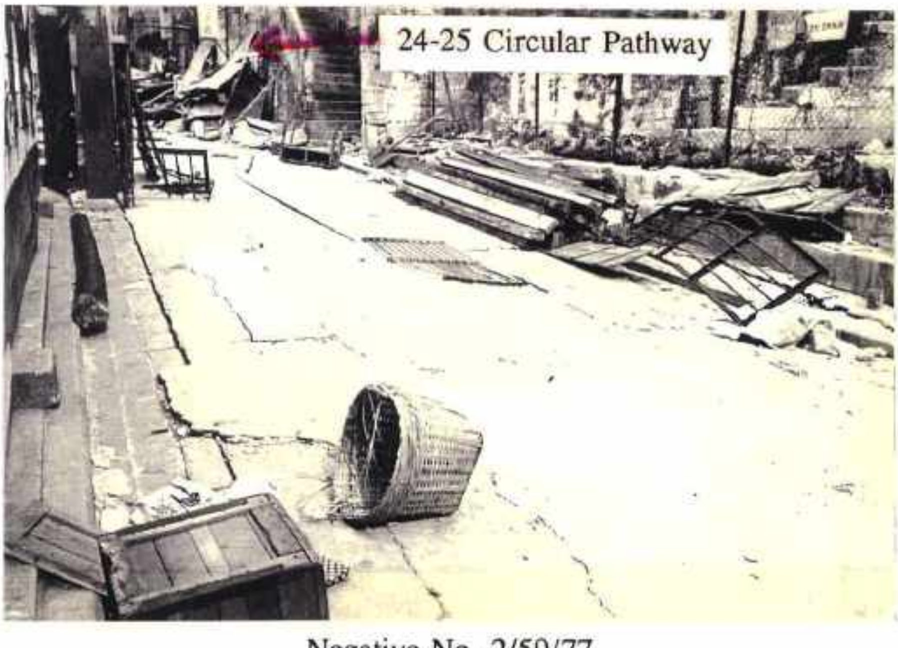


Negative No. 21/59/77

Plate D6.6 - Retaining Wall at Circular Pathway -Details of the Front Elevation (II)

Negative No. 1/59/77

Plate D6.7 - Retaining Wall at Circular Pathway - Western Portion of the Pathway, Looking East (I)



Negative No. 2/59/77

Plate D6.8 - Retaining Wall at Circular Pathway - Western Portion of the Pathway, Looking East (II)

Negative No. 3/59/77

Plate D6.9 - Retaining Wall at Circular Pathway - Western Portion of the Pathway, Looking West

Negative No. 9/59/77

Plate D6.10 - Retaining Wall at Circular Pathway - Eastern Portion of the Pathway, Looking East

Negative No. 8/59/77

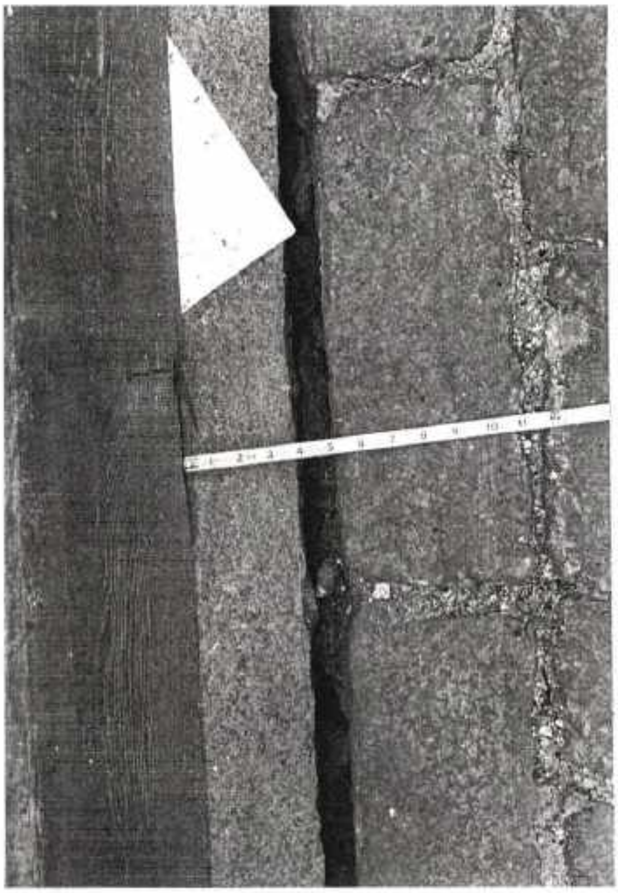
Plate D6.11 - Retaining Wall at Circular Pathway - Detail of the Cracks on the Pathway (I)

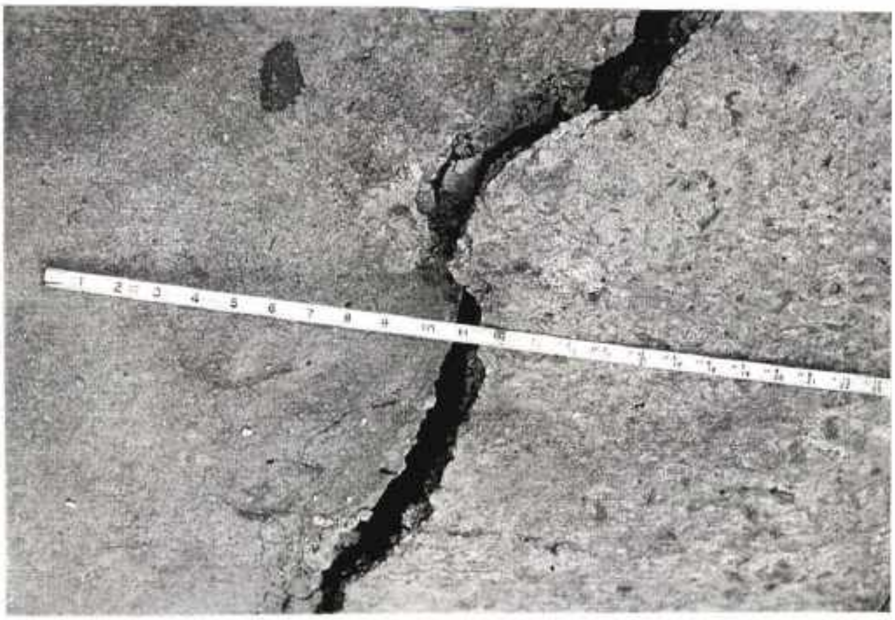
Negative No. 7/59/77

Plate D6.12 -
Retaining Wall at Circular Pathway - the Crack between the Wall and the Pathway

Negative No. 4/59/77

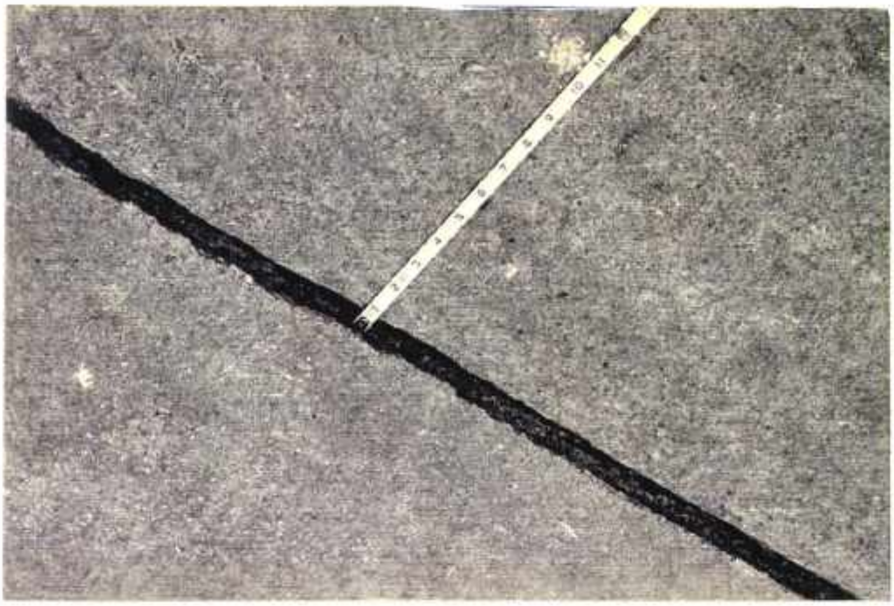
Plate D6.13 -
Retaining Wall at Circular Pathway -
Detail of the Cracks on the
Pathway (II)





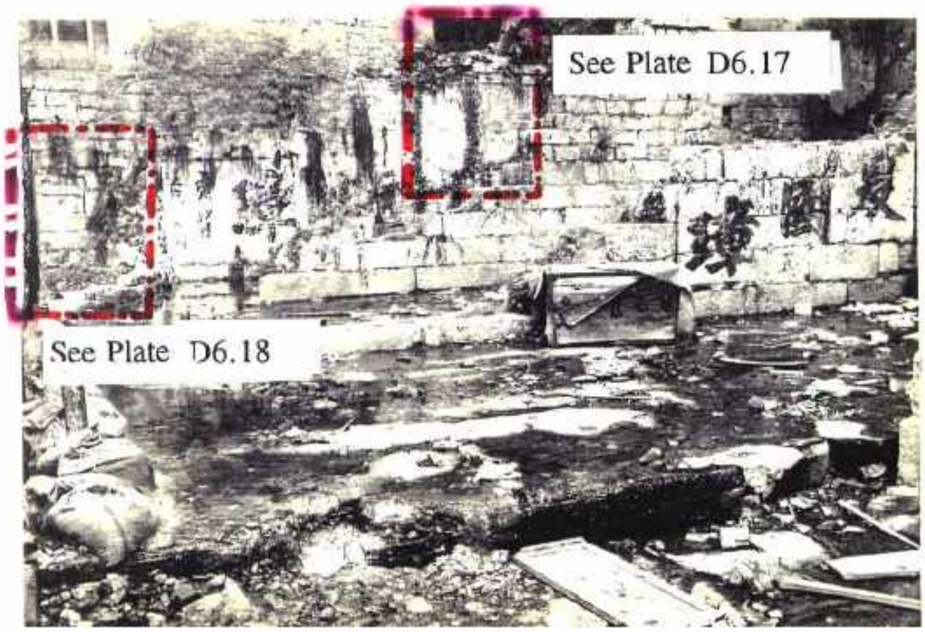
Negative No. 5/59/77

Plate D6.14 - Retaining Wall at Circular Pathway - Detail of the Cracks on the Pathway (III)



Negative No. 6/59/77

Plate D6.15 - Retaining Wall at Circular Pathway - Detail of the Cracks on the Pathway (IV)



Negative No. 15/59/77

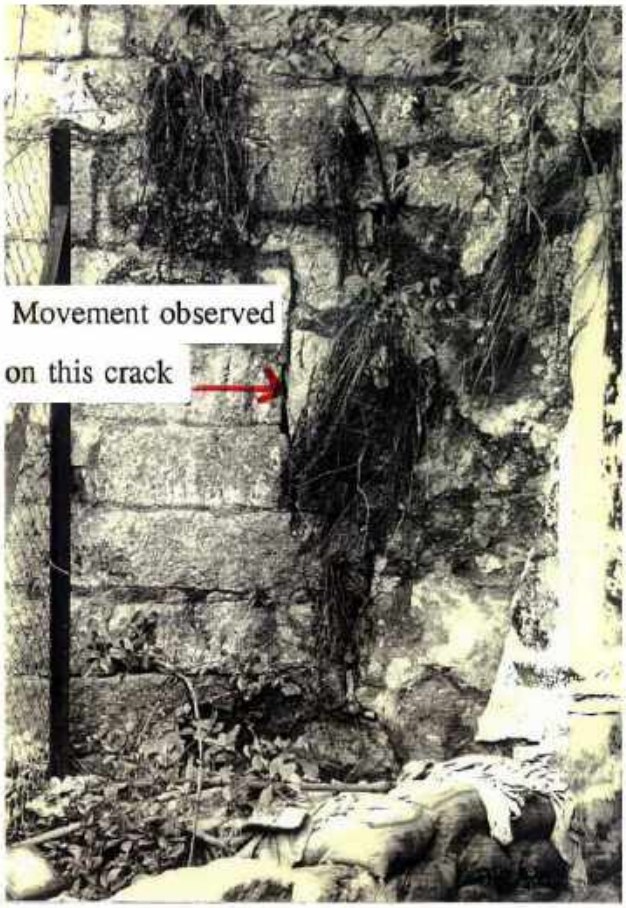
Plate D6.16 - Retaining Wall at Circular Pathway - Movements Observed on the Vacant Sites at the Crest Platform (below 127-133 Hollywood Road)

Negative No. 13/59/77

Plate D6.17 - Retaining Wall at Circular Pathway - Details of Movement Observed on the Site at the Crest Platform (I)

Negative No. 12/59/77

Plate D6.18 -
Retaining Wall at Circular Pathway -
Details of Movement Observed on
the Site at the Crest Platform (II)



Negative No. 14/59/77

Plate D6.19 - Retaining Wall at Circular Pathway - Details of Movement Observed on the Site at Crest Platform (III)

Negative No. 11/59/77

Plate D6.20 - Retaining Wall at Circular Pathway - Details of Movement Observed on the Site at the Crest Platform (IV)

Negative No. R/W 81.12.13

Plate D7.1 - Retaining Wall at 22, Old Peak Road - Bulging of the Retaining Wall

Negative No. R/W 81.12.11

Plate D7.2 - Retaining Wall at 22, Old Peak Road - Bulged Parapet at the Crest of the Wall

Negative No. R/W 81.12.12

Plate D7.3 - Retaining Wall at 22, Old Peak Road - Recent Road Works on Old Peak Road

Negative No. R/W 81.12.09

Plate D7.4 - Retaining Wall at 22, Old Peak Road - Longitudinal Crack on Old Peak Road

Negative No. R/W 81.17.17

Plate D8.1 - Retaining Wall at Fat Hing Street - View of the Wall before Work Commenced on the Adjacent Site (I)

Negative No. R/W 81.17.20

Plate D8.2 -
Retaining Wall at Fat Hing Street -
View of the Wall before Work
Commenced on the Adjacent
Site (II)

Negative No. R/W 81.17.19

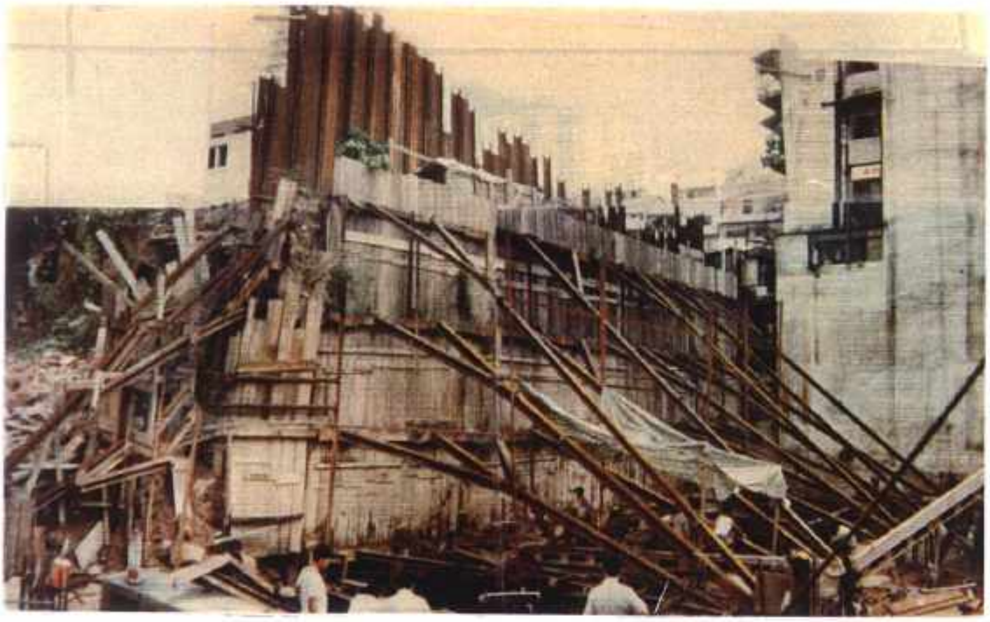
Plate D8.3 - Retaining Wall at Fat Hing Street - Panoramic View of the Shoring to the Tied Face Wall at the Adjacent Site (48-56, Queen's Road West)

Negative No. R/W 81.17.14

Plate D8.4 - Retaining Wall at Fat Hing Street - View of the Shoring Adjacent to the Portion of the Wall Which Later Failed

Negative No. R/W 81.17.13

Plate D8.5 -
Retaining Wall at Fat Hing Street -
Trench Work at the Crest Platform



Negative No. R/W 81.17.18

Plate D8.6 - Retaining Wall at Fat Hing Street - Panoramic View of the Shoring to the Tied Face Wall at the Adjacent Site, after the Incident

Negative No. R/W 81.17.16

Plate D8.7 -
Retaining Wall at Fat Hing Street - Enlarged View of the Collapsed Wall and the Deflected Shoring Adjacent

Negative No. R/W 81.09.04

Plate D9.1 -
Retaining Wall at Wing Wa Terrace
- the Wall before Failure

Negative No. R/W 81.09.16

Plate D9.2 - Retaining Wall at Wing Wa Terrace - the Failure, Looking South-East

Negative No. R/W 81.12.08

Plate D9.3 - Retaining Wall at Wing Wa Terrace - the Failure, Looking South

Negative No. R/W 81.12.00

Plate D9.4 - Retaining Wall at Wing Wa Terrace - the Failure, Looking South-West

Negative No. R/W 81.12.05

Plate D9.5 - Retaining Wall at Wing Wa Terrace - the Failure, Looking West

APPENDIX E

STRENGTH OF MASONRY :

AN ABSTRACT OF RELEVANT TABLES AND CLAUSES FROM BUILDING STANDARDS

E.1 Note

In this appendix, tables and clauses are presented and quoted according to their reference numbers in the original building standards.

E.2 The Chinese Specifications on Masonry Designs (Draft), 1973
Specification No. GBJ 3-73

E.2.1 Compressive Strength of Masonry

See Tables E1, E2 & E3

E.2.2 Tensile Strength

See Table E4

E.2.3 Shear Strength

See Table E5

E.3 Code of Practice for Structural Use of Masonry
BS 5628 : Part 1 : 1978

E.3.1 General

The BS 5628 : I : 1978 uses the limiting state design concept which is different from the load factor and permissible stress concept used in the Chinese and American building standards. The strength values given in this code are characteristic strengths with a level of confidence of 95%. Sizes of structural members are so designed that the combined effects of the loadings do not cause stresses higher than the characteristic strength. Two partial safety factors, m, f are introduced in the calculation to allow for inferior quality control on site, unusual increase in loading, inaccurate structural analyses and inaccuracy in member dimensions. The usual calculation procedure is summarised in the flow chart in Figure E1.

In masonry design, f has an average value of 1.4 (Clause 22). The value of m varies from 2.5 to 3.5 (Clause 27.3) depending on the degree of quality control. Under normal situations, the combined effect of these two partial factors is equivalent to a safety factor of 4.2.

E.3.2 Compressive Strength of Masonry

Clause 23 Characteristic Compressive Strength of Masonry, f_k

Clause 23.1 Normal masonry. The characteristic compressive strength, f_k, of any masonry may be determined by tests on wall specimens, following the procedures laid down in A.2.

For normally bonded masonry, defined in terms of the shape and compressive strength of the structural units and the designation of the mortar (see Table E6), the values given in Table E7 inclusive may be taken to be the characteristic compressive strength, f_k, of walls constructed under laboratory conditions tested at an age of 28 days under axial compression in such a manner that the effects of slenderness may be neglected. Linear interpolation within the tables is permitted.

Table E7(a) applies to masonry built with standard format bricks complying with the requirements of BS 187, BS 1180 or BS 3921.

Table E7(b) applies to masonry built with structural units with a ratio of height to least horizontal dimension of 0.6.

Table E7(c) applies to structural units, other than solid concrete blocks, with a ratio of height to least horizontal dimension of between 2.0 and 4.0, and makes due allowance for the enhancement in strength resulting from the unit shape.

Table E7(d) applies to solid concrete blocks, i.e. those without cavities, with a ratio of height to least horizontal dimension of between 2.0 and 4.0, and makes due allowance for the enhancement in strength resulting from the unit shape.

Clause 23.1.1 Walls or columns of small plan area. Where the horizontal cross-sectional area of a loaded wall or column is less than 0.2 m², the characteristic compressive strength should be multiplied by the factor :

$$(0.70 + 1.5A)$$

where A is the horizontal loaded cross-sectional area of the wall or column (m²)

Clause 23.1.8 Natural stone masonry. Natural stone masonry should be designed on the basis of solid concrete blocks of an equivalent compressive strength. Where masonry is constructed from large, carefully shaped pieces with relatively thin joints, its load bearing capacity is more closely related to the intrinsic strength of the stone than is the case where small structural units are used. Design stresses in excess of those obtained from this code may be allowed in such massive stone masonry, provided that the designer is satisfied that the properties of the stone warrant an increase.

Clause 23.1.9 Random rubble masonry. The characteristic strength of random rubble masonry may be taken as 75% of the corresponding strength of natural stone masonry built with similar materials. In the case of rubble masonry built with lime mortar, the characteristic strength may be taken as one-half of that for masonry in mortar designation (iv).

E.3.3 Tensile Strength of Masonry

Clause 24 Characteristic Flexural Strength of Masonry, f_{kx}

Clause 24.1 General. The characteristic flexural strength, f_{kx}, should be used only in the design of masonry in bending. In general, no direct tension should be allowed in masonry. However, at the designer's discretion half the values in Table E8 may be allowed in direct tension when suction forces arising from wind loads on roof structures are transmitted to masonry walls, or when the probable effects of misuse or accidental damages (see Section 5) are being considered. In no circumstances may the combined flexural and direct tensile stresses exceed the values given in Table E8.

Flexural tension should be relied on at a damp proof course only if the damp proof course consists of a material which had been proved by tests to permit the joint to transmit tension or if it is of bricks complying with the requirements of BS 743.

E.3.4 Shear Strength of Masonry

Clause 25. Characteristic Shear Strength of Masonry, f_v

The characteristic shear strength f_v, of masonry may be taken as $0.35 + 0.6g_A$ N/mm² with a maximum of 1.75 N/mm² for walls built in mortar designations (i), (ii) or (iii) or $0.15 + 0.6g_A$ N/mm² with a maximum of 1.4 N/mm² for walls built in mortar designation (iv), where g_A is the design vertical load per unit area of wall cross section due to the vertical dead and imposed loads calculated from the appropriate loading condition specified in Clause 22.

Clause 26. Coefficient of friction

The coefficient of friction between clean concrete and masonry faces may be taken as 0.6.

E.4 The American Specifications on Strength of Masonry
(After Cross & Brennan, 1976)

E.4.1 Compressive Strength

See Tables E9 & E10

LIST OF TABLES

Table E1 - Compressive Strength of Masonry Constructed with Ashlars or Squared Rubble

Compressive Strength of Units (MPa)	Compressive Strength of Mortar (MPa)			
	5	2.5	1.0	0
100	34	31	29	25
80	28	25.5	23.5	20
60	22	20	18	15
50	19	17	15	12.5
40	15.5	14	12.5	10
30	12.5	11	9.5	7.5
20	9	7.5	6.5	5
15	7	6	5	3.5
10	5	4.5	3.5	2.5
7.5	4	3.5	3	2
5.0	3	2.5	2	1.2

Notes: (1) The table applies to masonry with heights of building blocks (h) equal to 400 mm

For $150 < h < 400$ apply modification factor

$$C = 0.4 + 0.0015\,h$$

For $h > 400$ apply modification factor

$$C = 1 + 0.0004\,(h - 400)\ 1.2$$

(2) For different shapes of blocks, apply different modification factors.

Ashlar	1.0
Coarse ashlar	0.7
Squared rubble	0.6

(3) If pure cement/sand mortar is used, apply a modification factor of 0.85.

(4) For permissible strength, apply a safety factor of 2.3 (Table 13 of Chinese Specification GBJ 3-73).

(5) This Table is reproduced from Table 3 of Chinese Specification GBJ 3-73.

Table E2 - Compressive Strength of Masonry Constructed with Random Rubble

Compressive Strength of Units (MPa)	Compressive Strength of Mortar (MPa)					
	10	5	2.5	1	0.4	0
100	7.3	5.5	4.2	3	2.3	1
80	6.5	4.8	3.1	2.6	1.9	0.8
60	5.5	4.1	3	2.1	1.6	0.6
50	5	3.6	2.7	1.9	1.4	0.5
40	4.4	3.2	2.4	1.7	1.2	0.4
30	3.8	2.7	2	1.4	1	0.3
20	3	2.2	1.6	1.1	0.8	0.2
15	2.6	1.9	1.4	0.9	0.6	0.15
10	2.1	1.5	1.1	0.7	0.5	0.1

Notes :
(1) If pure cement/sand mortar is used, apply a modification factor of 0.85.
(2) For permissible strength, apply a safety factor of 3.0 (Table 13 of Chinese Specification GBJ 3-73).
(3) This Table is reproduced from Table 4 of Chinese Specification GBJ 3-73.

Table E3 - Compressive Strength of Masonry Constructed with Standard Format Bricks

Compressive Strength of Bricks (MPa)	Compressive Strength of Mortar (MPa)					
	10	5	2.5	1	0.4	0
30	7	6	5.2	4.5	4.0	3.3
25	6.3	5.3	4.5	3.9	3.5	2.8
20	5.5	4.6	3.9	3.3	2.9	2.3
15	4.7	3.8	3.2	2.7	2.4	1.8
10	3.8	3.1	2.5	2.1	1.8	1.3
7.5	-	2.7	2.2	1.8	1.5	1.0
5	-	2.2	1.8	1.4	1.2	0.7

Notes : (1) Nominal dimensions of brick 240 x 115 x 53 mm^3

(2) If special size bricks are used, apply modification factor

$$c = 2\sqrt{\frac{1}{10}\left(\frac{h+7}{L}\right)}$$

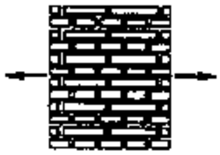
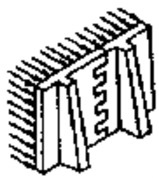
where h, L, are the height and length of the brick in mm.

(3) If pure cement/sand mortar is used, apply a modification factor of 0.85.

(4) For permissible strength apply a safety factor of 2.3 (Table 13 of Chinese Specification GBJ 3-73).

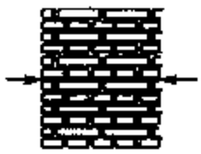
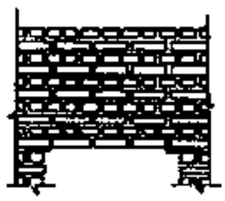
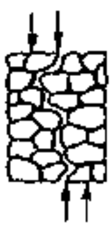
(5) This Table is reproduced from Table 1 of Chinese Specification GBJ 3-73.

Table E4 - Permissible Direct and Flexural Tensile Strength of Masonry (Failure along Joints)

Nature of Stress	Failure Mode	Type of Masonry	Compressive Strength of Mortar (MPa)				
			10	5	2.5	1	0.4
Direct Tension	 Failure along saw-tooth path	Bricks	0.4	0.3	0.25	0.15	0.09
		Random Rubble	0.25	0.2	0.18	0.1	0.05
Flexural Tension	 Plane of failure perpendicular to bed joints	Bricks	0.7	0.55	0.4	0.25	0.15
		Random Rubble	0.5	0.4	0.3	0.2	0.1
	 Plane of failure parallel to bed joints	Bricks	0.4	0.3	0.2	0.12	0.06

Notes :
(1) Table not applicable to squared rubble and ashlar walls.
(2) If pure cement/sand mortar is used, apply a modification factor of 0.75.
(3) For permissible strength, apply a safety factor of 2.5 (Table 13 of Chinese Specification GBJ 3-73).
(4) This Table is reproduced from Table 5 of Chinese Specification GBJ 3-73.

Table E5 - Permissible Shear Strength of Masonry (Failure along Joints)

Nature of Force	Failure Mode	Type of Masonry	MORTAR Compressive Strength (MPa)				
			10	5	2.5	1	0.4
Shear	 Shear thro' plane	Bricks	0.4	0.3	0.2	0.12	0.06
	 Shear along saw-tooth path	Bricks	0.4	0.3	0.2	0.12	0.06
	 Shear along an irregular path	Random Rubble	0.6	0.45	0.3	0.18	0.09

Notes :
(1) Table not applicable to squared rubble and ashlar walls.
(2) If pure cement/sand mortar is used, apply a modification factor of 0.75.
(3) For permissible strength, apply safety factor of 2.5 and 3.3 for brickwork and random rubble walls respectively.
(4) This Table is reproduced from Table 5 of Chinese Specification GBJ 3-73.

Table E6 - Requirements for Mortar (BS 5628 : Part 1 : 1978)

		Mortar designation	Type of mortar (proportion by volume)			Mean compressive strength at 28 days	
			Cement : lime : sand	Masonry cement : sand	Cement : sand with plasticizer	Preliminary (laboratory) tests	Site tests
Increasing strength	Increasing ability to accommodate movement, e.g. due to settlement, temperature and moisture changes					N/mm^2	N/mm^2
		(i)	1 : 0 to ¼ : 3	–	–	16.0	11.0
		(ii)	1 : ½ : 4 to 4½	1 : 2½ to 3½	1 : 3 to 4	6.5	4.5
		(iii)	1 : 1 : 5 to 6	1 : 4 to 5	1 : 5 to 6	3.6	2.5
		(iv)	1 : 2 : 8 to 9	1 : 5½ to 6½	1 : 7 to 8	1.5	1.0
Direction of change in properties is shown by the arrows			Increasing resistance to frost attack during construction Improvement in bond and consequent resistance to rain penetration				

Table E7 - Characteristic Compressive Strength of Masonry (BS 5628 : Part 1 : 1978)

(a) Constructed with standard format bricks

Mortar designation	Compressive strength of unit (N/mm²)								
	5	10	15	20	27.5	35	50	70	100
(i)	2.5	4.4	6.0	7.4	9.2	11.4	15.0	19.2	24.0
(ii)	2.5	4.2	5.3	6.4	7.9	9.4	12.2	15.1	18.2
(iii)	2.5	4.1	5.0	5.8	7.1	8.5	10.6	13.1	15.5
(iv)	2.2	3.5	4.4	5.2	6.2	7.3	9.0	10.8	12.7

(b) Constructed with blocks having a ratio of height to least horizontal dimension of 0.6

Mortar designation	Compressive strength of unit (N/mm²)							
	2.8	3.5	5.0	7.0	10	15	20	35 or greater
(i)	1.4	1.7	2.5	3.4	4.4	6.0	7.4	11.4
(ii)	1.4	1.7	2.5	3.2	4.2	5.3	6.4	9.4
(iii)	1.4	1.7	2.5	3.2	4.1	5.0	5.8	8.5
(iv)	1.4	1.7	2.2	2.8	3.5	4.4	5.2	7.3

(d) Constructed from solid concrete blocks having a ratio of height to least horizontal dimension of between 2.0 and 4.0

Mortar designation	Compressive strength of unit (N/mm²)							
	2.8	3.5	5.0	7.0	10	15	20	35 or greater
(i)	2.8	3.5	5.0	6.8	8.8	12.0	14.8	22.8
(ii)	2.8	3.5	5.0	6.4	8.4	10.6	12.8	18.8
(iii)	2.8	3.5	5.0	6.4	8.2	10.0	11.6	17.0
(iv)	2.8	3.5	4.4	5.6	7.0	8.8	10.4	14.6

Table E8 - Characteristic Flexural Strength of Masonry (BS 5628 : Part 1 : 1978)

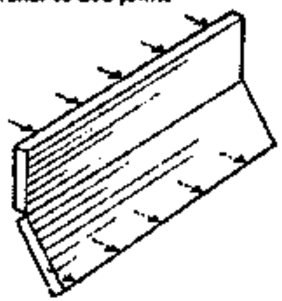
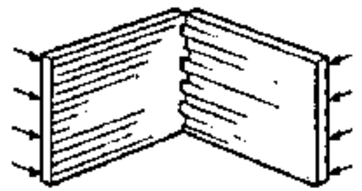
	Plane of failure parallel to bed joints 			Plane of failure perpendicular to bed joints 		
Mortar designation	(i)	(ii) and (iii)	(iv)	(i)	(ii) and (iii)	(iv)
Clay bricks having a water absorption						
less than 7 %	0.7	0.5	0.4	2.0	1.5	1.2
between 7 % and 12 %	0.5	0.4	0.35	1.5	1.1	1.0
over 12 %	0.4	0.3	0.25	1.1	0.9	0.8
Calcium silicate bricks	0.3		0.2	0.9		0.6
Concrete bricks	0.3		*	0.9		*
Concrete blocks of compressive strength in N/mm^2: 2.8				0.4		0.4
3.5				0.45		0.4
7.0				0.60		0.5
10.5	0.25		0.20	0.75		0.6
14.0 and over				0.90†		0.7†

Table E9 - Allowable Compressive Stresses for Unreinforced Stone Masonry (MPa)

Material	National Board Fire Underwriters				San Franciso		New York City	
	Mortar Type				Mortar Type		Cement	Cement
	A	B	C	D	E	F	Lime Mortar	Mortar
Granite, ashlar	5.6	4.5	3.5	2.8			4.5*	5.6*
Limestone, ashlar	3.5	2.8	2.3	1.7	0.9	0.9	2.8*	3.5*
Marble, ashlar	3.5	2.8	2.3	1.7			2.8*	3.5*
Sandstone, ashlar	2.8	2.2	1.7	1.1			1.7*	2.1*
Gneiss							4.2*	5.2*
Bluestone							2.1*	2.8*
Rubble Stone	1.0	0.7	0.6	-	-	-	0.8	1.0
Cast Stone	2.8	2.2	1.7	1.1	2.8	2.4	-	-

Note : *Specified for dressed or cut beds.

Mortar Type	Strength (MPa)	Portland Cement	Lime	Aggregate	
A	17.5	1	0 to ¼	Not over 3 parts)	
B	4.2 - 17.5	1	1 to 1¼	Not over 6 parts)	Proportions by Volume
C	1.4 - 4.2	1	2 to 2½	Not over 9 parts)	
D	0.5 - 1.4	0 to ½	1 to 1¼	Not over 3 parts)	
E	17.5	1	1¼	3	
F	12.5	1	½	4½	

Table E10 - Allowable Compressive Stresses for Unreinforced Masonry of Artificial Blocks (MPa)

	Mortar Type			
	A	B	C	D
Brick, average compressive stress :				
55.8 +	2.8	2.1	1.4	0.7
31.4 - 55.8	1.7	1.4	1.0	0.7
17.5 - 31.4	1.2	1.0	0.8	0.5
10.5 - 17.5	0.9	0.7	0.5	0.3
Cavity and hollow walls :				
Solid unit	0.9	0.7		
Hollow unit	0.4	0.3		
Solid concrete units, compressive stress :				
8.4 - 10.4	0.9	0.7	0.4	
10.4 +	1.2	0.9	0.6	
Hollow masonry units	0.6	0.5		

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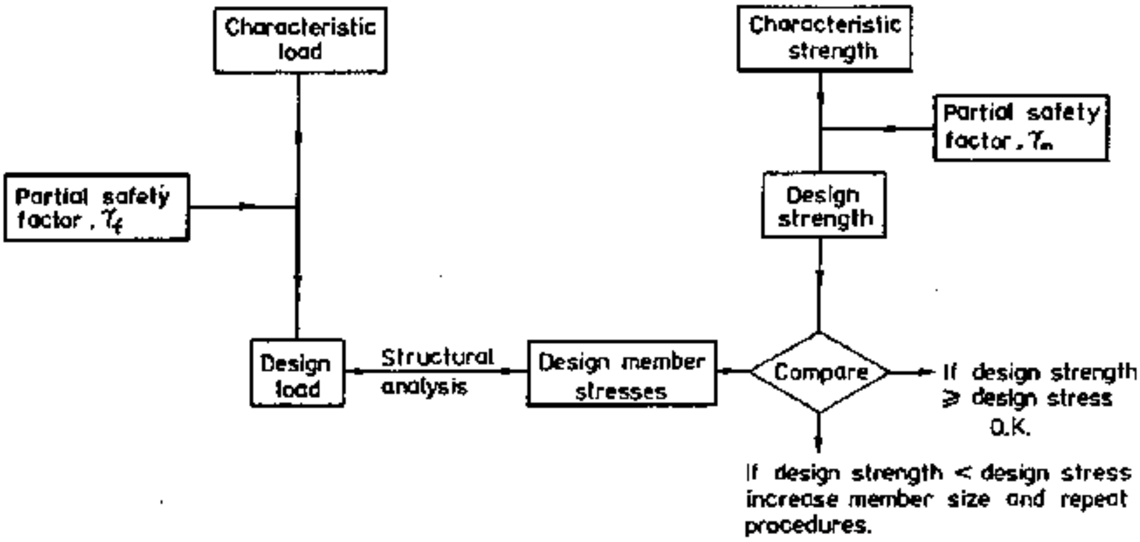


Figure E1 - Procedures of Limiting State Design

APPENDIX F

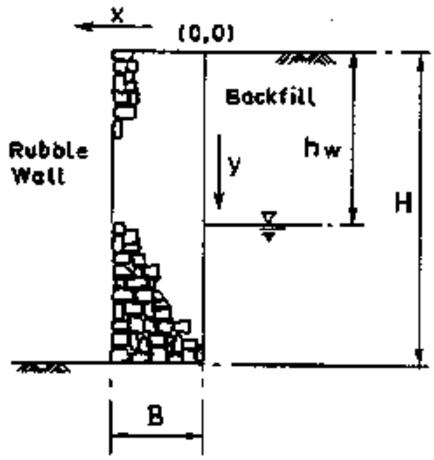
ANALYTICAL SOLUTIONS ON THE DISTRIBUTION OF STRESSES IN A GRAVITY RETAINING WALL

F.1 Note

A minor mistake involving the $\gamma_w(y - h_w)$ term has been found in the equation on σ_y. Its effect on the distribution of stresses presented in Figure 6.5 to 6.10 has been examined briefly and was found to be insignificant.

Stress Analysis in Masonry Walls

F.2 Definition of Terms : -



- K_a = coefficient of active earth pressure based on Coulomb's method.
- δ = angle of friction between backfill soil & back of wall
- ϕ_m = angle of internal friction of rubble wall
- C_m = cohesion of rubble wall
- γ_m = unit weight of rubble wall
- ϕ = angle of internal friction of backfill
- γ = saturated unit weight of backfill
- γ' = buoyant unit weight of backfill
- γ_w = unit weight of water
- H = height of wall
- B = width of wall
- h_w = depth to ground water table as measured from top of wall
- M = net moment about midpoint on wall base
- M' = rate of change of M with respect to depth, y
- M" = rate of change of M' with respect to depth, y
- σ_x = horizontal internal normal stress
- σ_y = vertical internal normal stress
- τ = horizontal/vertical internal shear stress
- σ_1 = major principal stress
- σ_3 = minor principal stress
- τ_{max} = maximum shear stress
- α = angle between principal stress plane & horizontal
- S_c = shear strength of rubble wall

F.3 Equations Describing Stresses in a Gravity Retaining Wall

F.3.1 The Equations

$$\sigma_x = K_a\gamma'\left(\frac{x^2\tan\delta}{2B} - x\tan\delta\right) + \frac{x^2M''}{B^2}\left(\frac{2x}{B} - 3\right) + [K_a\gamma h_w + K_a\gamma'(y - h_w) + \gamma_w(y - h_w)]$$

$$\sigma_y = \frac{1}{B}\{\gamma_m By + K_a[\frac{\gamma h_w^2}{2} + \gamma h_w(y - h_w) + \frac{\gamma'(y - h_w)^2}{2}]\tan\delta\} - \frac{6M}{B^2} - \gamma_w(y - h_w) + \frac{x}{B}\left(\frac{12M}{B^2}\right)$$

$$\tau = K_a[\gamma h_w + \gamma'(y - h_w)](1 - \frac{x}{B})\tan\delta + \frac{6xM'}{B^2}(1 - \frac{x}{B})$$

$$(F.O.S.)_x = \frac{C_m + \sigma_y\tan\phi_m}{\tau}$$ = Factor of safety against horizontal internal slip

$$(F.O.S.)_y = \frac{C_m + \sigma_x\tan\phi_m}{\tau}$$ = Factor of safety against vertical internal slip

Note that for $y < h_w$, terms with γ_w and $(y\text{-}h_w)$ vanish and h_w is replaced by y.

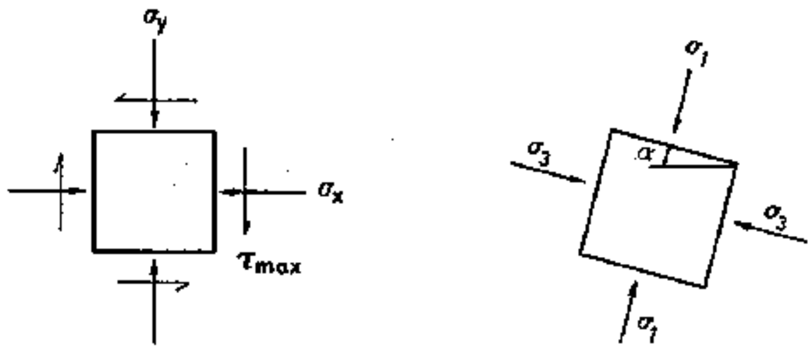
$$\sigma_{1,3} = \frac{\sigma_x + \sigma_y}{2} \pm \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau^2}$$

$$\tau_{max} = \frac{(\sigma_1 - \sigma_3)}{2}$$

$$\alpha = \frac{1}{2}\tan^{-1}\left(\frac{2\tau}{\sigma_y - \sigma_x}\right)$$

$$(F.O.S.)_{sliding} = \frac{2S_c}{\sigma_1 - \sigma_3}$$ = Factor of safety against sliding in the direction of maximum shear stress

F.3.2 Sign Conventions :-



Positive Convention

F.3.3 Assumptions & Limitation :-

(1) Coulomb's state of earth pressure

(2) Cohesionless backfill

(3) Upthrust due to water at base of wall linearly distributed with maximum at heel and zero at toe.

(4) Upthrust has no effect on τ

(5) Rectangular wall section with a height/base-width ratio of 3

(6) Level backfill

(7) Boundary conditions :-

$$\text{At } x = 0 \quad , \quad \tau = [K_a\gamma h_w + K_a\gamma'(y-h_w)]\tan\delta$$
$$\sigma_x = K_a\gamma h_w + K_a\gamma'(y-h_w) + \gamma_w(y-h_w)$$

$$\text{At } x = B \quad , \quad \tau = 0$$
$$\sigma_x = 0$$

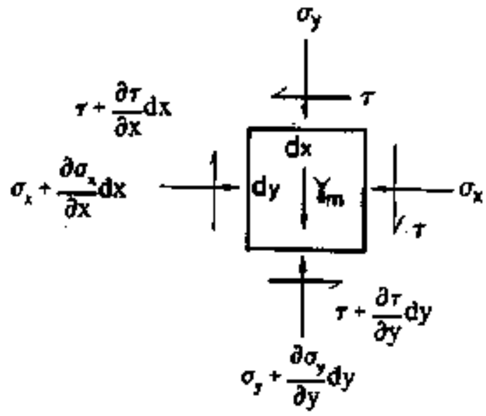
(8) All other assumptions pertaining to the Coulomb's state of earth pressure

(9) All other assumptions pertaining to the beam theory

F.4 Derivation of Stress Equations :-

F.4.1 2-Dimensional Equations of Equilibrium :-

Assume element has unit thickness



Moment equilibrium yields

$$\tau_{xy} = \tau_{yx}$$

$\Sigma F_X = 0$ then

$$(\sigma_x + \frac{\partial \sigma_x}{\partial x}dx)dy + (\tau + \frac{\partial \tau}{\partial y}dy)dx - \tau dx - \sigma_x dy = 0$$

or

$$\frac{\partial \sigma_x}{\partial x}dxdy + \frac{\partial \tau}{\partial y}dydx = 0$$

or

$$\frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau}{\partial y} = 0$$

$$\therefore \quad \sigma_x = -\int \frac{\partial \tau}{\partial y}dx \text{ ——— } ①$$

$\Sigma F_y = 0$ then

$$(\sigma_y + \frac{\partial \sigma_y}{\partial y}dy)dx + (\tau + \frac{\partial \tau}{\partial x}dx)dy - \sigma_y dx - \tau dy - \gamma_m dxdy = 0$$

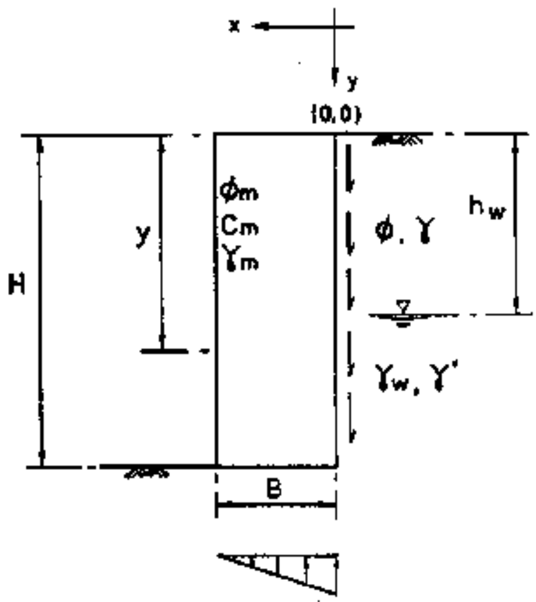
or

$$\frac{\partial \sigma_y}{\partial y} + \frac{\partial \tau}{\partial x} - \gamma_m = 0$$

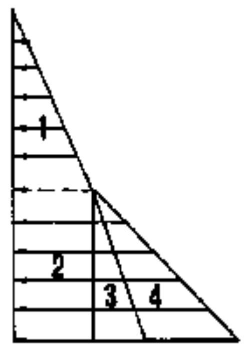
$$\therefore \quad \tau = \int (\gamma_m - \frac{\partial \sigma_y}{\partial y})dx \text{ ——— } ②$$

F.4.2 External Forces Acting on the Wall :-

Lateral forces acting on the wall.



Upthrust



Central Pressure Distribution

(1) Due to dry backfill

$$Fx_1 = 1/2K_a\gamma h_w^2$$

(2) Due to dry backfill & is uniform from $y = h_w$ to $y = H$:-

$$Fx_2 = K_a\gamma h_w(y - h_w)$$

(3) Due to submerged backfill

$$Fx_3 = 1/2K_a\gamma'(y - h_w)^2$$

(4) Due to water

$$Fx_4 = 1/2\gamma_w(y - h_w)^2$$

F.4.3 Forces and Moments Acting on the Wall :-

Force	Lateral Force Fx	Vertical Force Fy	Moment Arm	Moment about Midpoint on Wall Base	
				Overturning	Restoring
(1)	$1/2K_a\gamma h_w^2$	-	$y-2/3h_w$	$1/2K_a\gamma h_w^2(y-2/3h_w)$	-
(2)	$K_a\gamma h_w(y-h_w)$	-	$(y-h_w)/2$	$K_a\gamma h_w(y-h_w)^2/2$	-
(3)	$1/2K_a\gamma'(y-h_w)^2$	-	$(y-h_w)/3$	$1/6K_a\gamma'(y-h_w)^3$	-
(4)	$1/2\gamma_w(y-h_w)^2$	-	$(y-h_w)/3$	$1/6\gamma_w(y-h_w)^3$	-
(1)	-	$\gamma_m By$	0	-	-
(2)	-	$1/2K_a\gamma h_w^2\tan\delta$	B/2	-	$1/4K_a\gamma h_w^2 B\tan\delta$
(3)	-	$K_a\gamma h_w(y-h_w)\tan\delta$	B/2	-	$1/2K_a\gamma h_w(y-h_w)B\tan\delta$
(4)	-	$1/2K_a\gamma'(y-h_w)^2\tan\delta$	B/2	-	$1/4K_a\gamma'(y-h_w)^2 B\tan\delta$
Upthrust	-	$-\gamma_w(y-h_w)B/2$	B/6	$\gamma_w(y-h_w)B^2/12$	-

ΣM_o (Overturning moment about midpoint on wall base)

$$= 1/2K_a\gamma h_w^2(y-2/3h_w) + K_a\gamma h_w(y-h_w)^2/2 + 1/6K_a\gamma'(y-h_w)^3 + 1/6\gamma_w(y-h_w)^3 + \gamma_w(y-h_w)B^2/12$$

ΣM_r (Restoring moment about midpoint on wall base)

$$= 1/4K_a\gamma h_w^2 B\tan\delta + 1/2K_a\gamma h_w(y-h_w)B\tan\delta + 1/4K_a\gamma'(y-h_w)^2 B\tan\delta$$

$\therefore$ Net moment about midpoint on wall base, M

$$\begin{aligned} = & -\frac{K_a B}{2}\left[\frac{\gamma h_w^2}{2} + \gamma h_w(y-h_w) + \gamma'(y-h_w)^2/2\right]\tan\delta \\ & + K_a\left[\frac{\gamma h_w^2}{2}\left(y-\frac{2}{3}h_w\right) + \gamma h_w(y-h_w)^2/2 + \gamma'(y-h_w)^3/6\right] \\ & + \gamma_w(y-h_w)^3/6 + \gamma_w(y-h_w)B^2/12 \qquad \text{③} \end{aligned}$$

F.4.4 Normal Stress, σ_y :-

σ_y toe / heel due to flexure $= \pm\dfrac{6M}{B^2}$

σ_y toe / heel due to self wt. + friction $= \frac{1}{B}[\gamma_m By + K_a(\frac{\gamma h_w^2}{2} + \gamma h_w(y-h_w) + \frac{\gamma'(y-h_w)^2}{2})\tan\delta] - \frac{\gamma_w(y-h_w)}{2}$

Superimposing :-

$$\therefore\ \sigma_y\text{toe} = 1/B\{\gamma_m By + K_a[\gamma h_w^2/2 + \gamma h_w(y-h_w) + \gamma'(y-h_w)^2/2]\tan\delta\} + 6M/B^2 - \gamma_w(y-h_w)/2$$

$$\sigma_y\text{heel} = 1/B\{\gamma_m By + K_a[\gamma h_w^2/2 + \gamma_w(y-h_w) + \gamma'(y-h_w)^2/2]\tan\delta\} - 6M/B^2 - \gamma_w(y-h_w)/2$$

$\therefore$ Rate of change of σ_y w.r.t. x

$$= [12M/B^2]1/B$$

$\therefore$ At any point x from heel,

$$\sigma_y = 1/B\{\gamma_m By + K_a[\gamma h_w^2/2 + \gamma h_w(y-h_w) + \gamma'(y-h_w)^2/2]\tan\delta\} - 6M/B^2 - \frac{\gamma_w(y-h_w)}{2} + (\frac{12M}{B^2})\frac{x}{B} \qquad \text{For } y \geq h_w \quad \text{——④}$$

Note that for $y < h_w$, terms with γ_w and $(y\text{-}h_w)$ vanish and h_w is replaced by y.

F.4.5 <u>Shear Stress, τ</u> :-

From equation ③

$$M' = \frac{-K_a B}{2}[\gamma h_w + \gamma'(y-h_w)]\tan\delta + K_a[\frac{\gamma h_w^2}{2} + \gamma h_w(y-h_w) + \frac{\gamma'(y-h_w)^2}{2}] + \frac{\gamma_w(y-h_w)^2}{2} \qquad \text{——⑤}$$

Direct stress due to upthrust does not contribute to shear stress because it is a body force. The following equation was used to calculate τ :

$$\sigma_y = 1/B\{\gamma_m By + K_a[\gamma h_w^2/2 + \gamma h_w(y-h_w) + \gamma'(y-h_w)^2/2]\tan\delta\} - 6M/B^2 + 12Mx/B^3$$

$$\therefore \quad \frac{\partial \sigma_y}{\partial y} = \frac{1}{B}\{\gamma_m B + K_a[\gamma h_w + \gamma'(y - h_w)]\tan\delta\} - \frac{6M'}{B^2} + \frac{12xM'}{B^3} \quad \text{———⑥}$$

From equation ②

$$\frac{\partial \sigma_y}{\partial y} + \frac{\partial \tau}{\partial x} - \gamma_m = 0$$

$$\therefore \quad \frac{\partial \tau}{\partial x} = \gamma_m - \frac{\partial \sigma_y}{\partial y}$$

$$= \gamma_m - \gamma_m - \frac{K_a}{B}[\gamma h_w + \gamma'(y - h_w)]\tan\delta + \frac{6M'}{B^2} - \frac{12xM'}{B^3}$$

Integration yields :-

$$\tau = \frac{-K_a x}{B}[\gamma h_w + \gamma'(y - h_w)]\tan\delta + \frac{6M'x}{B^2} - \frac{6x^2M'}{B^3} + C_1 \quad \text{———⑦}$$

Boundary Conditions :-

(i) $\tau(x = 0) = [K_a\gamma h_w + K_a\gamma'(y-h_w)]\tan\delta$
(ii) $\tau(x = B) = 0$

Substitute for boundary condition (i) into equation ⑦

$$[K_a\gamma h_w + K_a\gamma'(y - h_w)]\tan\delta = 0 + 0 - 0 + C_1$$

$$\therefore \quad C_1 = K_a[\gamma h_w + \gamma'(y - h_w)]\tan\delta$$

Substitute for C_1 into equation ⑦

$$\therefore \quad \tau = \frac{-K_a x[\gamma h_w + \gamma'(y - h_w)]\tan\delta}{B} + \frac{6M'x}{B^2} - \frac{6x^2M'}{B^3}$$
$$+K_a[\gamma h_w + \gamma'(y - h_w)]\tan\delta$$

which also satisfies $\tau(B) = 0$

Simplifying

$$\tau = K_a[\gamma h_w + \gamma'(y - h_w)](1 - \frac{x}{B})\tan\delta + \frac{6xM'}{B^2}(1 - \frac{x}{B}) \quad \text{———⑧}$$

F.4.6 Normal Stress, σ_x -

From equation ⑤

$$M'' = \frac{-K_aB}{2}\gamma' \tan\delta + K_a[\gamma h_w + \gamma'(y - h_w)] + \gamma_w(y - h_w)$$

From equation ⑧

$$\frac{\partial \tau}{\partial y} = K_a\gamma'(1 - \frac{x}{B})\tan\delta + \frac{6xM''}{B^2}(1 - \frac{x}{B})$$

From equation ①

$$\frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau}{\partial y} = 0$$

$$\therefore \frac{\partial \sigma_x}{\partial x} = -K_a\gamma'(1 - \frac{x}{B})\tan\delta - \frac{6xM''}{B^2}(1 - \frac{x}{B})$$

Integrating yields

$$\sigma_x = -K_a\gamma'(x - \frac{x^2}{2B})\tan\delta - \frac{3M''x^2}{B^2} + \frac{2M''x^3}{B^3} + C_2 \qquad \text{———⑨}$$

Boundary Conditions :-

(i) $\sigma_x(x = 0) = K_a\gamma h_w + K_a\gamma'(y - h_w) + \gamma_w(y - h_w)$
(ii) $\sigma_x(x = B) = 0$

Substitute boundary condition (i) into equation ⑨

$$\therefore K_a\gamma h_w + K_a\gamma'(y - h_w) + \gamma_w(y - h_w) = 0 - 0 + 0 + C_2$$

Substitute for C_2 into equation ⑨

$$\therefore \sigma_x = -K_a\gamma'(x - \frac{x^2}{2B})\tan\delta - \frac{3M''x^2}{B^2} + \frac{2M''x^3}{B^3} + K_a\gamma h_w + K_a\gamma'(y - h_w) + \gamma_w(y - h_w)$$

LIST OF PLATES

Negative No. R/W 81.10.17

Plate 3.1 -
Masonry Retaining Wall at
Tung Lung Fort. Collapsed section
exposing the structure of the wall.

Plate 3.2 - 400 Year Old 'Box-bonded' Masonry Wall in HUASHAN, China

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Plate 3.3 - Tied Face Retaining Wall at Caine Road J/o Castle Road

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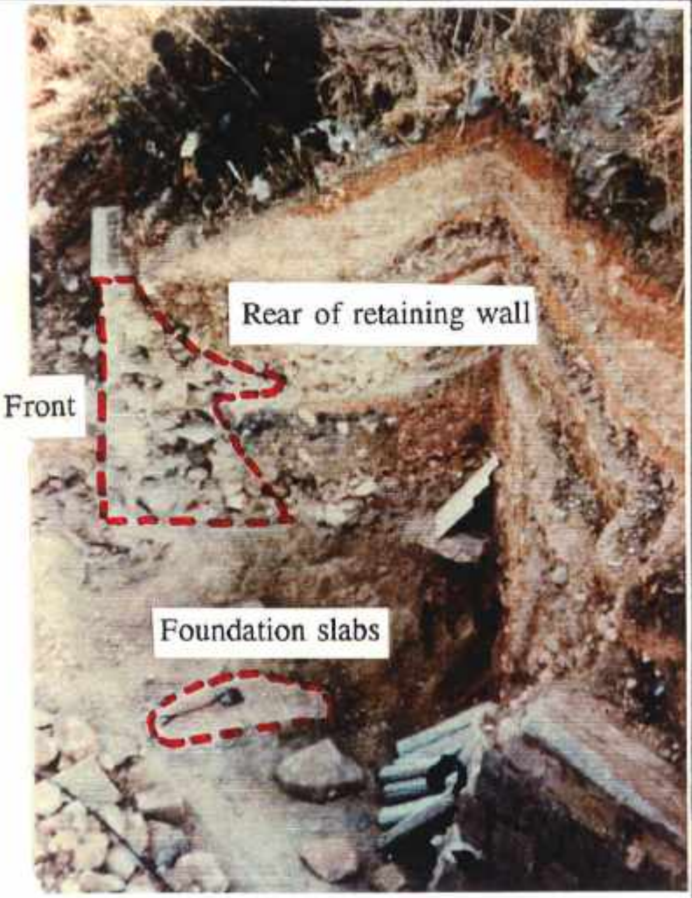
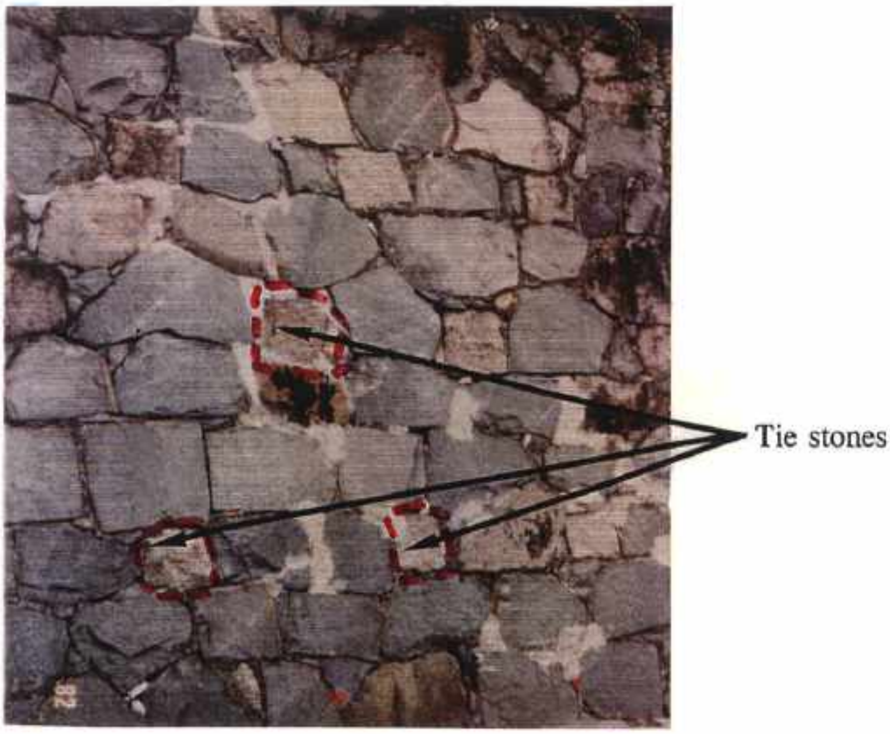
Plate 3.4 -
Tied Face Wall Used as
Foundation Wall

Negative No. 201514

Plate 3.5 -
Stone Rubble Retaining Wall
without Rear Blocks





Negative No. 201807

Plate 3.6 - Tied Stone Rubble Retaining Walls

Negative No. R/W 81.06.8A

Plate 3.7 - Recent Masonry Retaining Walls

Negative No. 201805

Plate 3.8 - Rough-picked Polygonal Wall

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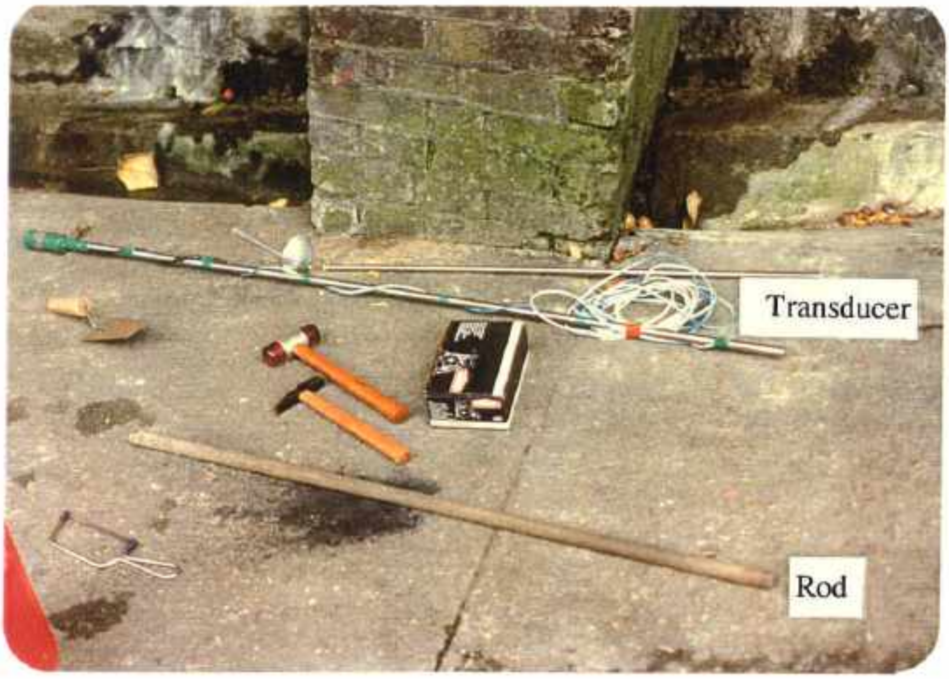
Plate 4.1 - Typical Example of Stone Rubble Retaining Walls Reaching the Limiting Point of Stability, Yorkshire Region (after Jones 1979)

Negative No. R/W 81.19.18

Plate 4.2 - Stone Rubble Retaining Wall Failures in Yorkshire Region, U.K. (after Jones 1979)



Negative No. R/W 81.13.2

Plate 7.1 - Seismic Probing Equipment - the Inducers

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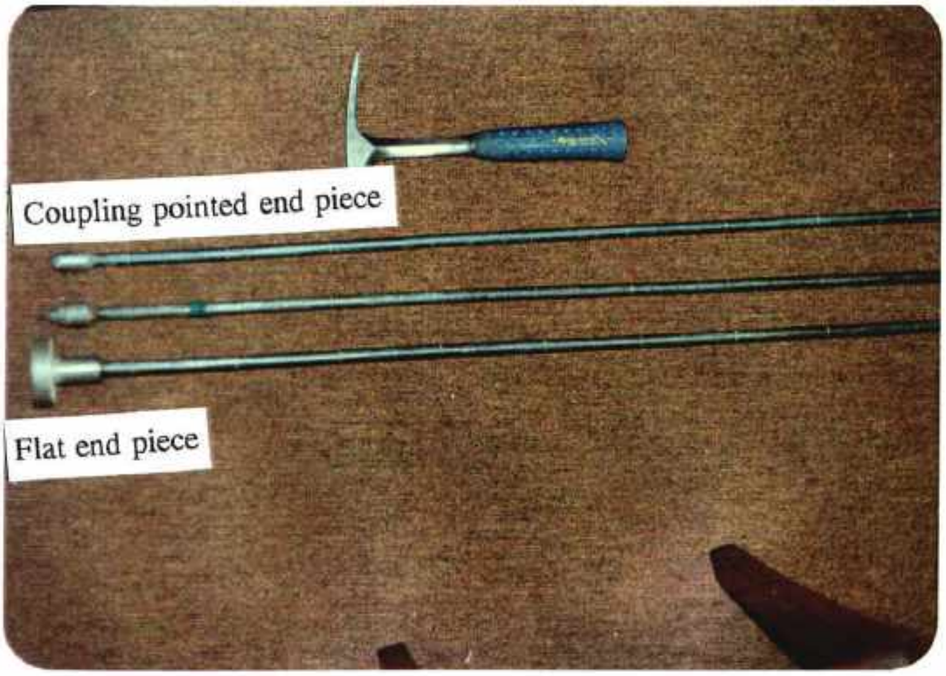
Plate 7.2 - Seismic Probing Equipment - the Timer



Negative No. R/W 81.23.10

Plate 7.3 - Weephole Probe - Rods and End Pieces

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Plate 7.4 - Electric Coring Equipment (I)

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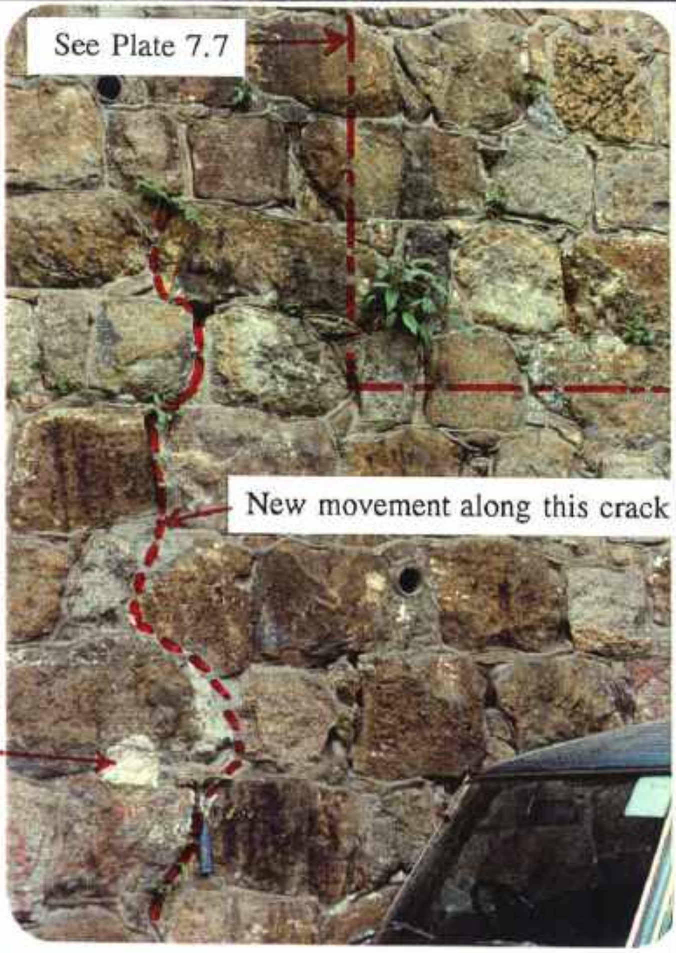
Plate 7.5 - Electric Coring Equipment (II)

Negative No. R/W 81.23.3

Plate 7.6 -
Disturbance to Old Masonry Retaining Walls due to Drilling - Reactivation of Cracks



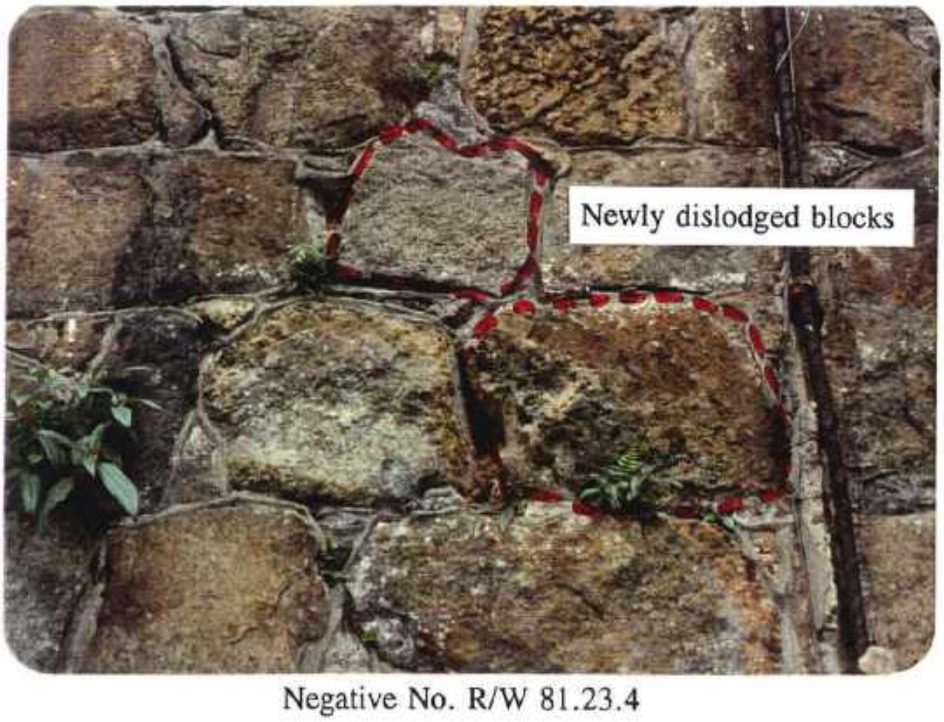


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Plate 7.7 - Disturbance to Old Masonry Retaining Walls due to Drilling - Dislodging of Stone Blocks

Negative No. R/W 81.07.05

Plate 7.8 - Example of Crack Origin Diagnosis - Retaining Wall at Hok Sze Terrace (I)

Negative No. R/W 81.07.01

Plate 7.9 -
Example of Crack Origin Diagnosis -
Retaining Wall
at Hok Sze Terrace (II)

Negative No. R/W 81.21.22

Plate 8.1 - Chinese Banyan (Ficus Microcarpa) - Characteristic Aerial Roots

Negative No. R/W 81.21.17

Plate 8.2 - Chinese Banyan - Surface Network of Ramifying Roots

Negative No. R/W 81.21.19

Plate 8.3 - Displacement of Stone Blocks due to Growth of Trees on Masonry Walls (I)

Negative No. R/W 81.21.20

Plate 8.4 - Displacement of Stone Blocks due to Growth of Trees on Masonry Wall (II)