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)

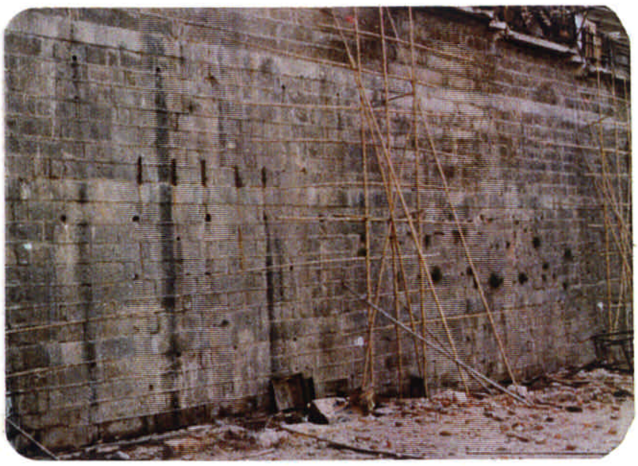


Plate B10 - Tied Face Wall with
Horizontal Beams
(11SW-A/R45)

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.

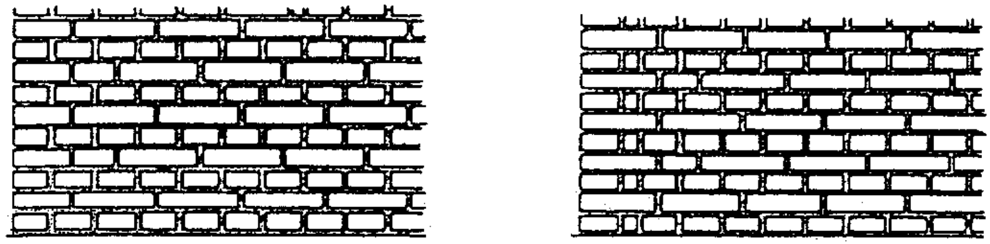
POINTING - The external finish to beds and joints. It can either be put in as part of the mortar or else the mortar may be raked out for approximately 40 mm deep before the final set and be replaced by better quality cement/sand mixes. For dry-packed masonry walls, pointing may also be applied to the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

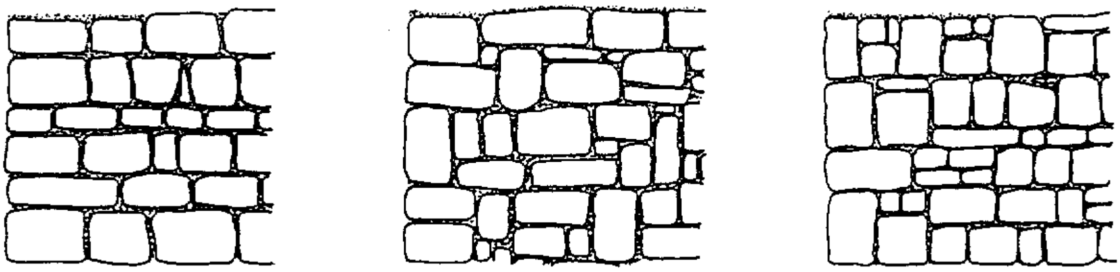
LIST OF FIGURES



(a) Dutch Bond

(b) English Bond

Figure C1 - Bond Patterns for Walls of Standard Format Bricks

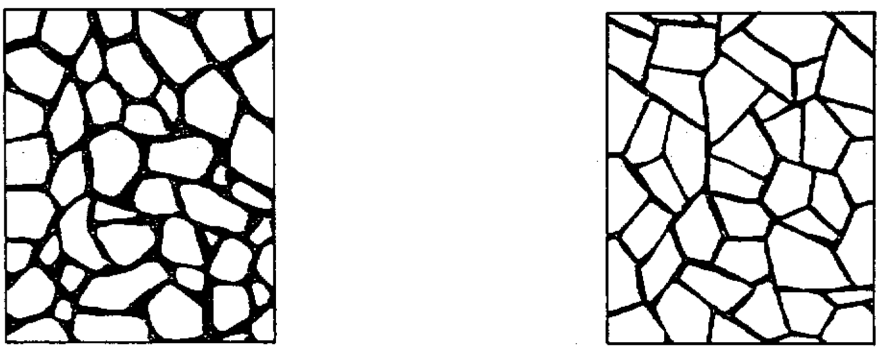


(a) Coursed Stone Rubble Wall

(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

Location : Failure of Retaining Wall at the Jewish Recreation Club, Robinson Road

Date : 3.8.1979

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

Table D1 - Summary of Observations on the Crack at the Corner of the Retaining Wall at St. Joseph Terrace, Caine Road

Date/Time(1)	Observations	Remarks
- 6 yr		Wall constructed on top of an old dry packed rubble wall.
- 2 yr	A small crack appeared at the corner.	The crack was infilled with cement.
- 2 months	The crack reappeared.	
- 5¼ hr	Crack noticeably widened.	
- 4½ hr	Crack widened to 2 inches.	
- 1½ hr	Crack widened to 6 inches or more	
0 hr	Wall collapsed.	

Note : (1) The time of the failure (11:00 a.m., 16.7.1917) is adopted as the datum time. All quoted values are approximations.

LIST OF FIGURES

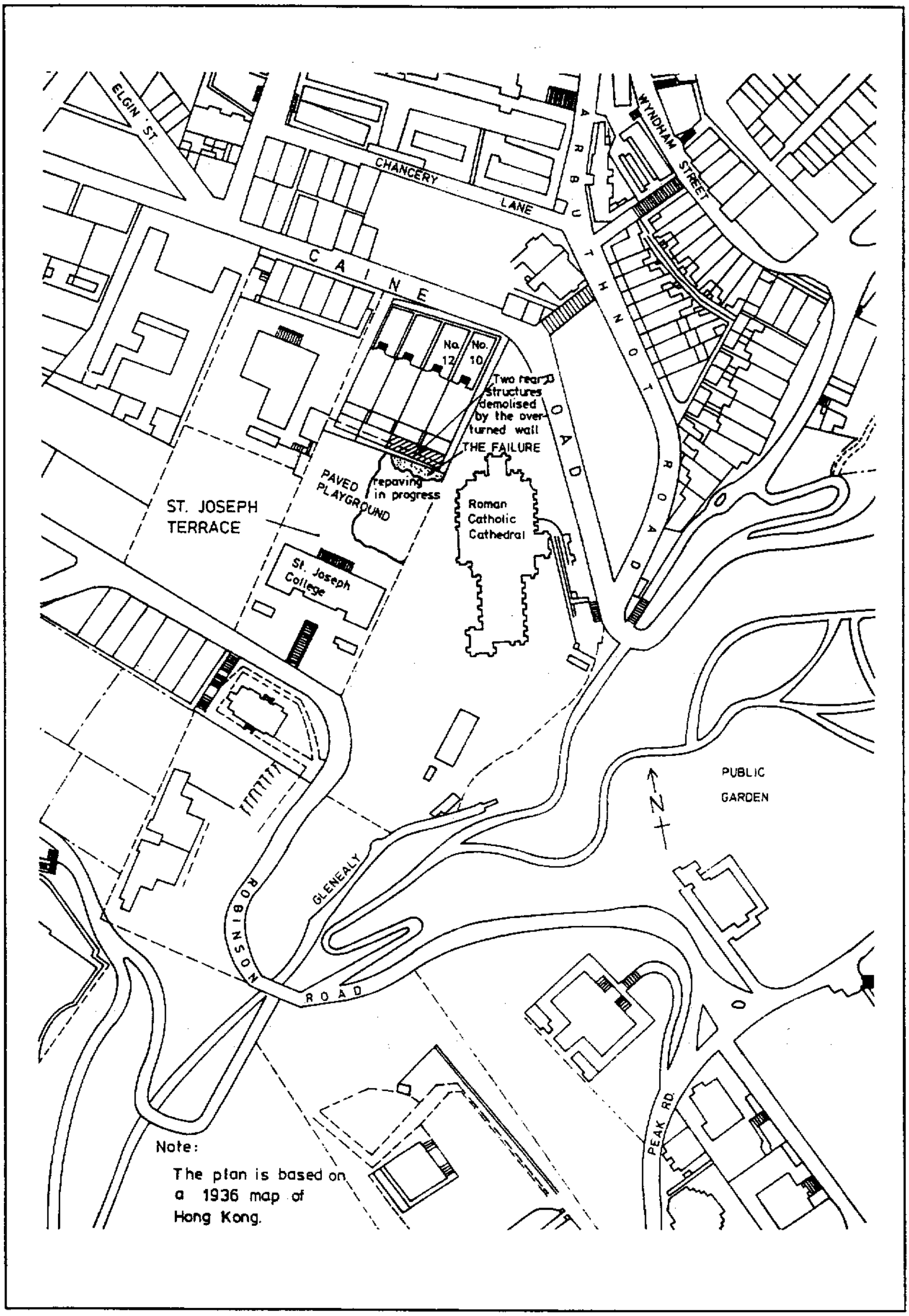


Figure D1.1 - Retaining Wall at St. Joseph Terrace - Site Plan

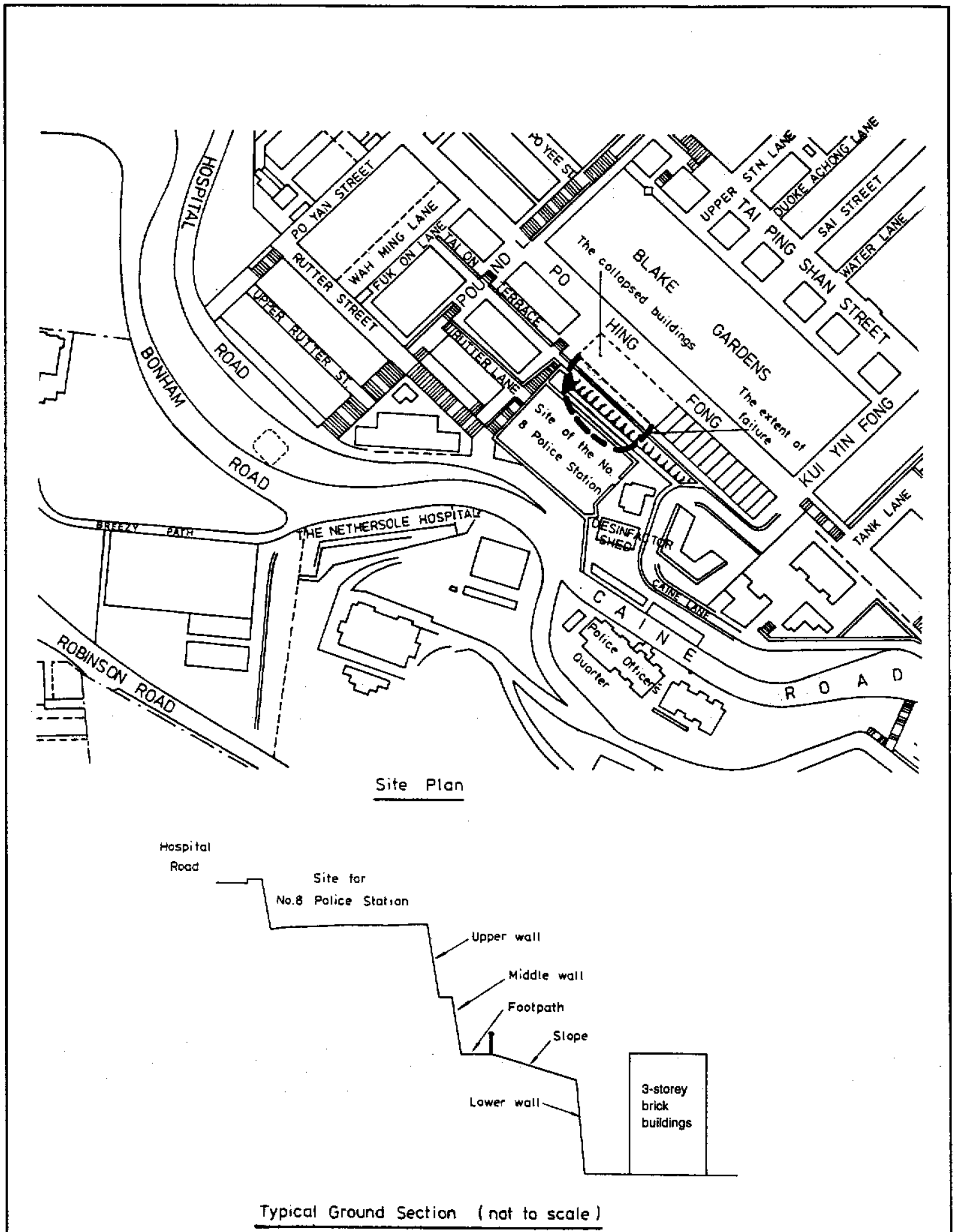


Figure D2.1 - Retaining Wall at Po Hing Terrace - Site Plan and Typical Sections

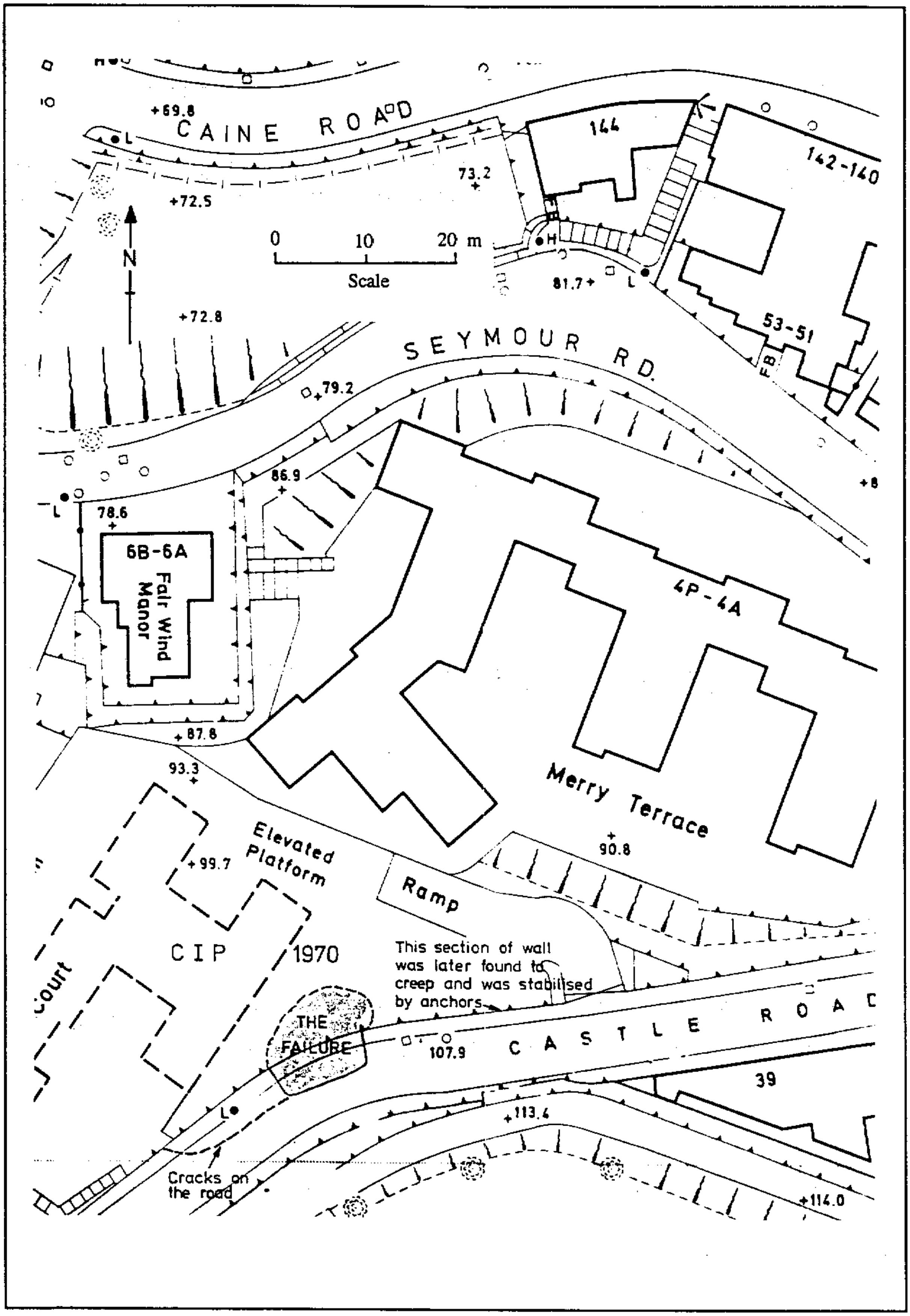


Figure D3.1 - Retaining Wall at 10, Castle Road - Site Plan

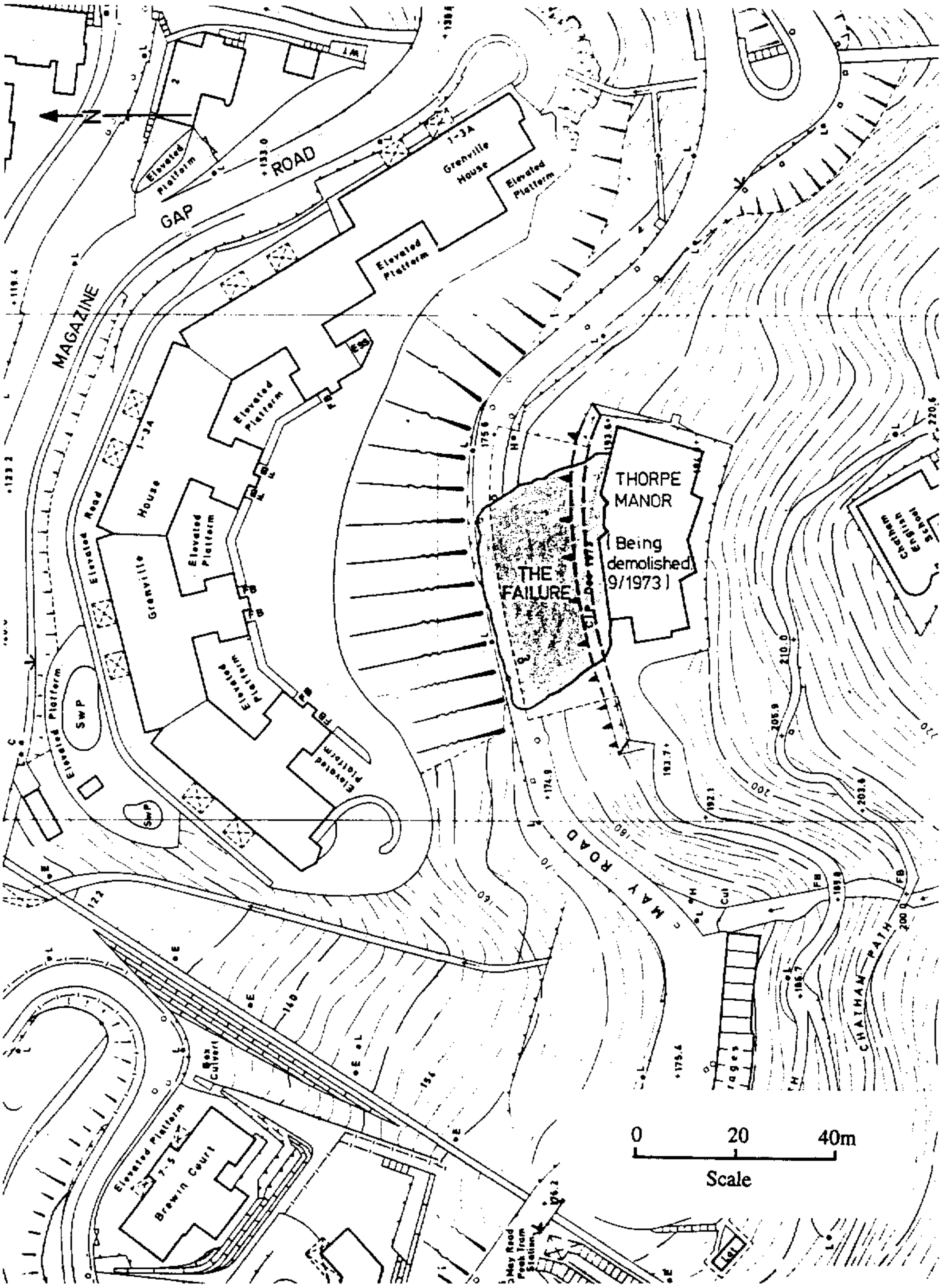


Figure D4.1 - Retaining Wall at 1, May Road - Site Plan

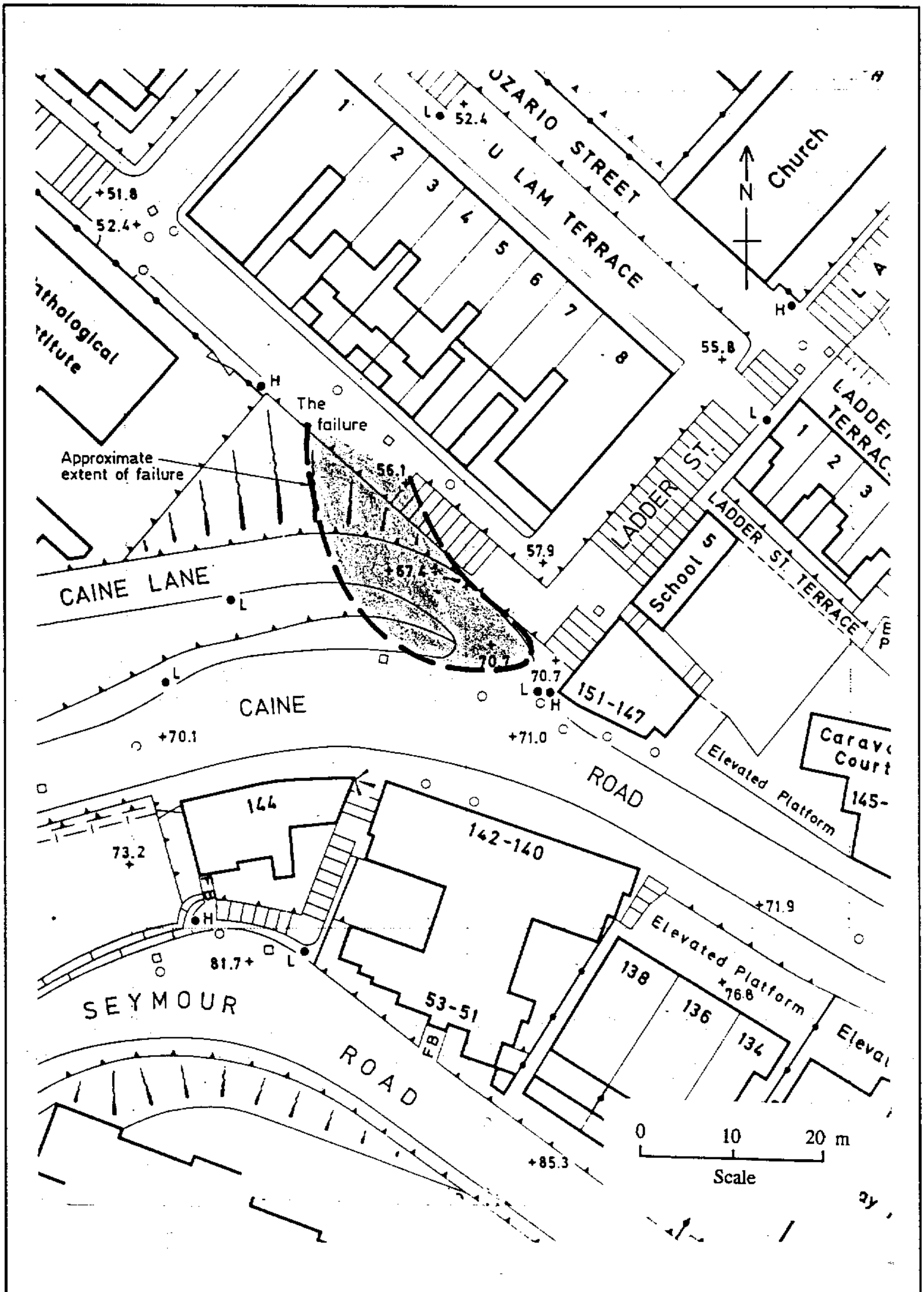


Figure D5.1 - Retaining Wall at Caine Lane - Site Plan

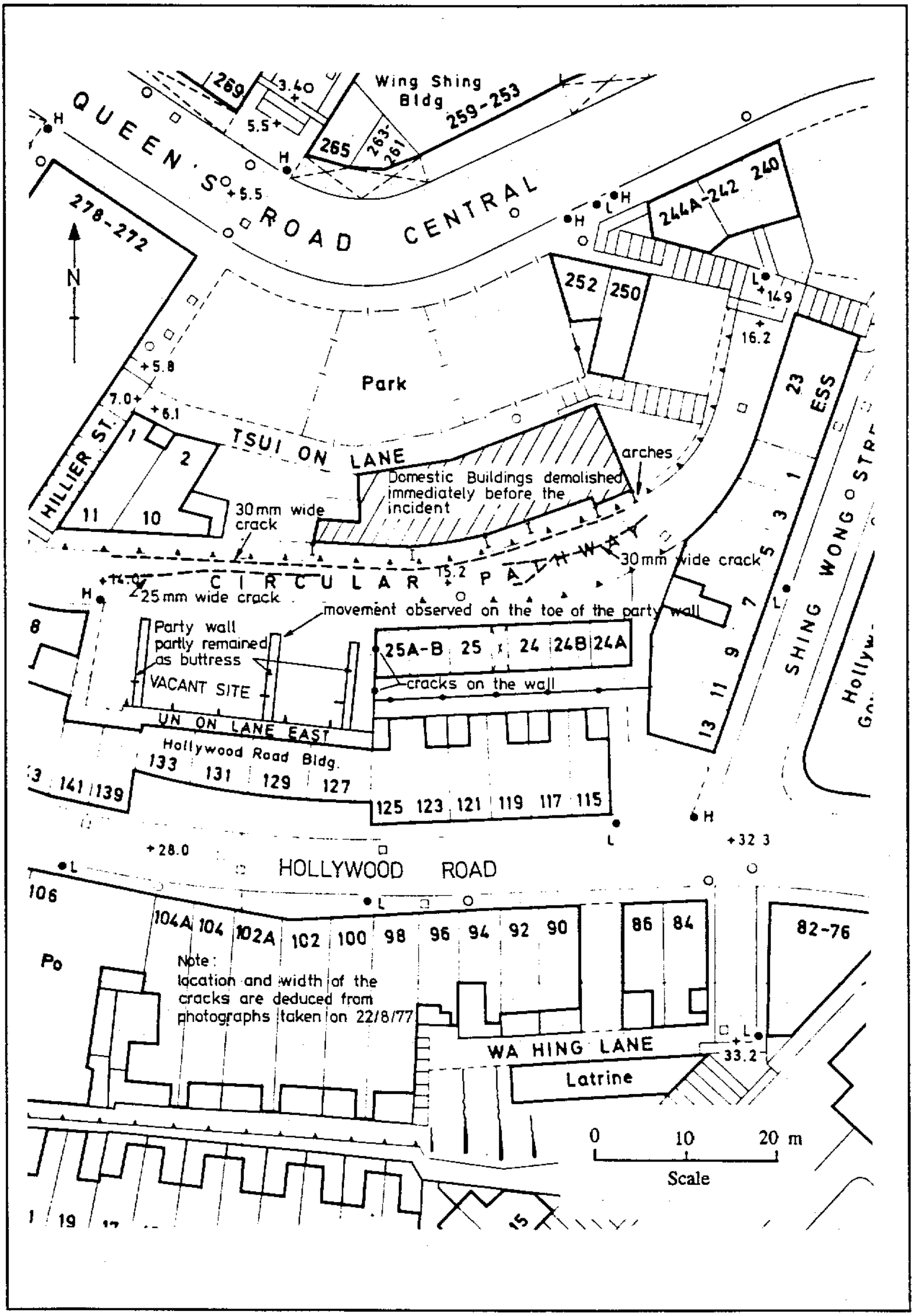


Figure D6.1 - Retaining Wall at Circular Pathway - Site Plan

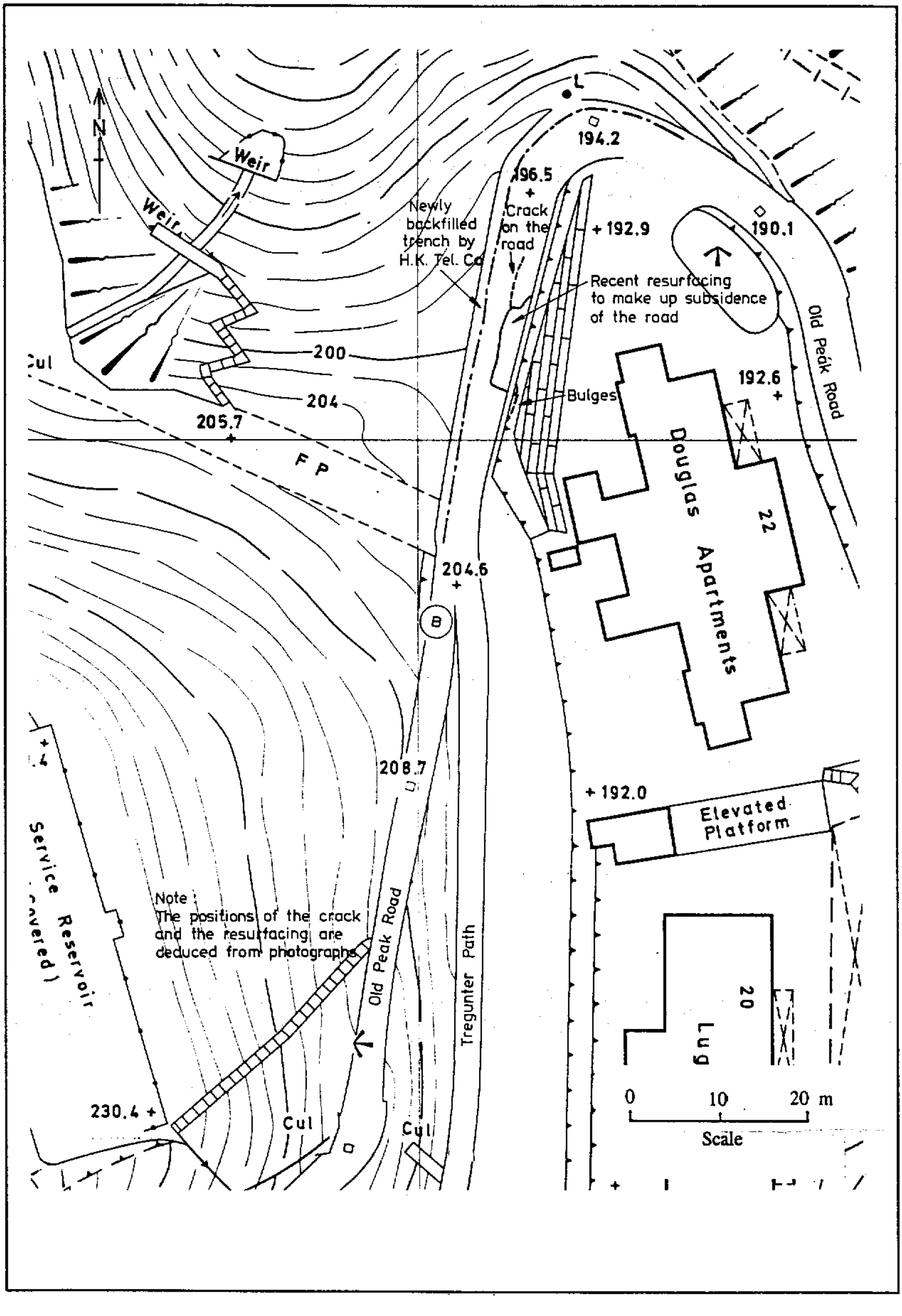


Figure D7.1 - Retaining Wall at 22, Old Peak Road - Site Plan

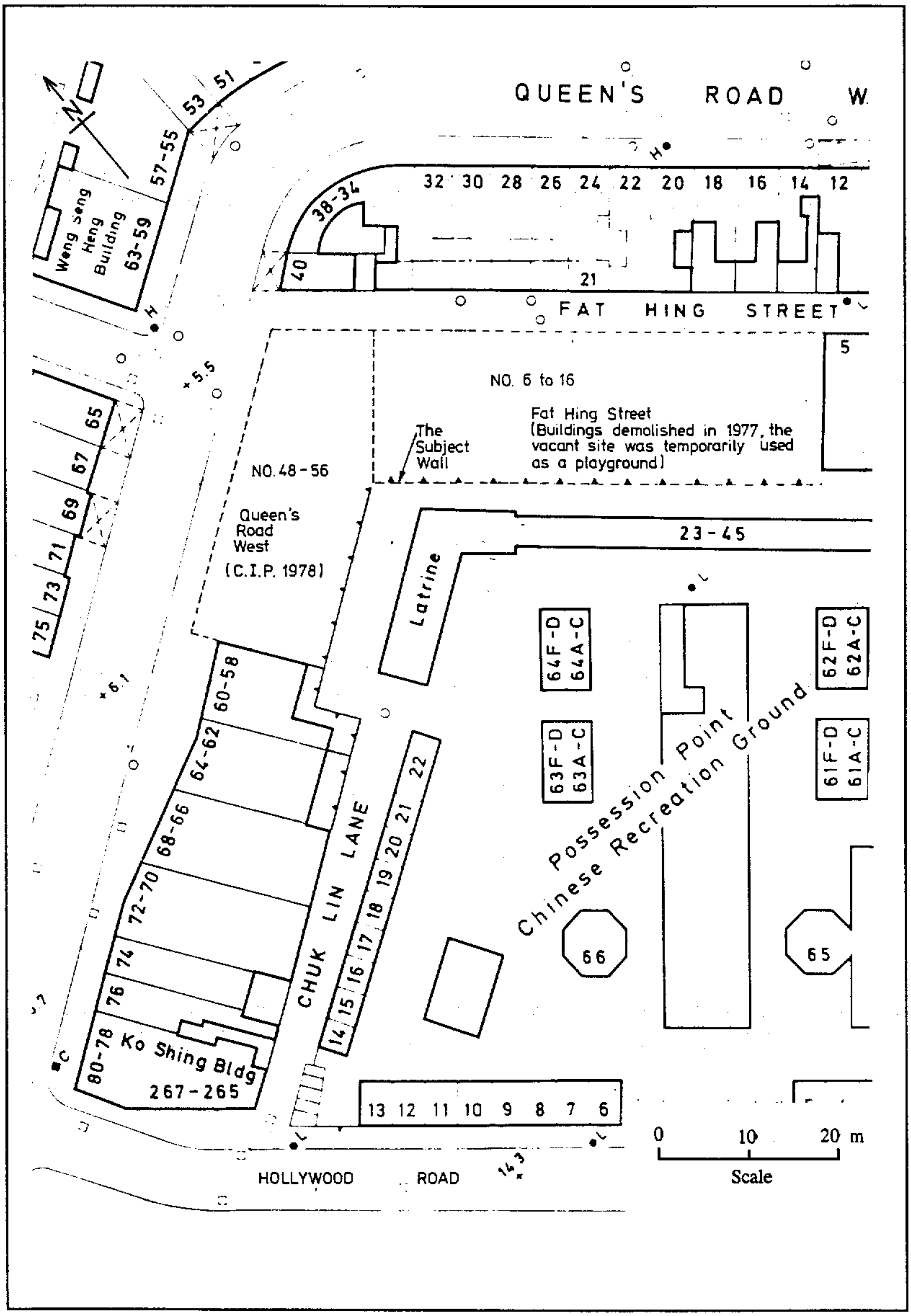


Figure D8.1 - Retaining Wall at Fat Hing Street - Site Plan

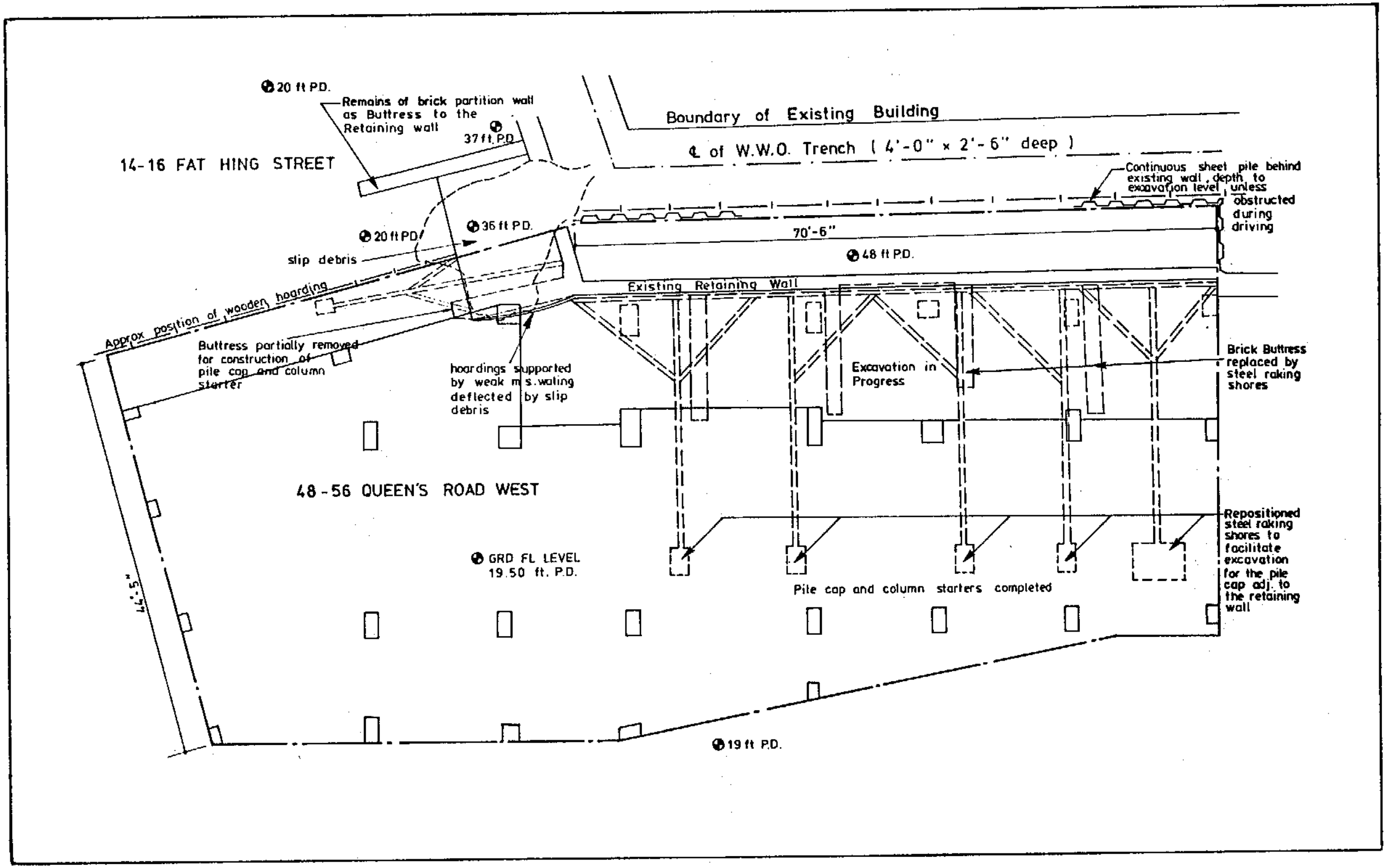


Figure D8.2 - Retaining Wall at Fat Hing Street - Approximate Extent of the Failed Wall and the Temporary Shorings on the Adjacent Site

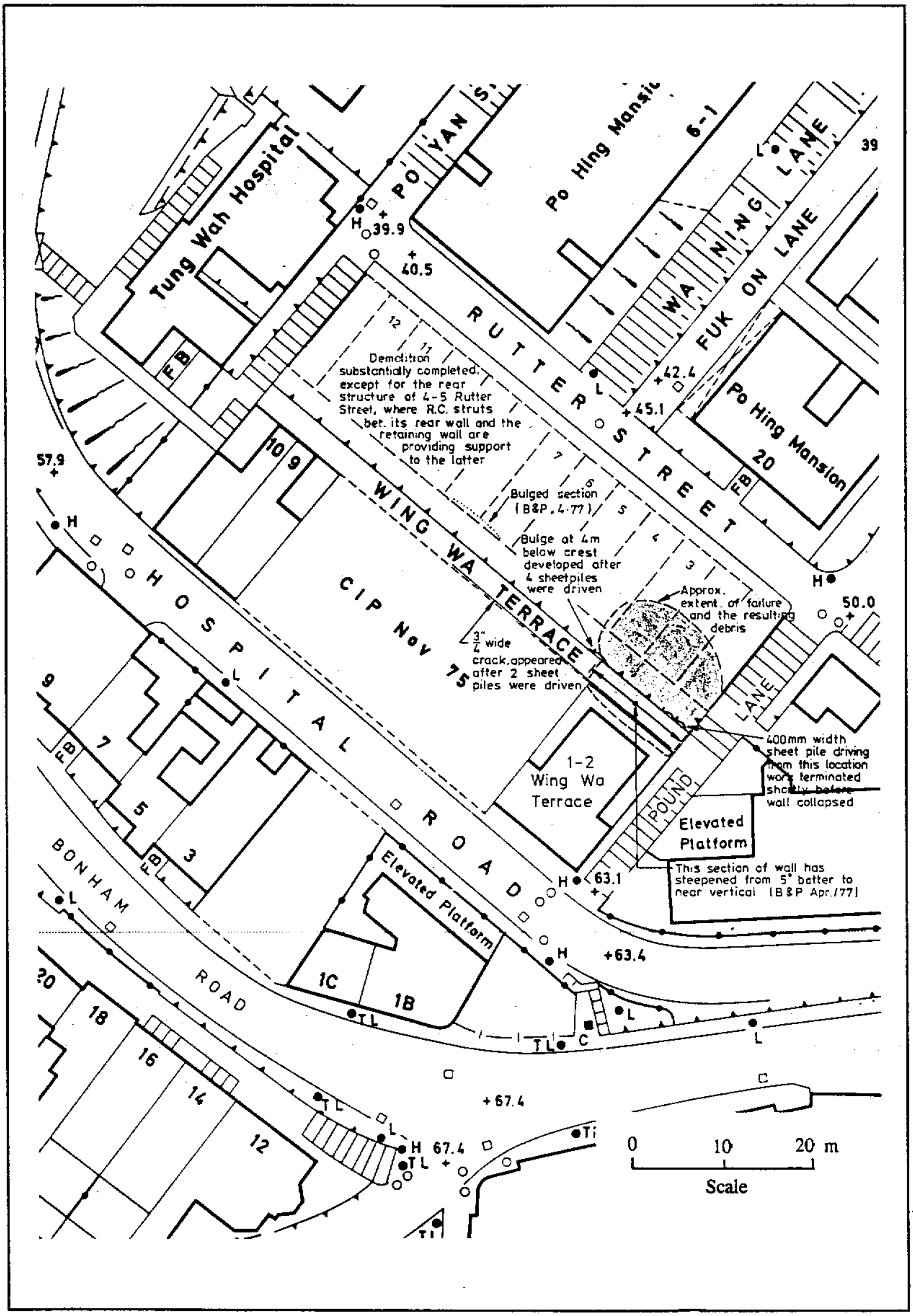
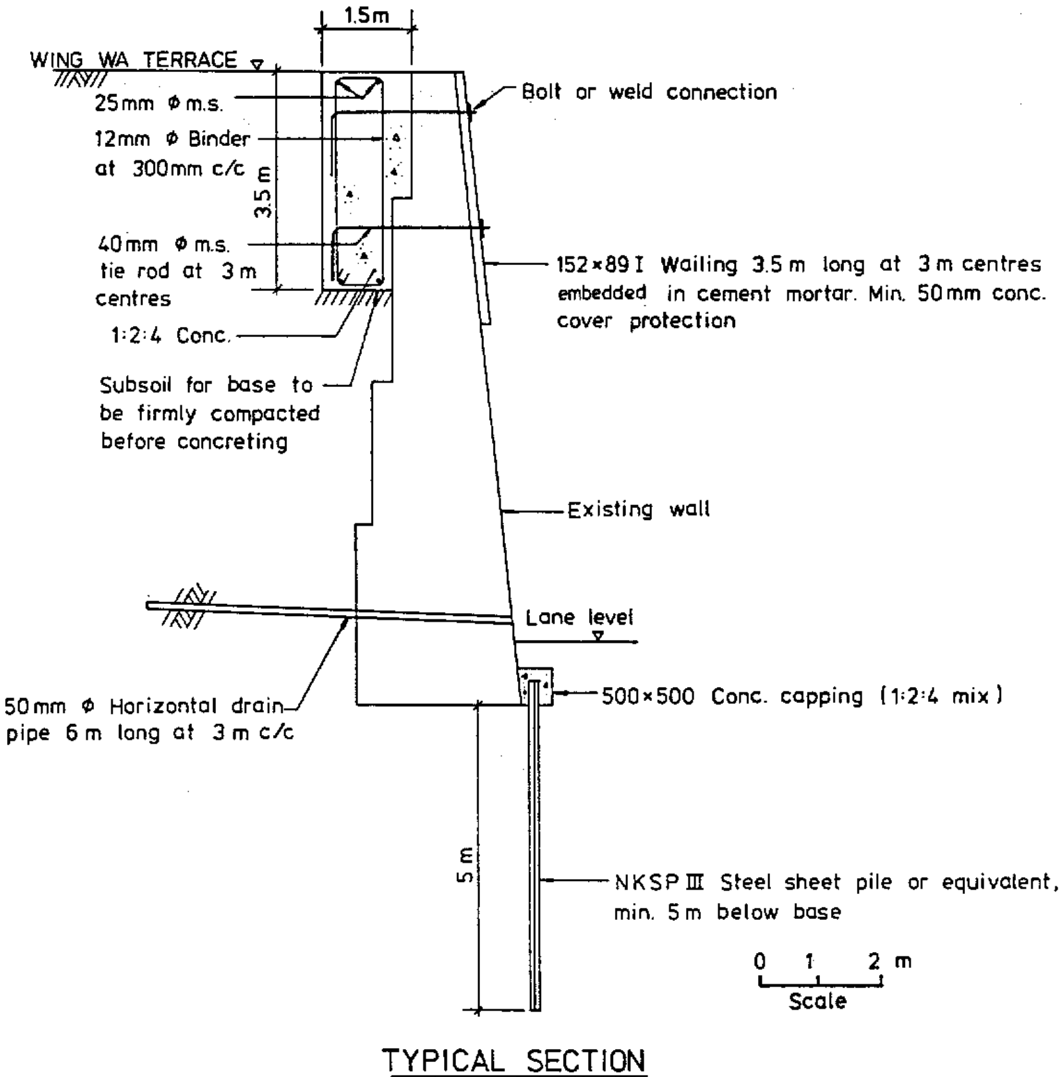


Figure D9.1 - Retaining Wall at Wing Wa Terrace - Site Plan



Notes:

1. Excavation to the conc. backing should be in stages of no longer than 6 m 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

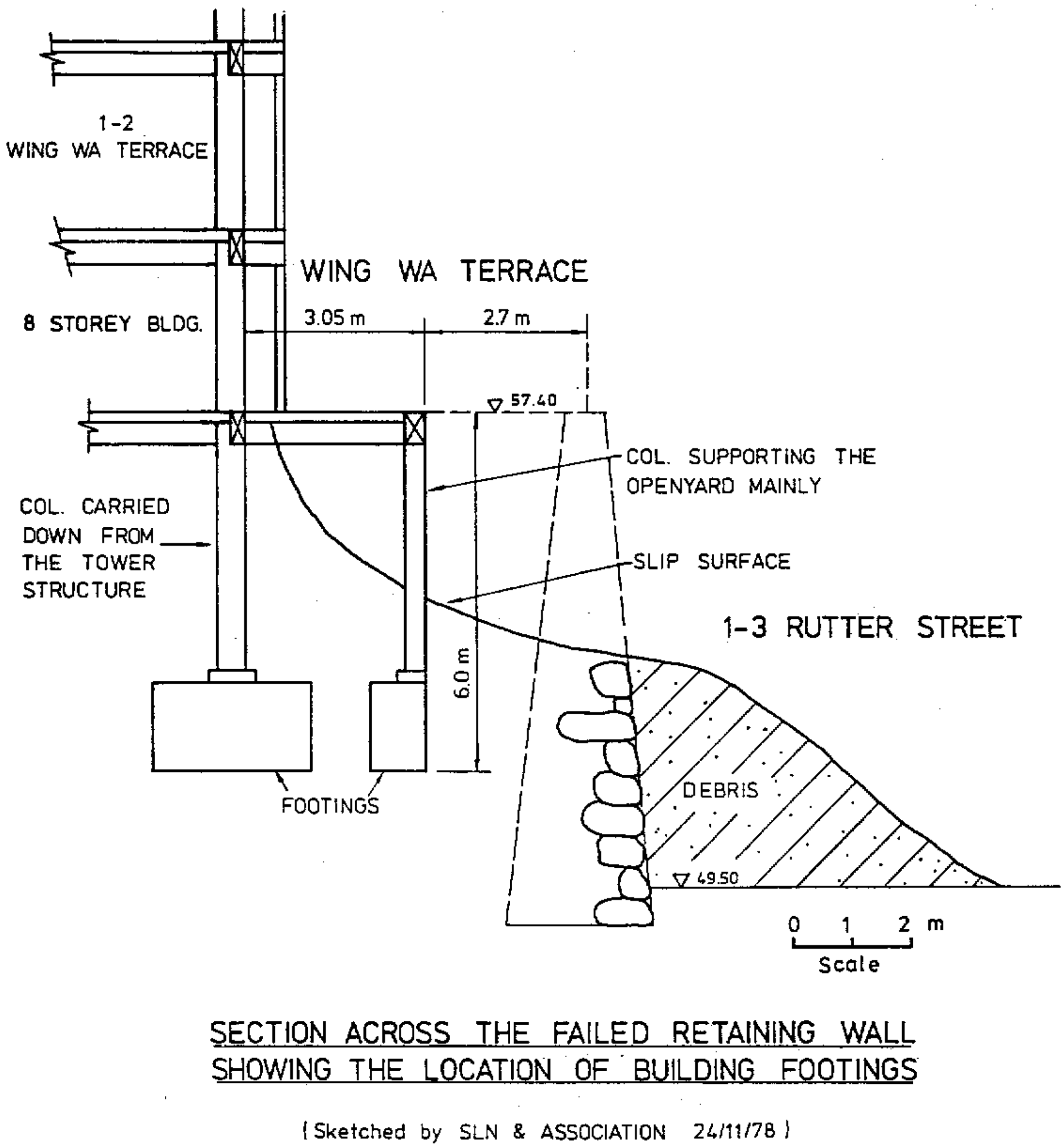


Figure D9.3 - Retaining Wall at Wing Wa Terrace - Section of the Failure

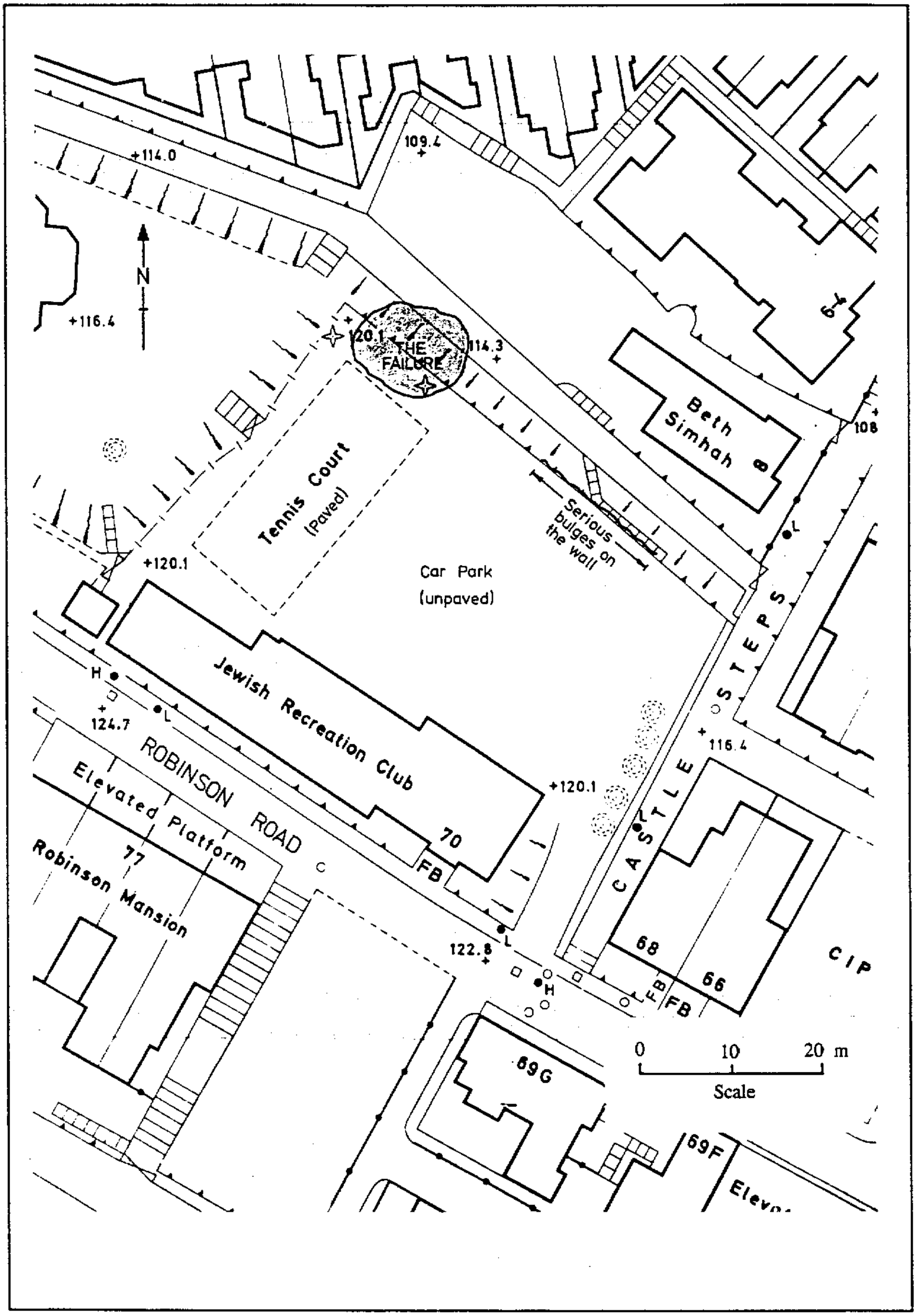


Figure D10.1 - Retaining Wall at the Jewish Recreation Club - Site Plan

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

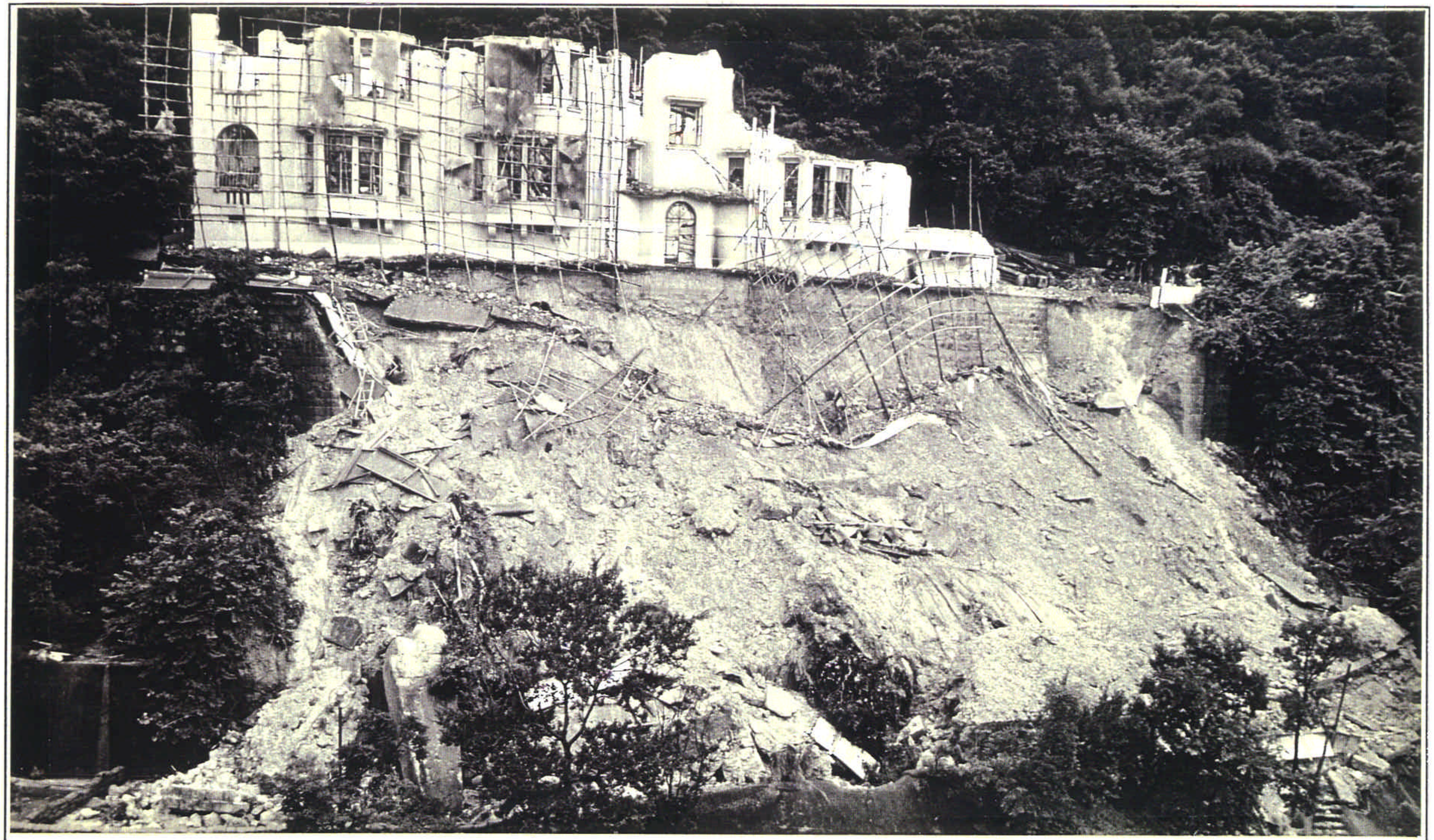
Plate D3.3 - Retaining Wall at Castle Road - Bulging of the Wall Caused Failure of the Plaster Layer

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

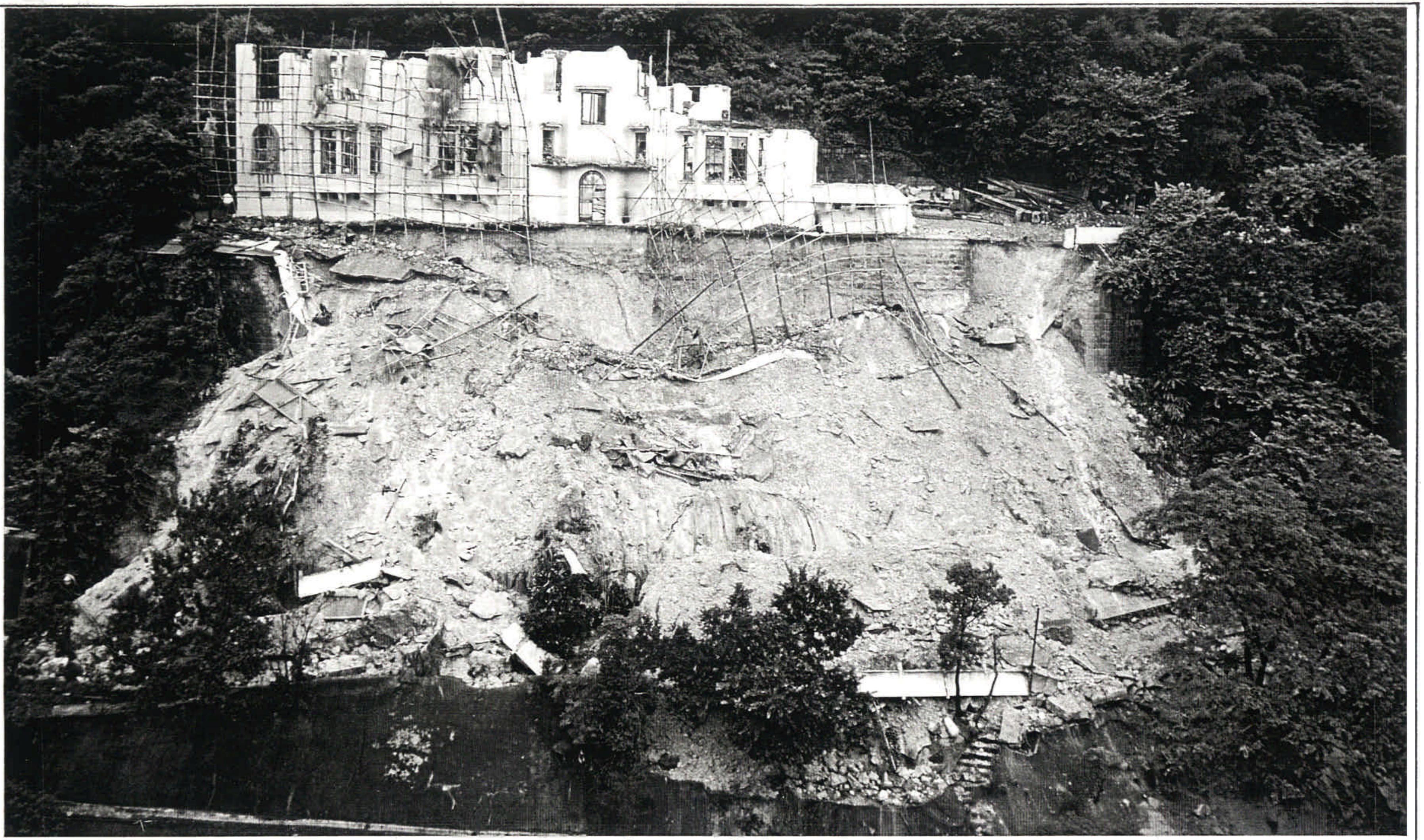
Negative No. 7/98/73

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)

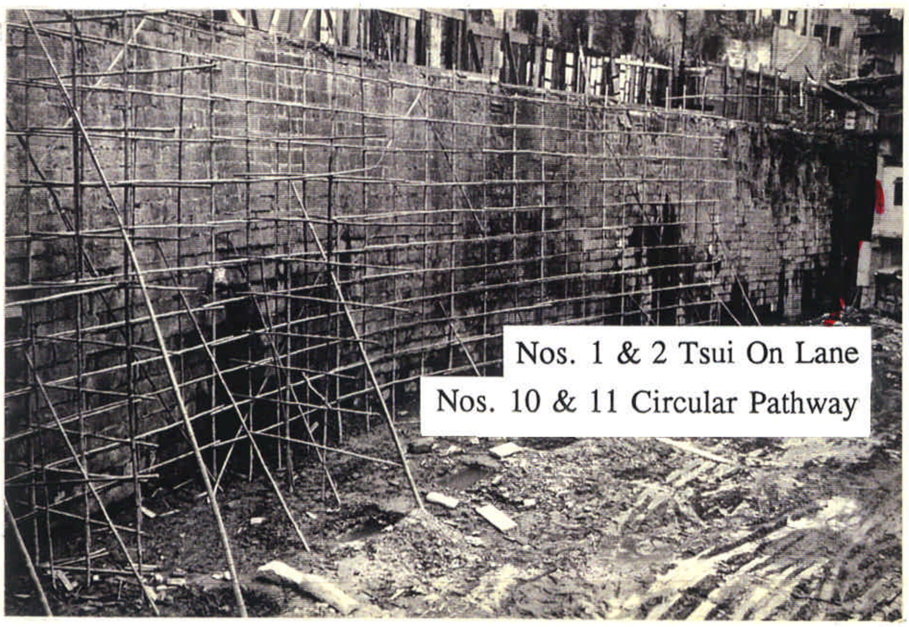


Negative No. 3/98/73

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

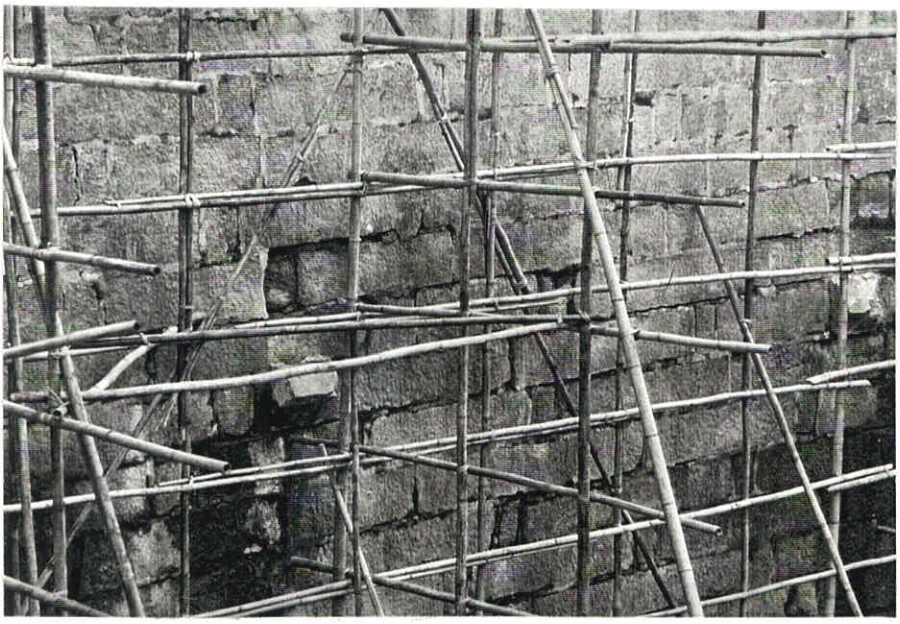


Negative No. 17/59/77

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

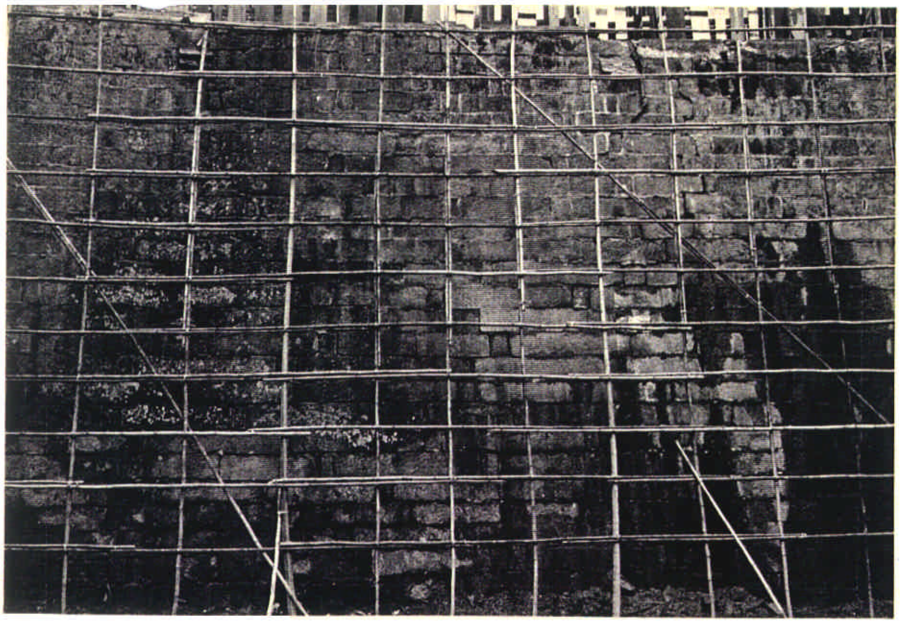
Negative No. 19/59/77

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



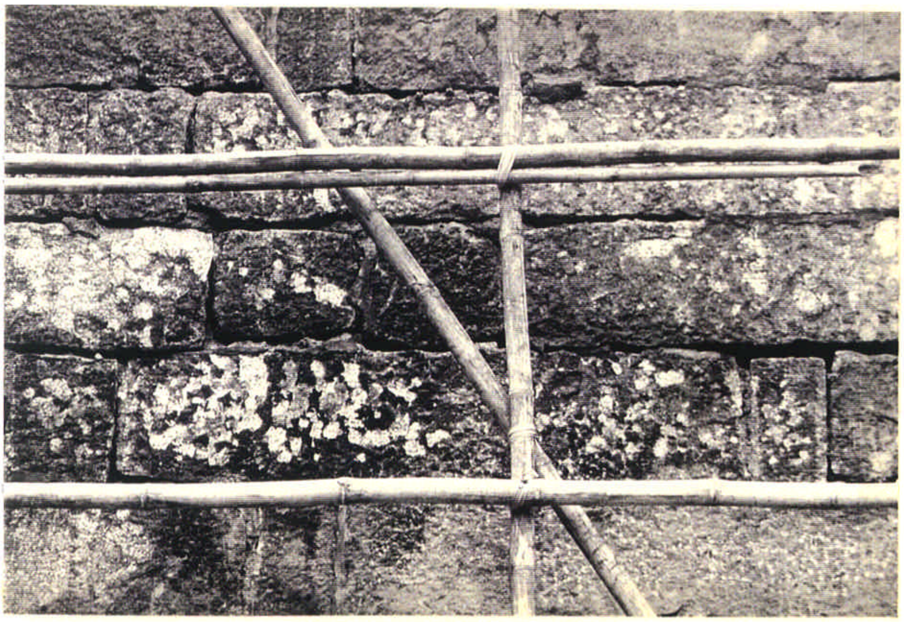
Negative No. 20/59/77

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



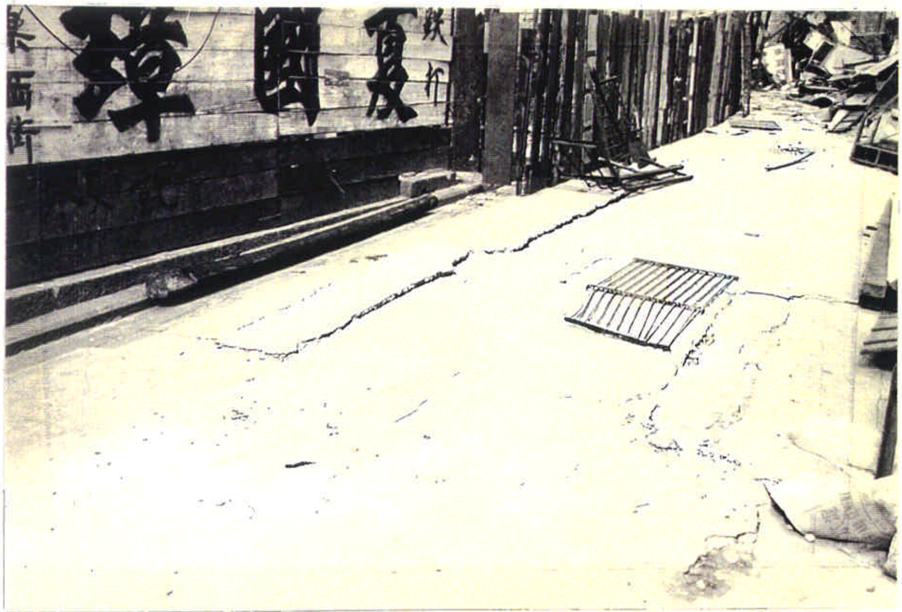
Negative No. 18/59/77

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



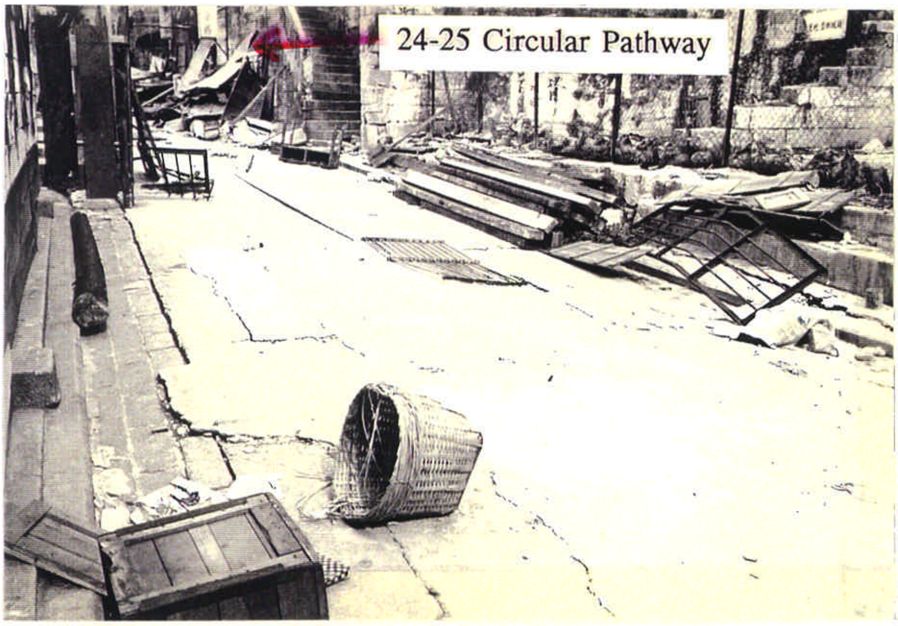
Negative No. 21/59/77

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



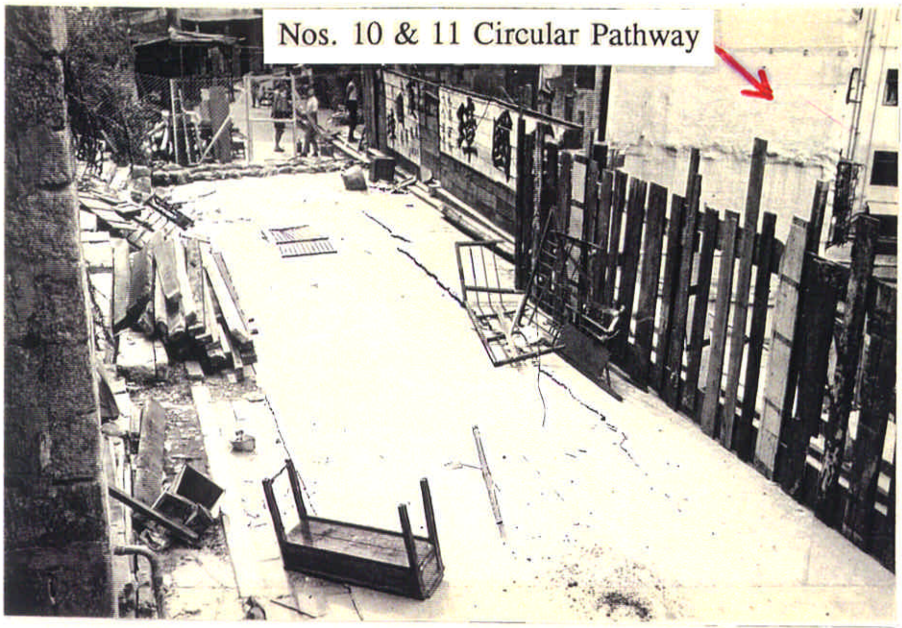
Negative No. 1/59/77

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

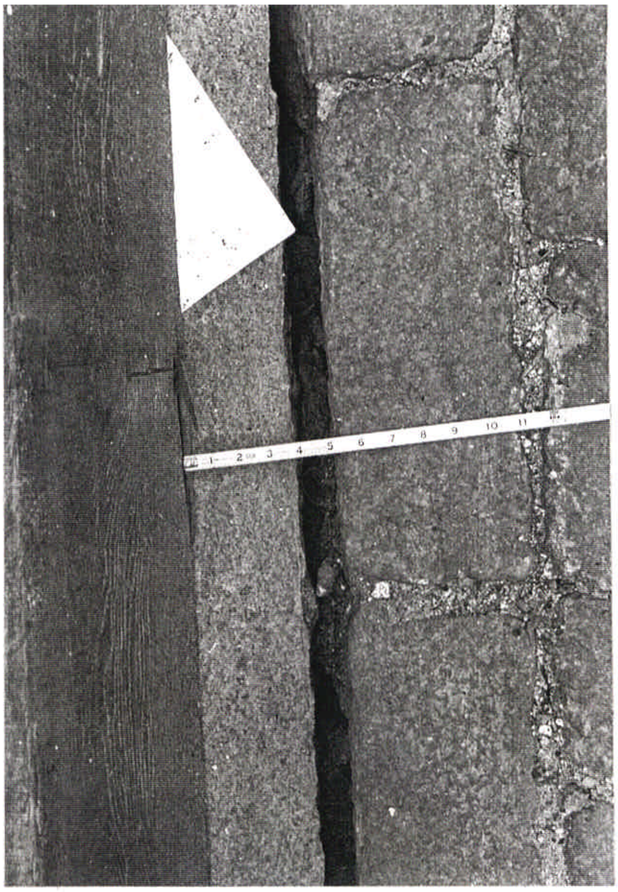
Plate D6.10 - Retaining Wall at Circular Pathway - Eastern Portion of the Pathway, Looking East

Negative No. 8/59/77

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

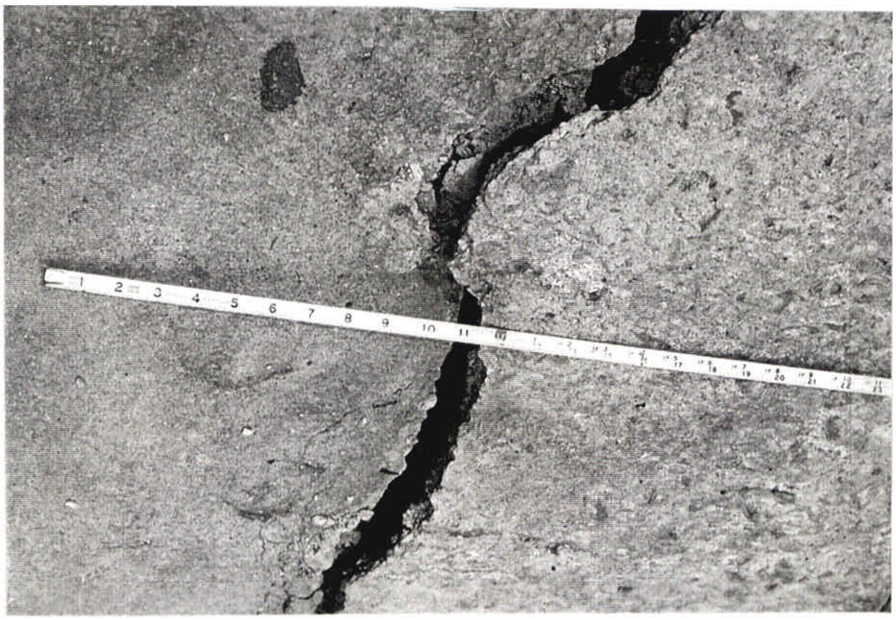
Negative No. 7/59/77

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



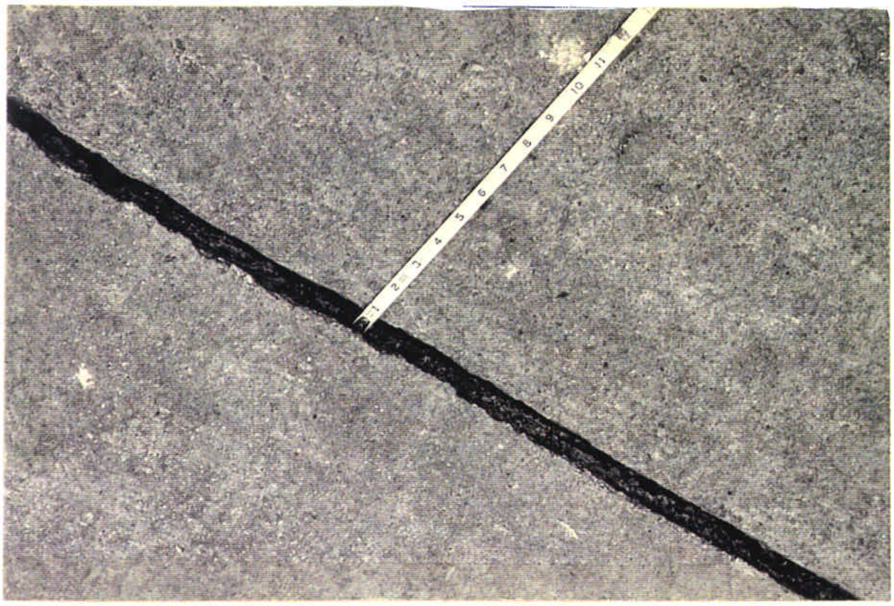
Negative No. 4/59/77

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



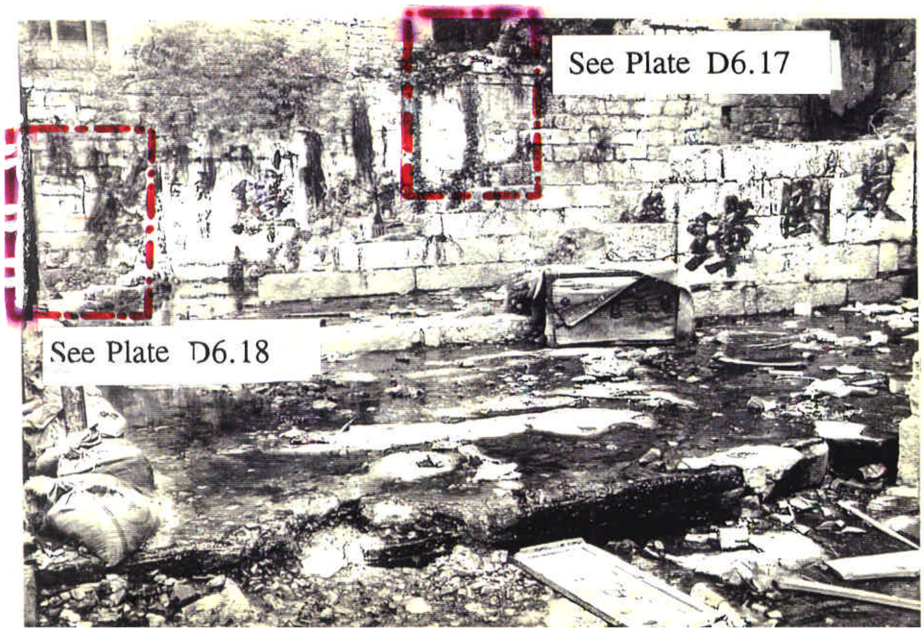
Negative No. 5/59/77

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



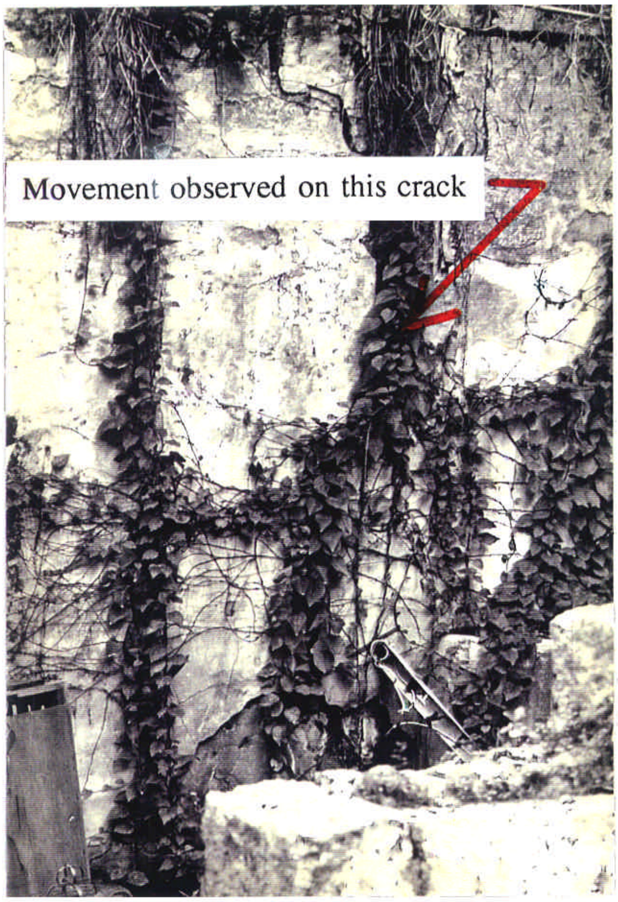
Negative No. 6/59/77

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



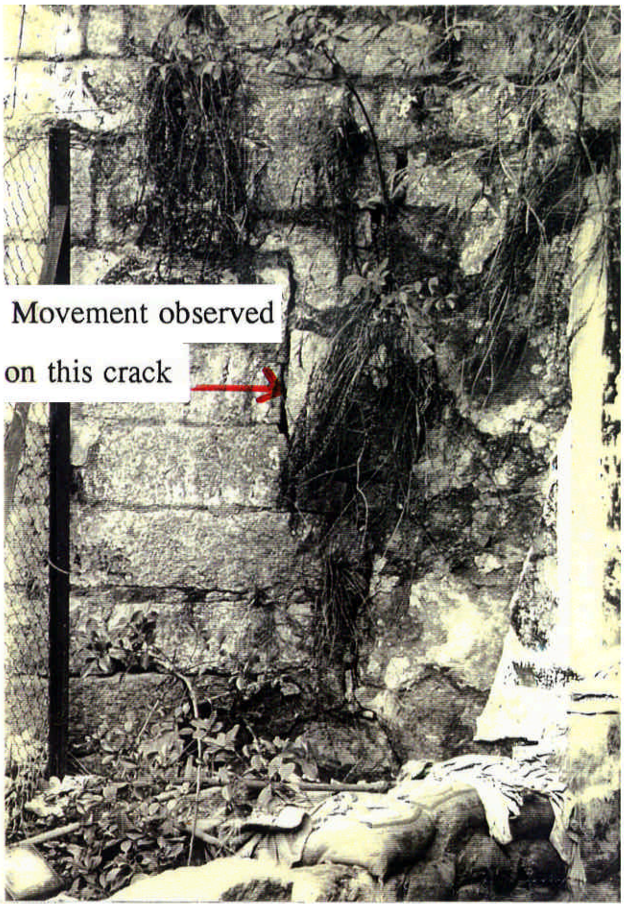
Negative No. 15/59/77

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



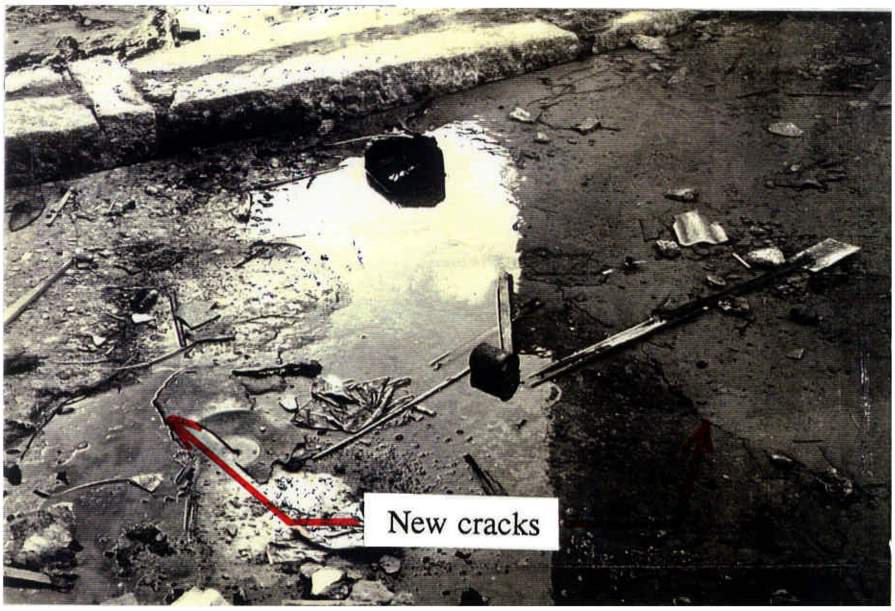
Negative No. 13/59/77

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



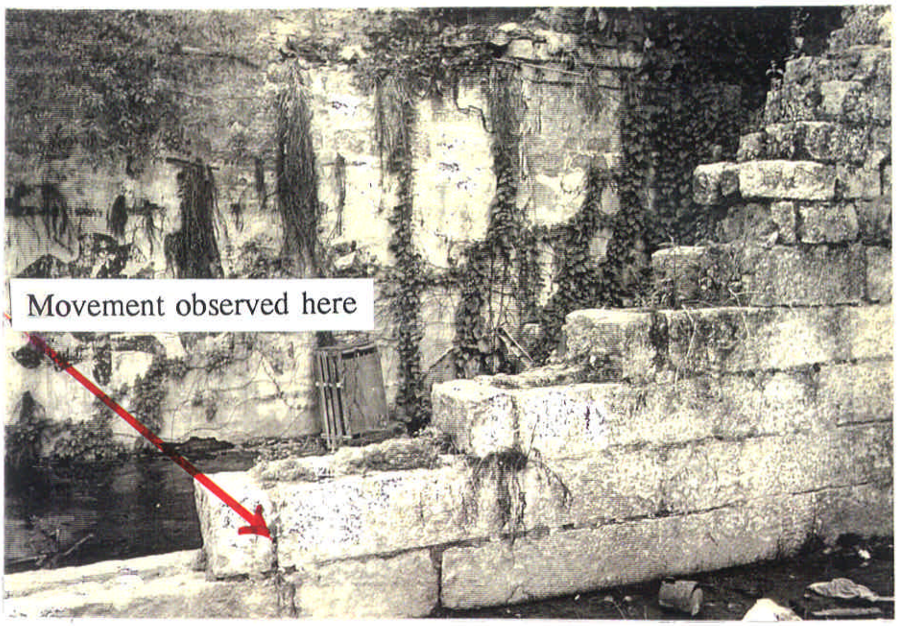
Negative No. 12/59/77

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

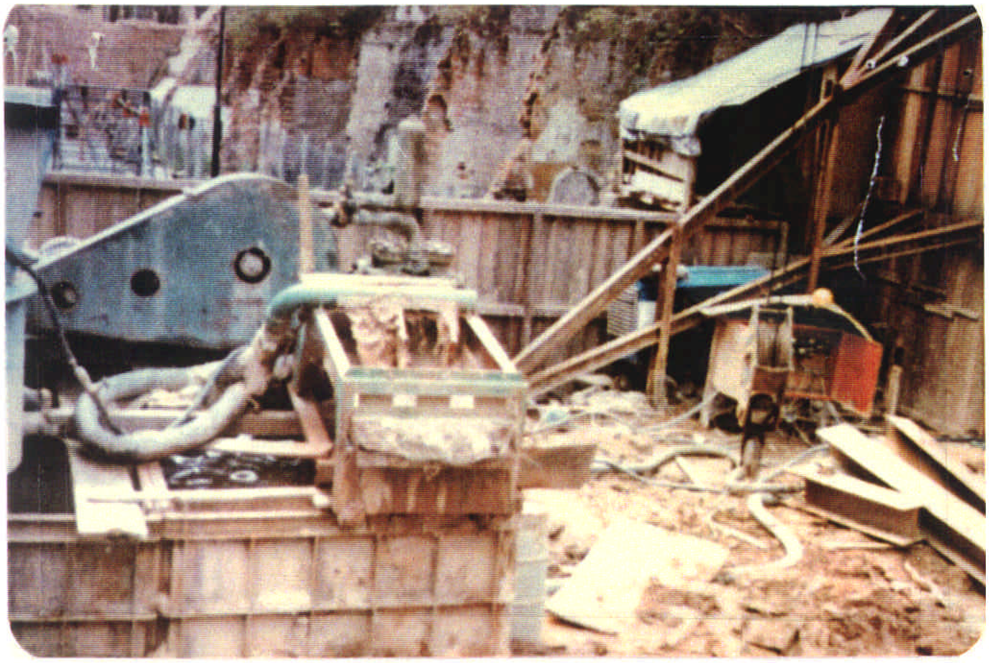
Plate D8.1 - Retaining Wall at Fat Hing Street - View of the Wall before Work Commenced on the Adjacent Site (I)

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)

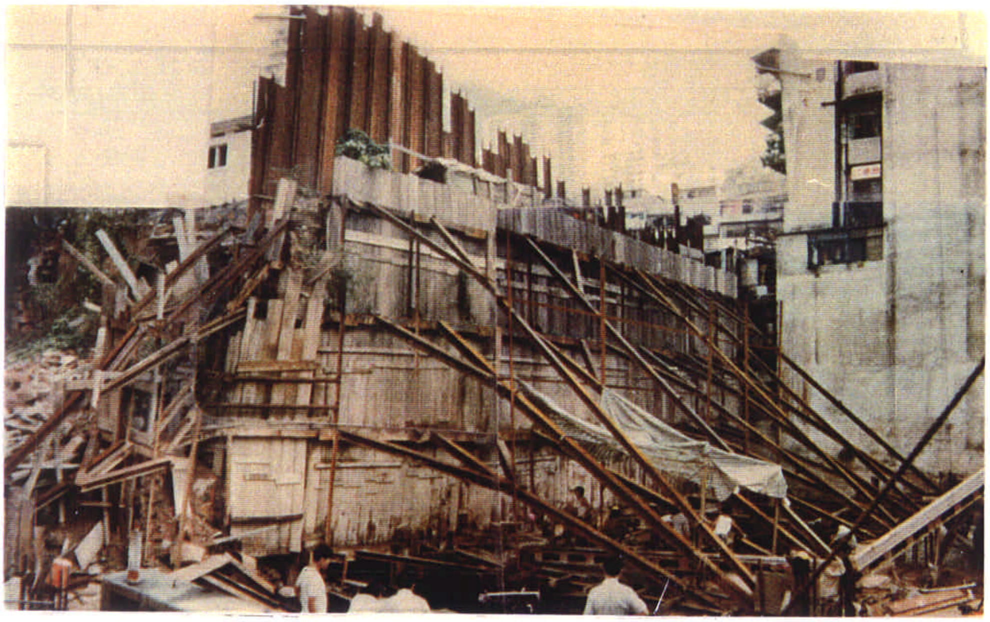


Negative No. R/W 81.17.14

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 m^2, 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 (m^2)

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/mm^2 with a maximum of 1.75 N/mm^2 for walls built in mortar designations (i), (ii) or (iii) or $0.15 + 0.6g_A$ N/mm^2 with a maximum of 1.4 N/mm^2 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

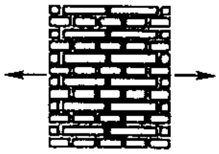
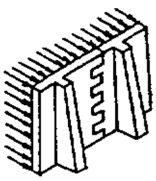
Compressive Strength of Bricks (MPa)	Compressive Strength of Mortar (MPa)					
	10	5	2.5	1	0.4	0
30	7	6	5.2	4.5	4.0	3.3
25	6.3	5.3	4.5	3.9	3.5	2.8
20	5.5	4.6	3.9	3.3	2.9	2.3
15	4.7	3.8	3.2	2.7	2.4	1.8
10	3.8	3.1	2.5	2.1	1.8	1.3
7.5	-	2.7	2.2	1.8	1.5	1.0
5	-	2.2	1.8	1.4	1.2	0.7

Notes :
(1) Nominal dimensions of brick 240 x 115 x 53 mm^3
(2) If special size bricks are used, apply modification factor

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

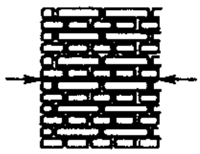
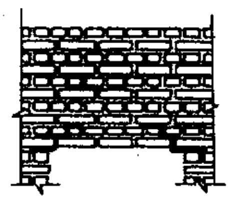
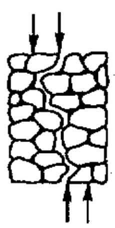
where h, L, are the height and length of the brick in mm.
(3) If pure cement/sand mortar is used, apply a modification factor of 0.85.
(4) For permissible strength apply a safety factor of 2.3 (Table 13 of Chinese Specification GBJ 3-73).
(5) This Table is reproduced from Table 1 of Chinese Specification 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)

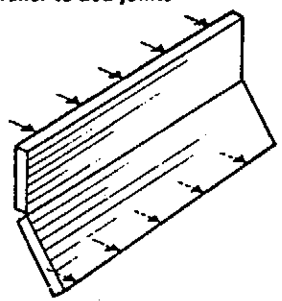
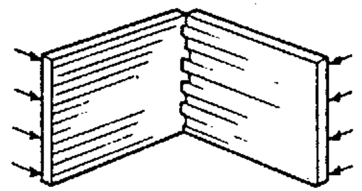
	Plane of failure parallel to bed joints 			Plane of failure perpendicular to bed joints 		
Mortar designation	**(i)**	**(ii) and (iii)**	**(iv)**	**(i)**	**(ii) and (iii)**	**(iv)**
Clay bricks having a water absorption						
less than 7 %	0.7	0.5	0.4	2.0	1.5	1.2
between 7 % and 12 %	0.5	0.4	0.35	1.5	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/mm²: 2.8	0.25		0.20	0.4		0.4
3.5				0.45		0.4
7.0				0.60		0.5
10.5				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)	Proportions by Volume
B	4.2 - 17.5	1	1 to 1¼	Not over 6 parts)	
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		

LIST OF FIGURES

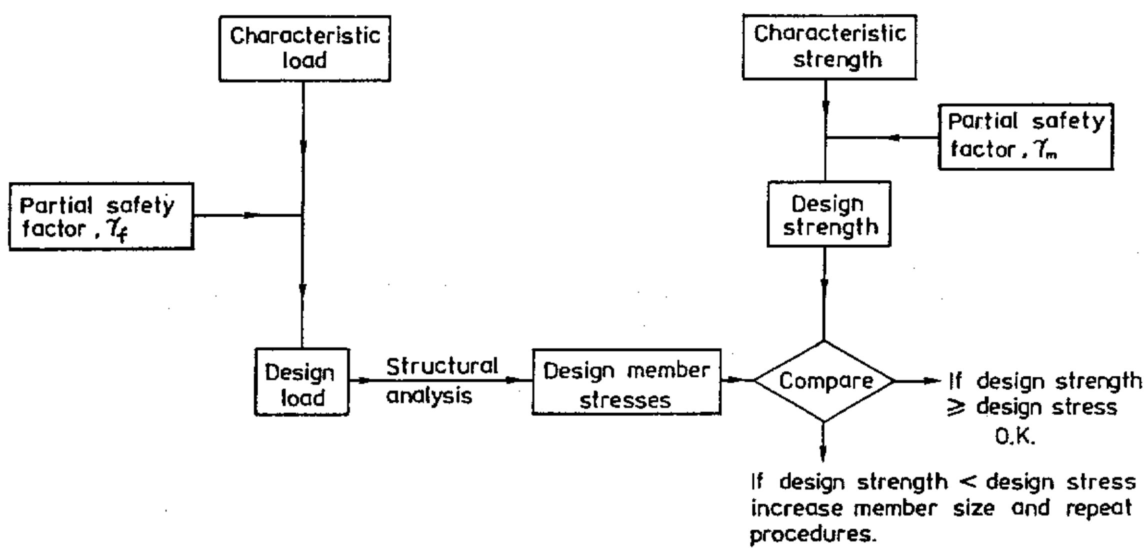


Figure E1 - Procedures of Limiting State Design

APPENDIX F

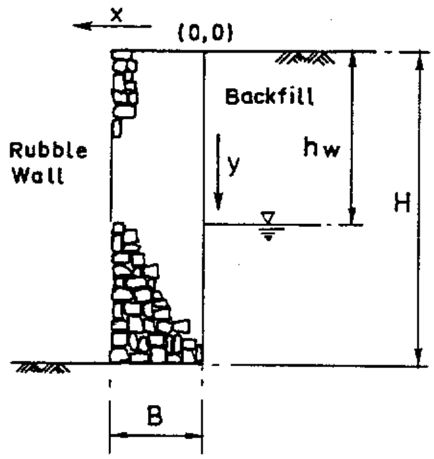
ANALYTICAL SOLUTIONS ON THE DISTRIBUTION OF STRESSES IN A GRAVITY RETAINING WALL

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 $(y-h_w)$ vanish and h_w is replaced by y.

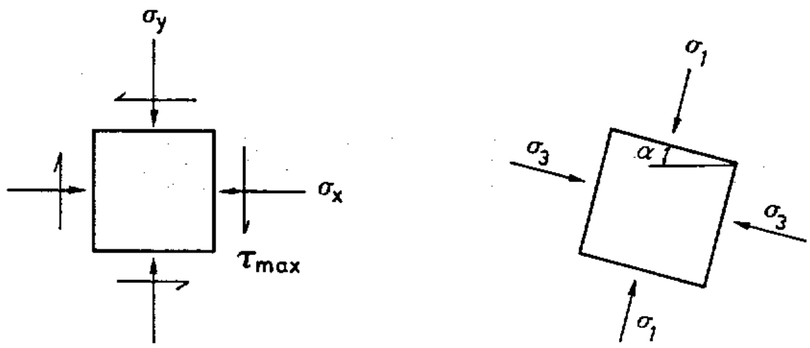
$$\sigma_{1,3} = \frac{\sigma_x+\sigma_y}{2} \pm \sqrt{\left(\frac{\sigma_x-\sigma_y}{2}\right)^2 + \tau^2}$$

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

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

$$(F.O.S.)_{sliding} = \frac{2S_c}{\sigma_1-\sigma_3}$$ = 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 :-

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

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

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

(9) All other assumptions pertaining to the beam theory

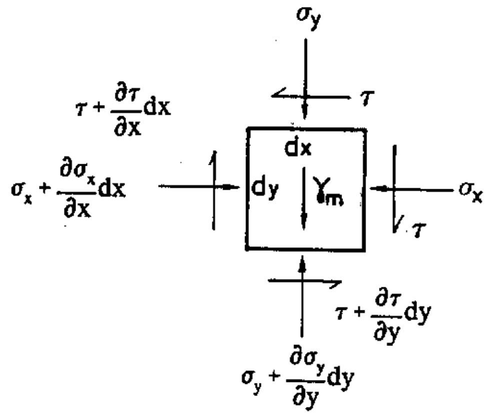
F.4 Derivation of Stress Equations :-

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

$$\therefore \quad \sigma_x = -\int \frac{\partial \tau}{\partial y}dx \quad \text{\_\_\_\_\_①}$$

$\Sigma F_y = 0$ then

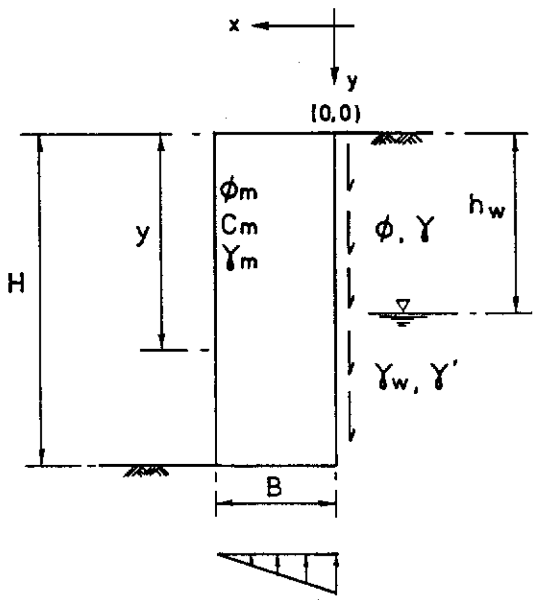
$$(\sigma_y + \frac{\partial \sigma_y}{\partial y}dy)dx + (\tau + \frac{\partial \tau}{\partial x}dx)dy - \sigma_y dx - \tau dy - \gamma_m dxdy = 0$$

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

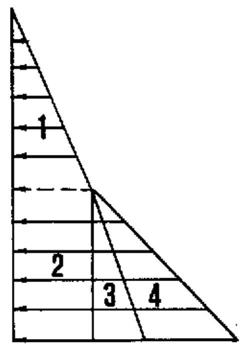
$$\therefore \quad \tau = \int (\gamma_m - \frac{\partial \sigma_y}{\partial y})dx \quad \text{\_\_\_\_\_②}$$

F.4.2 <u>External Forces Acting on the Wall</u> :-

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

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

F.4.4 Normal Stress, σ_y :-

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

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 Shear Stress, τ :-

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

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

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

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

which also satisfies boundary condition (ii)

Simplifying

$$\sigma_x = K_a\gamma'(\frac{x^2\tan\delta}{2B} - x\tan\delta) + \frac{x^2M''}{B^2}(\frac{2x}{B} - 3) + [K_a\gamma h_w + K_a\gamma'(y - h_w) + \gamma_w(y - h_w)] \qquad ⑩$$