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Difficulties with ground anchorages in hard rock in Dublin, Ireland

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ABSTRACT

Engineers in Dublin favour founding structures on the hard interbedded limestone, which underlies the city. Despite the apparent good ground conditions problems have been experienced on several sites. A detailed description is given of the problems encountered during ground anchorage installation at the Jervis St. shopping centre site. In all 72% of the 443 anchorages encountered some difficulty. The pattern of zones within this site, where the difficulties were encountered, and the location of all 5 sites where problems have been recorded correlated well with the known bedrock conditions in Dublin. The case histories detailed here show that the process of construction (particularly drilling technique) and the effects of groundwater need to be carefully considered when designing underground works in hard interbedded limestone. Design and construction issues cannot be separated and this calls for more collaboration between designers and contractors.

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1. INTRODUCTION

The bedrock, which underlies central Dublin, is in general a succession of strong dark argillaceous limestone layers interbedded with weaker shales or mudstones. Engineers tend to favour founding conventional pad or strip foundations or end bearing piles on this stratum, rather than the overlying glacial deposits. Little thought is given to the effects of construction on the rock. The main objectives of this paper are to illustrate how significant problems can be encountered when using this stratum as a load bearing layer and to show that the process of design and construction need to be interlinked. Some geological background is first given along with a summary of previously reported problems. Most of the paper is devoted to giving a detailed account of the difficulties encountered during construction of rock anchorages at the site of the Jervis St. shopping centre.

2. BEDROCK GEOLOGY OF DUBLIN

Nolan (1985) describes the solid geology of Dublin in detail and the following is taken from his work as well as from that of Farrell and Wall (1990). Sedimentary calcareous rock (i.e. limestone) was deposited in a shallow sea in the Dublin area during the Lower Carboniferous period, some 330 to 360 million years ago. Within the shallow water region there were changes in the depositional conditions which are reflected in marked changes in the properties of the rock and in its thickness. These changes arose from the cyclical change sin water depth during this period. This has resulted in a variation in the sand and clay content of the limestone rocks and the inclusion of shale or mudstone

layers, some of which have weathered to form clay. The colloquial term used to describe these limestones is “calp”.

Evidence of folding and faulting in the Dublin limestone is sparse, due to the thick cover of overburden, which covers much of the city. Nolan (1985) indicates several faults with an orientation of south-west to north-east, see Figure 1. However faults of engineering significance would be expected to occur at more frequent intervals than shown in Nolan’s work. Various authors have suggested that the pre-glacial channel, which runs roughly along the course of the present day River Liffey, could be due to a structural weakness in the rock. Conditions during the period immediately before the ice age led to erosion and alteration of the rocks at “rockhead level”. This included the formation of buried rock channels and the erosion of the calcium, which has returned some of the muddy layers to clay.

There is no available record of solution features within the Dublin limestone, though Meehan and Parkes (1997) describe a small cave within the quaternary deposits at Parkgate St, close to the Jervis St. site (see Figure 1).

The limestone beds dip within the range of 5° to 30° and have typical layer thickness of 300 mm to 400 mm. Unconfined compressive strength values are normally of the order of 100 MPa.

3. PREVIOUS CONSTRUCTION DIFFICULTIES / EXPERIENCE

Prior to this contact ground anchorages had been installed successfully on several sites in Dublin. Examples from Ove Arup and Partners, Ireland files include those for the Liffey Valley Bridge (M50 Motorway) to the west of Dublin and for the RTE Radio

studio in Donnybrook to the south of the city. No difficulties were encountered in either of these sites. The first author was involved in a project where anchorages were installed into very similar rock at Leixlip Dam to the west of the city (O'Tuama and O'Mahony, 1989). Again there were no problems encountered, except for some minor collapses which resulted in the redrilling of a few boreholes.

Significant problems, during large diameter piling in bedrock at George's Quay near Dublin city center (see Figure 1), have been detailed by Lee (1993, Ref. 15 and 1994, Ref. 16). Bedrock at the site consisted of interbedded strong "calcisilite" limestone, weak weathered limestone and completely weathered (clay like) rock. The site was subdivided into 2 zones of good rock and poor quality rock. The buildings are founded on large diameter (0.75 m to 1.05 m) bored cast in situ concrete piles.

A pair of preliminary pile tests was carried out in each zone, one with a soft toe to enable skin friction to be determined, the other with a normal toe. In the zone of poor rock, testing of the "normal" pile proved successful with a 750 mm dia. pile settling approx. 5 mm at SWL (=3000 kN) and 25 mm at 3 SWL. In the zone of good rock testing showed significantly more settlement than originally envisaged and was between 23 mm for the soft toe pile and 13 mm for the "normal" pile. Although the ultimate bearing capacity of the piles was adequate, the settlement values were deemed to be unsatisfactory for the particular buildings involved as the columns were each supported on a single pile and it had relatively tight differential settlement criterion.

A programme of rock coring was undertaken to investigate the rock quality. It was concluded that the problems were caused by the damage to the bedrock by the large and powerful equipment used to drill the rock and exacerbated by the deterioration of the

rock surface due to the presence of water in the drilling process. The problem was overcome by a review of the rock socket lengths in the two zones.

Long and Collins (1999) report on problems during a small diameter piling contract (with air flush as the drilling medium) being undertaken for an industrial development near Heuston Station in Dublin, see Figure 1. The project was relatively routine, save for the fact that access to the building was only available during the night as production in the building had to continue during the day. Due to time pressure and against his better judgment the contractor was forced to drill some piles one day and leave the bore open until the piles were grouted the next day.

A number of failures under static test occurred. The contractor reasoned that the failure was due to the deterioration in the surface of the “calp” limestone overnight during which a slickensided surface developed. This process was likely to have been aided by ground water penetration. The problem was overcome by installing replacement piles, which were drilled and grouted in one day.

Finally Wall and Farrell (1990) point out that the weak layers within the “calp” limestone can give rise to problems with end bearing piles as occurred for example at the Civic Offices site on the opposite side of the River Liffey from the Jervis St. site, see Figure 1.

4. JERVIS ST. SITE

4.1 General

The Jervis St. Shopping Centre was constructed between July 1995 and November 1996. The site, which occupies an area of some 0.9ha, is located approximately 250m

north of the River Liffey between Mary St., Jervis St. and Upper Abbey St. in Central Dublin, see Figure 1.

The site area remained undeveloped until relatively recent times. Development of the area commenced in 1677 and the area became a fashionable residential quarter. Jervis St. hospital was constructed on the site in 1886. This building included a number of historically important façades, which were retained in the new development, and a single level basement constructed of heavily reinforced concrete. The hospital buildings were demolished to street level in the late 1980's leaving a generally flat area, at an elevation of about +3.5mOD.

4.2 Ground conditions from site investigations

Two site investigations were carried out prior to the development. Initially nine shell and auger boreholes, five of which were extended by rotary coring, were drilled and some laboratory testing were carried out. The second investigation involved the construction of a pumping well together with two observation holes. A summary of the ground conditions revealed by these investigations is given in Table 1. A general geological section through the site, together with details of the basement excavation, is shown on Figure 2. In the context of this paper the critical materials are the weathered and intact rock and these are described in more detail as follows:

A summary of the rock coring is shown on Figure 3. It was described on the borehole logs as a dark grey fine grained slightly weathered limestone interbedded with zones of black very fine grained severely weathered limestone. Unconfined compressive strength tests carried out specimens of the intact rock gave values in the

range 80 to 100 MPa. A summary of the rock coring parameters including total core recovery (TCR), rock quality designation (RQD), solid core recovery (SCR) and fracture spacing index (FSI) is given on Table 2.

The discontinuities are described as being roughly horizontal and the fractures are often closely spaced. The percentage of “severely weathered and fractured limestone” to intact limestone varies between 0% in BH1 to 27% in BH5, with an average of about 10%. This weathered rock generally had the appearance either of blocks of limestone surrounded both vertically and horizontally with bands of black silty clay or of cobbles and boulders of limestone in a matrix of silty clay. A photograph of a cutting (for a pad footing) in the weathered rock is shown on Figure 4.

4.3 Pumping trials

This work comprised a pump hole in rock and two observation holes in the gravels near to the center of the site (close to BH5). During the main test the ground water level was lowered by 4.35 m by pumping 68 l/min. Little effect was noticed on the water levels in the gravels, due to this stratum being hydraulically separated from the rock by the boulder clay layer at this location on the site. The main test was preceded by a small scale test where a lower volume of water was pumped. According to the test report the amount pumped in the main test was larger partly because “the surging action created during the pumping dislodged and removed the clayey silty infilling between joints and thereby increased the pathways for the flow of water”.

4.4 Development

The shopping centre development included a two level basement. Dig depth varied between about 5.5 m along Upper Abbey Street to about 7.25 m along Mary St., see Figure 2. The maximum dig depth for foundations was about 9.7m below ground level, which is about 6.5m below the water table. It is important to note that the excavation involved removal of rock material over a large part of the site.

A secant pile wall provided excavation support. This wall comprises of interlocking male and female piles, which are 900 mm dia. through the overburden, reducing to 810mm dia. in the rock. The overlap between the 900mm dia. piles is 130mm resulting in a centre to centre distance between the male piles of 1.54m. The male piles were constructed using C35N concrete and are reinforced with 8 No. T32 reinforcing bars. The female piles were constructed of C7N concrete and were un-reinforced. They were designed to have a toe level of at least 0.5m below the proposed excavation level to ensure an adequate water cut-off. For structural reasons the male piles needed a greater toe-in, see Figure 2. In areas where the imposed vertical loading is high, or sensitive structures are located adjacent to the wall, the minimum depth of embedment was set at 1.5m into intact bedrock. In areas were the vertical loading is significantly lower and only pavements are located next to the wall the minimum depth of embedment was set at 0.75m.

Full details of the design, construction and performance of the retaining structure are given in Dougan et al. (1996).

5. GROUND ANCHORGE DESIGN

As the basement slab is located beneath the ambient ground water level, some measures were required to prevent uplift. During construction it was decided that pumping would be maintained in order to ensure that no build up of water pressure occurred. However a long term pumping solution was unacceptable to the client for maintenance reasons. It was also decided that the 1.1 m thick slab, which would have been require to resist uplift loading, was uneconomic. It was therefore decided to install rock anchorages to provide long term uplift resistance.

The anchorages were designed to the requirements of BS8081:1989 “Ground Anchorages” and comprise 9m long, 50 mm dia. single bar anchorages. They were installed in a 125 mm dia. borehole on an approximate 2.5m x 2.5m grid. The entire 9 m length is in effect the “fixed length”. Double corrosion protection was included. They are designed to sustain a working load of 500kN. This allowed for some loading due to stress relief in the bedrock. A total of 443 anchorages were installed. A detail of the basement slab / anchorage head is shown in Figure 5.

6. BASEMENT CONSTRUCTION / DEWATERING

6.1 Basement construction

Excavation and construction of the basement and foundations commenced in the centre of the site as the secant piled wall was being constructed. The contractor elected to do this under the restriction that the water level in the gravels around the perimeter of the site did not drop by more than 1m until the secant piling had been completed. Excavation was carried out by conventional earth moving plant, together with mechanical rock breakers

when required. The upper 0.5 m to 1.0 m of the bedrock was confirmed to be weathered and fractured, see Figure 4.

6.2 Dewatering

Lowering of the groundwater level within the site was achieved by pumping from sumps at various locations around the site at levels just below the required formation or foundation level. The sumps used were usually located in the excavation for a foundation or for drainage pipes. Typically four to six, 100mm dia. to 150mm dia., pumps were in use at any one time. Pumping was frequently intermittent due to the nature of the weathered rock. It was not possible to predict the level of groundwater seepage until an excavated face was exposed. Groundwater level was successfully maintained at a sufficiently suppressed level through the basement construction period (including the anchorage construction works) to allow all construction to be completed in the dry.

6.3 Groundwater monitoring

Several piezometers were located around the site. A summary of the piezometer readings, for instruments in the gravel and rock strata, is given on Figures 6a and 6b respectively. Basement excavation commenced in mid - July 1995 and it was largely completed by the end of October 1995.

Fairly soon after pumping began, the water level in the rock was drawn down between approximately 1 m in the Jervis St. / Upper Abbey St. area and about 4 m in the Mary St. / Liffey St. area (Piezometers M2, M4 and PZ102) and remained

approximately at these levels during the monitoring period. This relatively high drop in piezometric level did not have implications for adjacent structures as all are either piled or founded at a higher level on the gravels. Regular inspections of adjacent structures revealed no effects. A general comment would be that the readings are “spiky” in nature because of the intermittent pumping effort. In general the response of the piezometers in the bedrock was in hydraulic continuity with that in the gravels. The exception is that for piezometers M2, M4 and PZ102, all of which are located at the Mary St. / Liffey St. corner of the site.

With the exception of PZ102, which was located between the secant pile wall and an adjacent building, all of the readings for the gravels remained relatively steady with drawdown levels of between 0.5 m and 1.0 m.

7. GROUND ANCHORAGE CONSTRUCTION

7.1 Experience prior to contract

Prior to the main works, the same anchorage contractor had successfully installed twenty 13 m long anchorages (six of which were raking) for the support system to the Jervis St. hospital façade. Casing extended through the overburden only. The “open hole” method of drilling was used in the bedrock. Water was encountered in each hole. No difficulties were encountered and each anchorage was successfully load tested.

Using this experience the contractor based his price for the main works on the assumption of a one-hour drill time per hole and on use of the “open hole” drilling technique.

For the main works, two construction methods were used; “open hole” drilling and Odex drilling. Drilling involved holes for anchorage installation and for pre-grouting in order to attempt to improve the drilling conditions.

7.2 “Open hole” drilling

This method of construction was as follows:

- Drill 125 mm diameter borehole through preformed hole in slab to a depth of 9 m using a down the hole hammer with a button bit.
- Flush out hole, then remove drill string.
- Grout hole through a tremie pipe, placed at the bottom of the hole.
- Place anchorage.
- Top up grout via independent tremie tube placed to the bottom of the hole.

Drilling was carried out with conventional rotary percussive equipment. Air flush (provided by 750 cfm / 12 bar compressors) was used in all holes. Drill bits were not generally changed as the rock was not abrasive.

7.3 Odex drilling

Shortly after commencement of the main works, it became apparent that the ”open hole” method was not suitable as difficulties were encountered in many of the holes. Odex drilling was then employed. This technique involves the use of an eccentric reaming bit, which allows a casing to be pulled down following the drill bit. It is normally used in difficult or rapidly varying ground conditions. The method of construction was as follows:

- Drill hole as above with casing lengths of 4.1 m to 7.1 m depending on rock quality encountered.
- Flush out hole. Remove drill string leaving temporary casing in place.
- Grout hole.
- Place anchorage.
- Remove casing.
- Top up hole with grout.

Odex drilling was typically used in 50% of the anchorage locations in the area along Upper Abbey St. It was mostly used on the west side of the site near gridlines 9 to 12 (see Figure 7). It was only infrequently required on the east near gridlines 12 to 14. Odex drilling was also required in 70 % to 80% of the holes in the lower basement area to the north east of the site.

8. DIFFICULTIES ENCOUNTERED

8.1 Drilling time

Figure 7a shows those areas of the site where the drilling time exceeded 90 minutes, was between 60 minutes and 90 minutes and was less than 60 minutes. Drilling time exceeded 1 hour (i.e. that time assumed at tender) in 228 of the holes. Lowering the air pressure did not ease the drilling difficulties. Some holes were also found to be dry. The main problem areas were in the south-west and north–east of the site.

8.2 Hole instability

Hole instability" is defined as those occasions where the collapse of the hole prevented the installation of the anchorage. This occurred in 83 of the holes, mostly "open holes.

Figure 7b illustrates areas where the number of boreholes drilled, which were either cased or collapsed, lay between 0 and 20%, 20% and 40% or exceeded 40%.

8.3 Interconnection between holes

Interconnection between holes, either in the form of compressed air or grout, was observed in 135 holes. This is probably an underestimate as conditions in the basement did not permit easy recognition of interconnection. The holes in question were up to 20 m apart. It should be noted that the basement slab is founded on, either impermeable black boulder clay or bedrock, the gravels having been removed. This experience than indicates the broken nature of the bedrock.

8.4 Grout take

A neat cement grout with a 0.4 water / cement ratio was used in the works. It had a required 28 day grout strength of 40 N/mm^2 and this was always achieved. The theoretical grout take per holes was made up of 2 (50 kg) bags of cement. The contractor budgeted for a take of 4.5 bags per hole. Figure 7c shows where the volume of cement, during anchorage installation only, lay between 0 and 0.19 m^3, 0.19 m^3 and 0.38 m^3 or exceeded 0.38 m^3 (0.38 m^3 is equivalent to 10 bags of cement). Figure 7d show the areas of the site where the total grout take lay between 0 and 100%, 100% and 500% or exceeded 500% of he budgeted take of 4.5 bags.

In all 6800 bags of cement were used (340 t), compared to the theoretical quantity of 900 bags. Cement take exceeded 10 bags in 136 holes and exceeded 200 bags in several

holes. Again most problems seem to have occurred in the south-west and north-east parts of the site.

8.5 Voids

Voids were encountered in 4 holes as shown on Figure 7e. This included two occasions where the void was encountered in the first drillhole, indicating the voids were pre-existing in the bedrock. These findings were particularly surprising as the Dublin "calp" limestone is regarded as being too argillaceous to permit the development of solution features.

8.6 Rounded gravel

Rounded gravel size fragments of limestone were encountered in 22 holes, particularly in the lower basement area, see Figure 7f. This is also an unusual occurrence for the Dublin limestone where the fragments are usually angular and platey, and may suggest flowing water conditions exist in these zones.

8.7 Poor ground encountered when drilling

Figure 8 shows the areas of the site where very poor ground was encountered during the drilling process. This information was obtained from the driller's logs. The figure is sub-divided into those areas where the percentage of very poor ground exceeded 52%, lay between 39% and 52%, between 26% and 39% and was less than 26%.

8.8 Performance of the anchorages

Each of the 443 anchorages were stressed to 1.5 times their working load (i.e. 750 kN) as a proof test. All the anchorges were tested successfully.

8.9 Summary of difficulties encountered

A summary of the difficulties encountered is given on Table 3. It can be seen that a total of 317 holes gave some difficulty. A significant portion of the remaining 126 holes were put down in areas where pre-grouting had been carried out to improve drilling conditions.

9. RELATIONSHIP BETWEEN PROBLEMS AND BEDROCK GEOLOGY

The orientation of the zone where most of the problems were encountered in the Jervis St. site is south-west to north-east. This is best illustrated on Figures 7a, 7f and 8. It has been previously described how the known faulting in the central Dublin areas follows the same trend. It is also interesting to note that the five sites where known problems with construction in the bedrock have occurred are all located along the River Liffey and are within the pre-glacial buried channel where structural weakness in the bedrock is suspected to exist.

10. SUMMARY AND CONCLUSIONS

The main objective of this paper is to report on difficulties encountered during ground anchorage works in interbedded hard limestone, so engineers and geologists involved in

future projects may benefit for the experience. The principal findings can be summarized as follows:

1) Despite good ground conditions and successful experience elsewhere in Dublin, including work on the same site, significant difficulties were encountered during anchorage installation in hard interbedded Dublin limestone.
2) In all 72% of the anchorages encountered some difficulty. This is despite pressure grouting which was carried out in two areas to improve drilling conditions.
3) The pattern of zones within the site, where the difficulties were encountered, correlated well with the general bedrock conditions in Dublin.
4) It is concluded that the following factors contributed significantly to the problems:
    - Unexpectedly poor quality limestone was undoubtedly encountered.
    - Other unusual ground condition in the form of voids, rounded gravel and local ground water flows in the bedrock were also found.
    - There was a failure to understand the implications of the dewatering regime, in washing out fine material, from the weathered limestone zones.
    - Stress relief, due to basement excavation, made the horizontally interbedded layers of bedrock more susceptible to erosion.
5) The Jervis St case history detailed here, and the problems encountered on other sites summarized in Section 3, show that the process of construction

(particularly drilling technique) and the effects of groundwater need to be carefully considered when designing underground works in hard interbedded limestones. Design and construction issues cannot be separated and this calls for more collaboration between designers and contractors.

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Table 1: Ground Conditions from Site Investigations (from ground level +3.5 mOD.)

Stratum	Thickness (m)	Thickness (m)
	Range	Average
Made ground	2.6 - 3.7	3.2
Alluvium	0.0- 0.6	0.1
Gravels	1.7 - 3.0	2.2
Black boulder clay	0.0 - 1.6	0.8
Weathered rock	0.1 - 1.9	0.5
Intact rock	7.5 proven	
Groundwater	3.0 to 3.5	

Table 2: Summary of rock coring parameters

Parameter	Minimum	Maximum	Average
TCR (%)	46	100	95
RQD (%)	0	83	38
SCR (%)	0	100	59
FSI (m)	0.00	0.36	0.13

Table 3: Summary of difficulties encountered

Incident	No. of occurrences
Drilling time > 1 hour	228
Hole instability	83
Interconnection	135
Cement take > 10 bags	136
Voids	4
Rounded gravel in rock	22
Total number of holes with difficulties	317
Total number of holes	443

Summary of figures

Fig. No.	Description	Computer File
1	Location of sites	WORD/c:/papers/jervisfig1.doc
2	Geological cross section	WORD/c:/papers/jervisfig2.doc
3	Summary of rock coring	WORD/c:/papers/jervisfig3.doc
4	Photograph of weathered rock	WORD/c:/papers/jervisfig4.doc
5	Detail of basement slab / anchorage head	WORD/c:/papers/jervisfig5.doc
6a	Piezometer readings - gravels	WORD/c:/papers/jervisfig6a.doc
b	Piezometer readings - bedrock	WORD/c:/papers/jervisfig6b.doc
7a	Average time to drill holes	WORD/c:/papers/jervisfig7a.doc
b	Percentage of cased holes or equivalent depth of collapsed hole	WORD/c:/papers/jervisfig7b.doc
c	Average volume of cement per hole (anchorage installation only)	WORD/c:/papers/jervisfig7c.doc
d	Total grout take	WORD/c:/papers/jervisfig7d.doc
e	Location of voids	WORD/c:/papers/jervisfig7e.doc
f	Location of rounded cobbles or gravels	WORD/c:/papers/jervisfig7f.doc
8	Percentage of very poor ground encountered during drilling	WORD/c:/papers/jervisfig8.doc

Figures for Difficulties with ground anchorages in hard rock in Dublin, Ireland by Long and Murphy

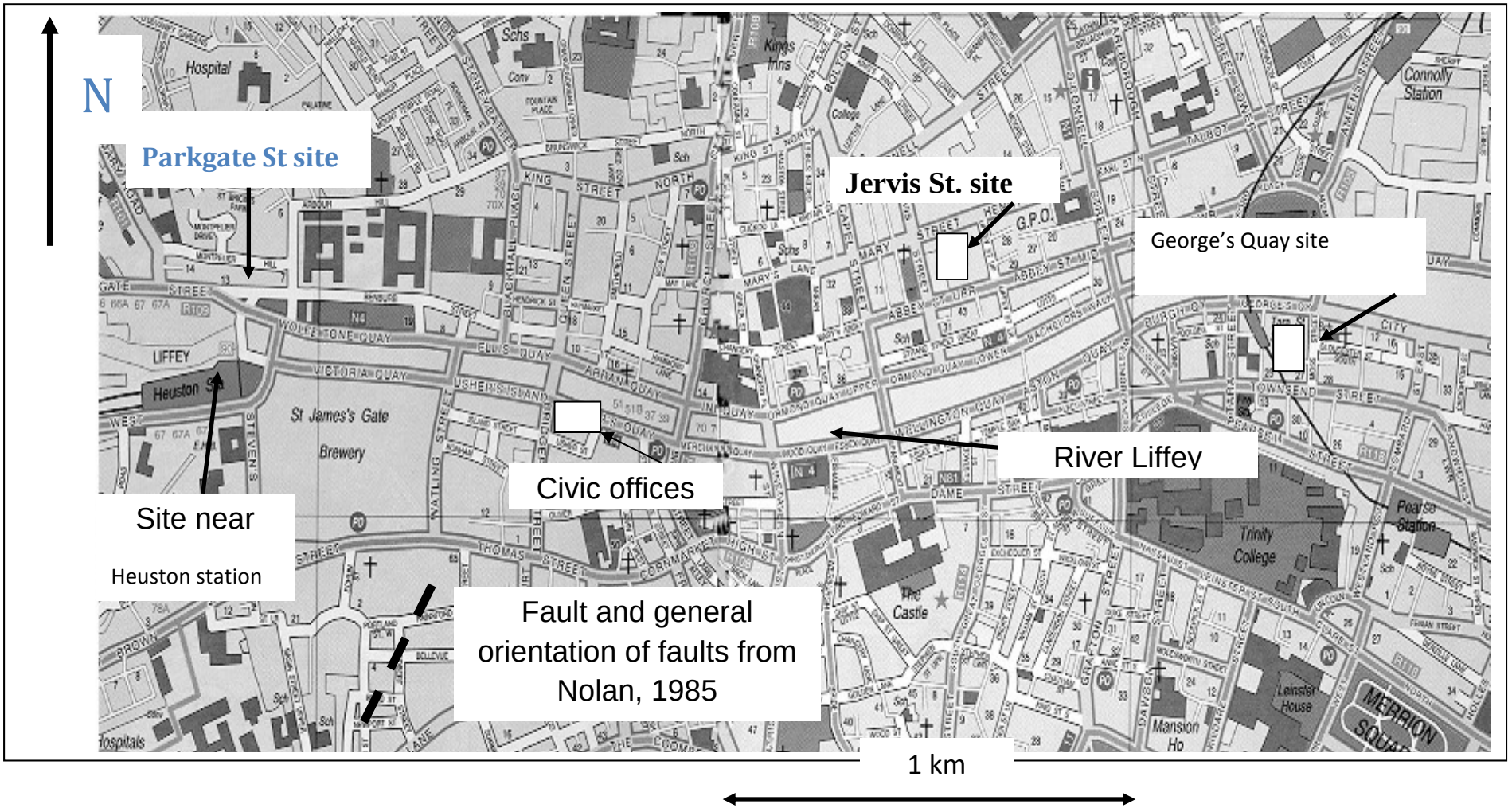

FIG 1. *Location of sites*

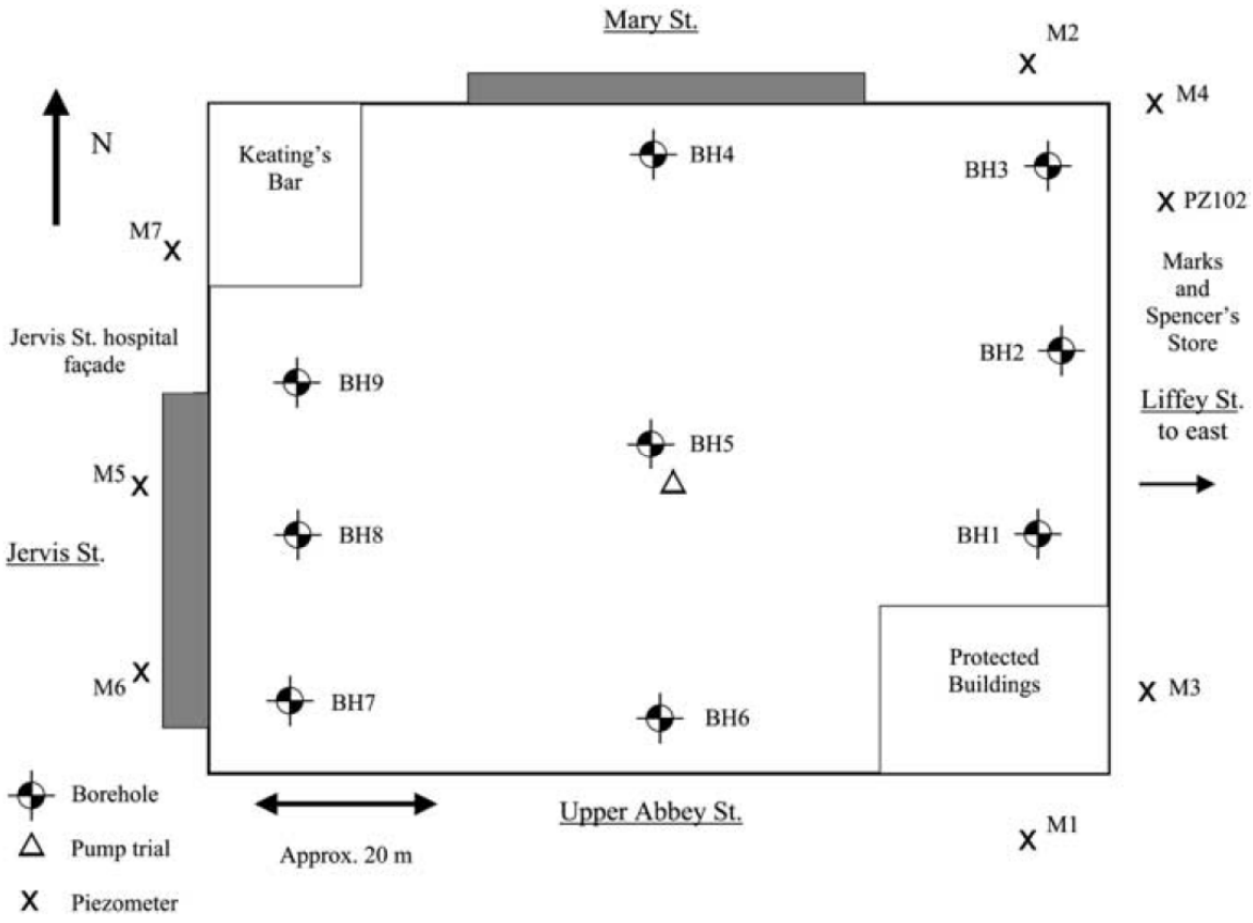

FIG 2. *Site Layout*

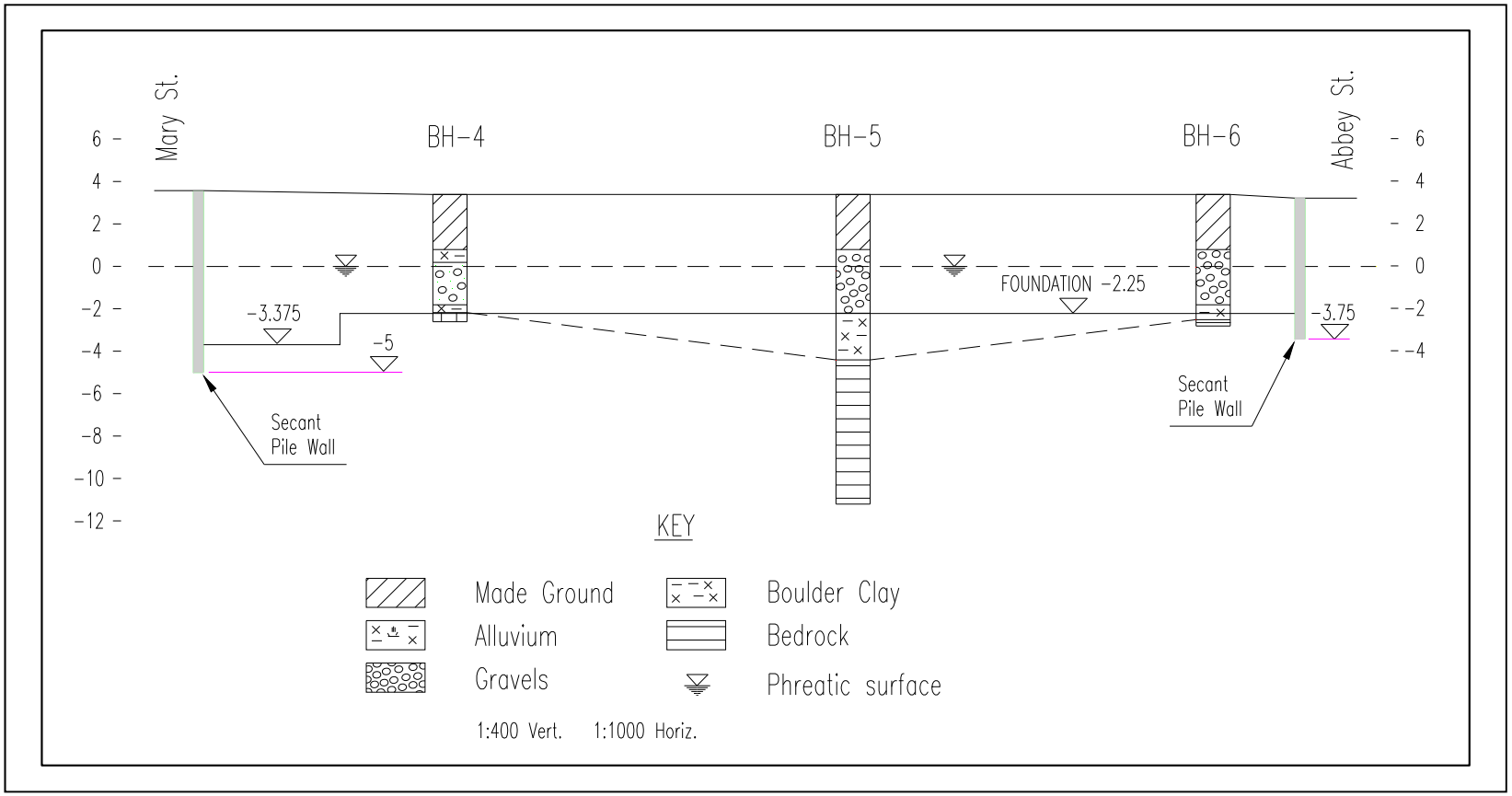

FIG. 3. *Geological cross section*

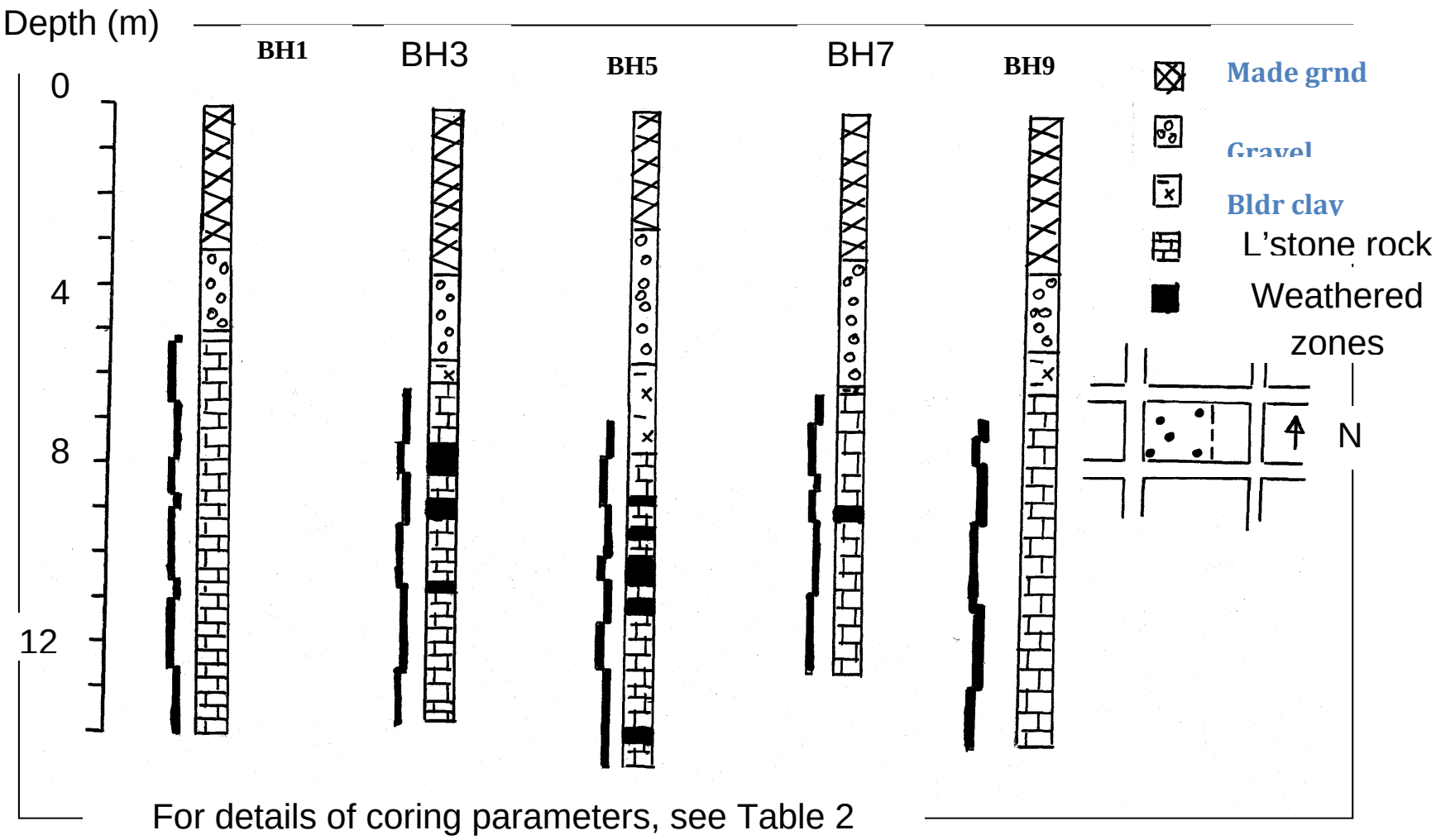

FIG 4. *Summary of rock coring*

FIG. 5. *Photograph of weathered rock*

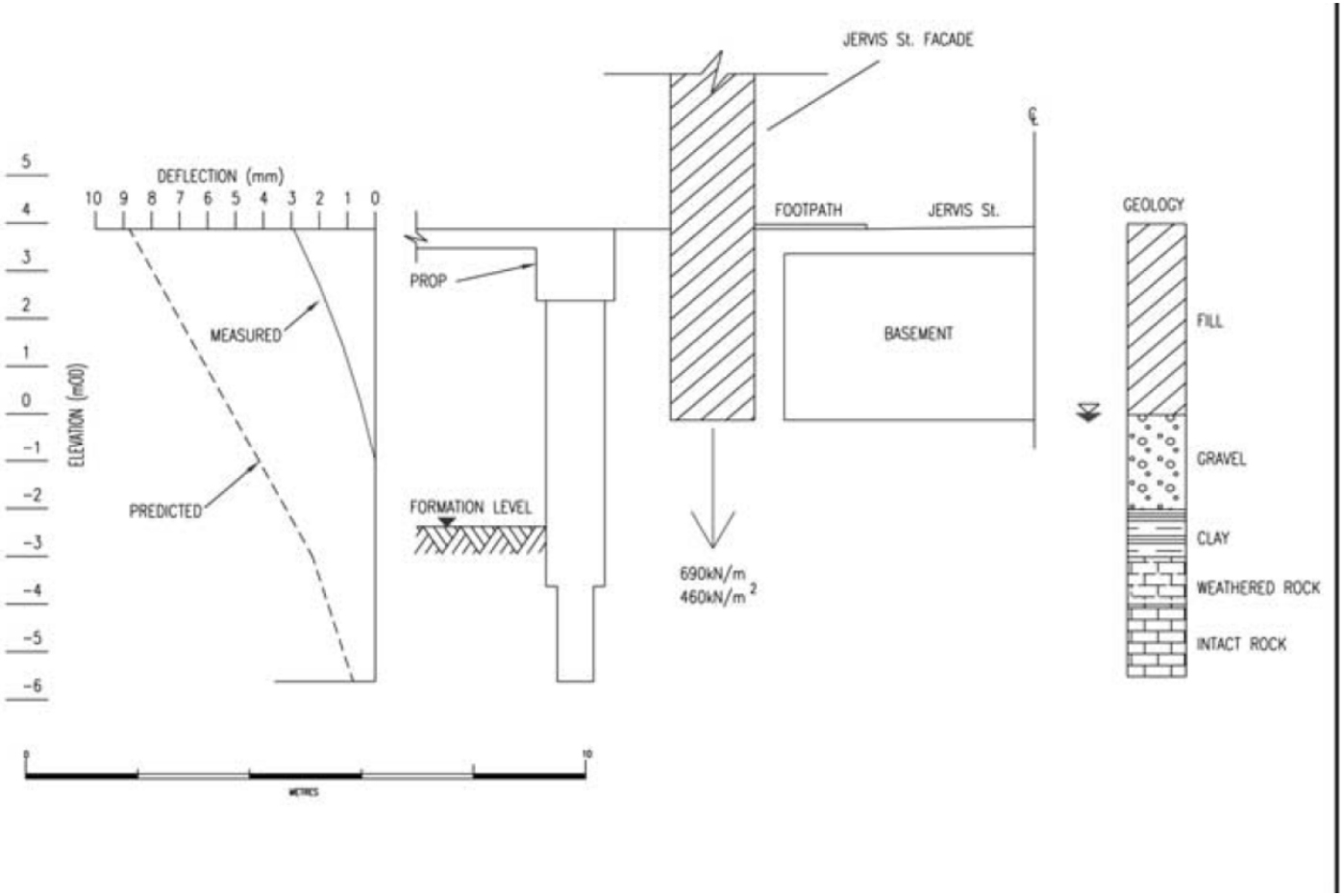

FIG 6. *Cross section through secant pile wall*

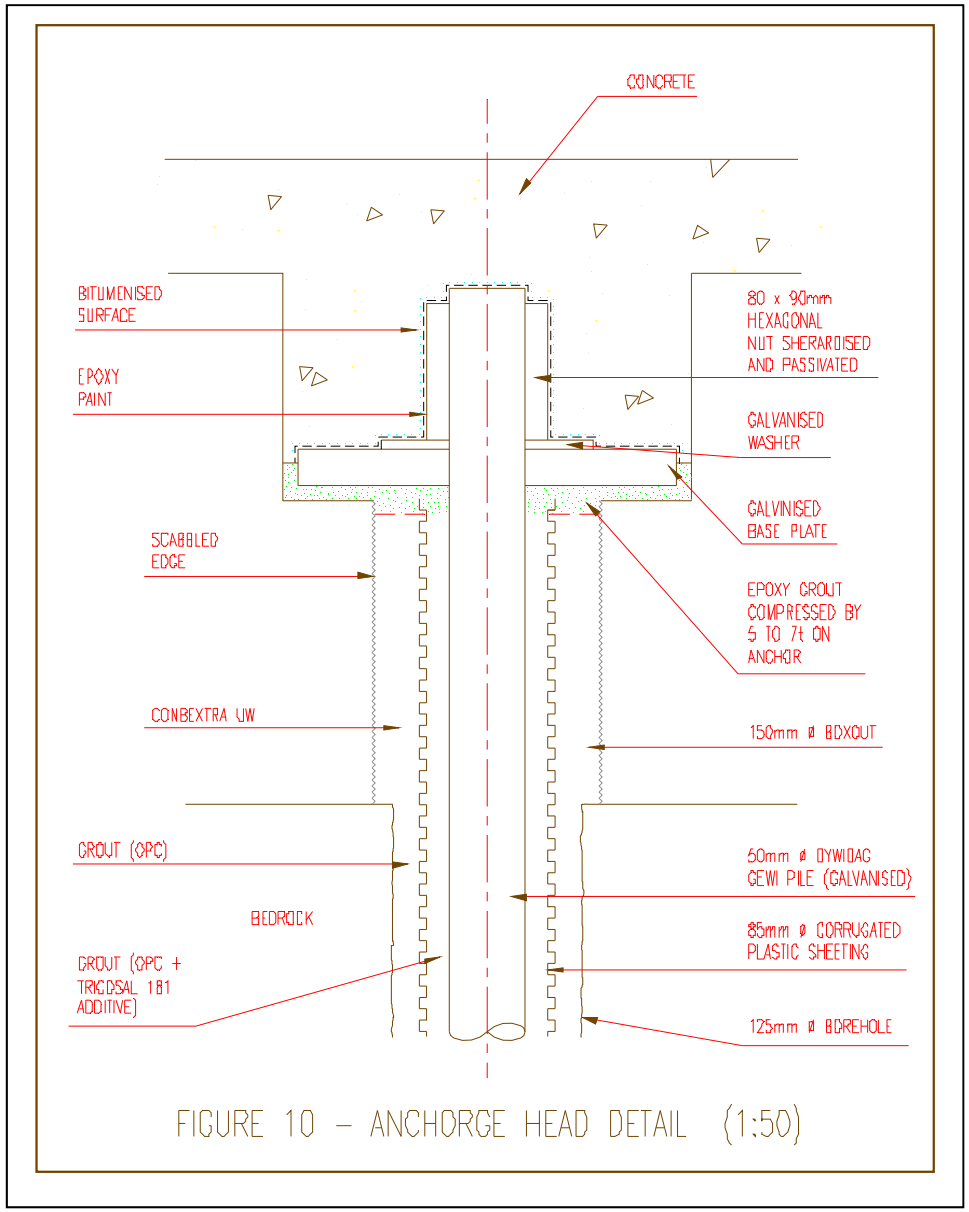

FIG 7. *Anchorage head detail*

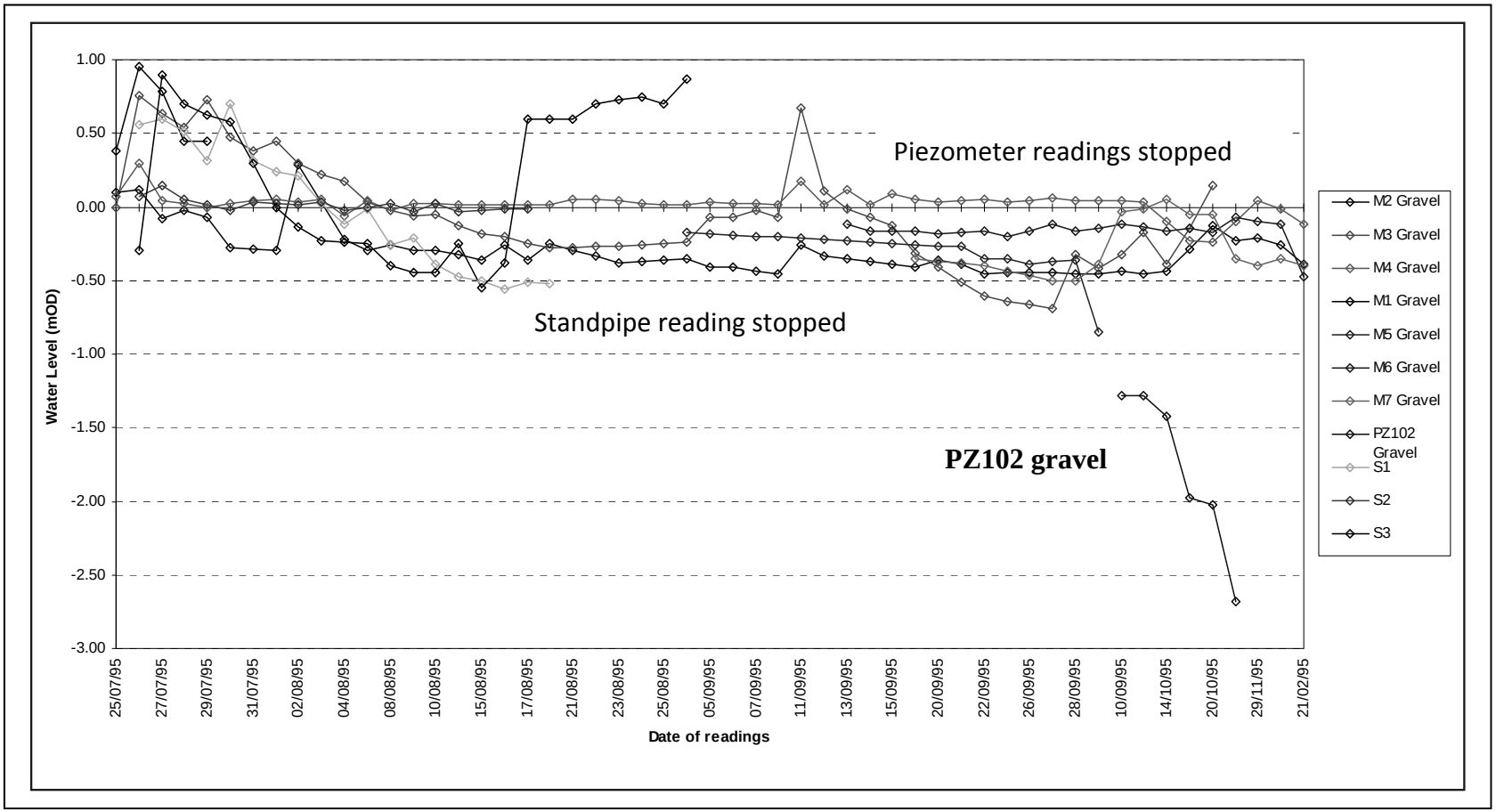

FIG 8a. *Piezometer readings in gravels*

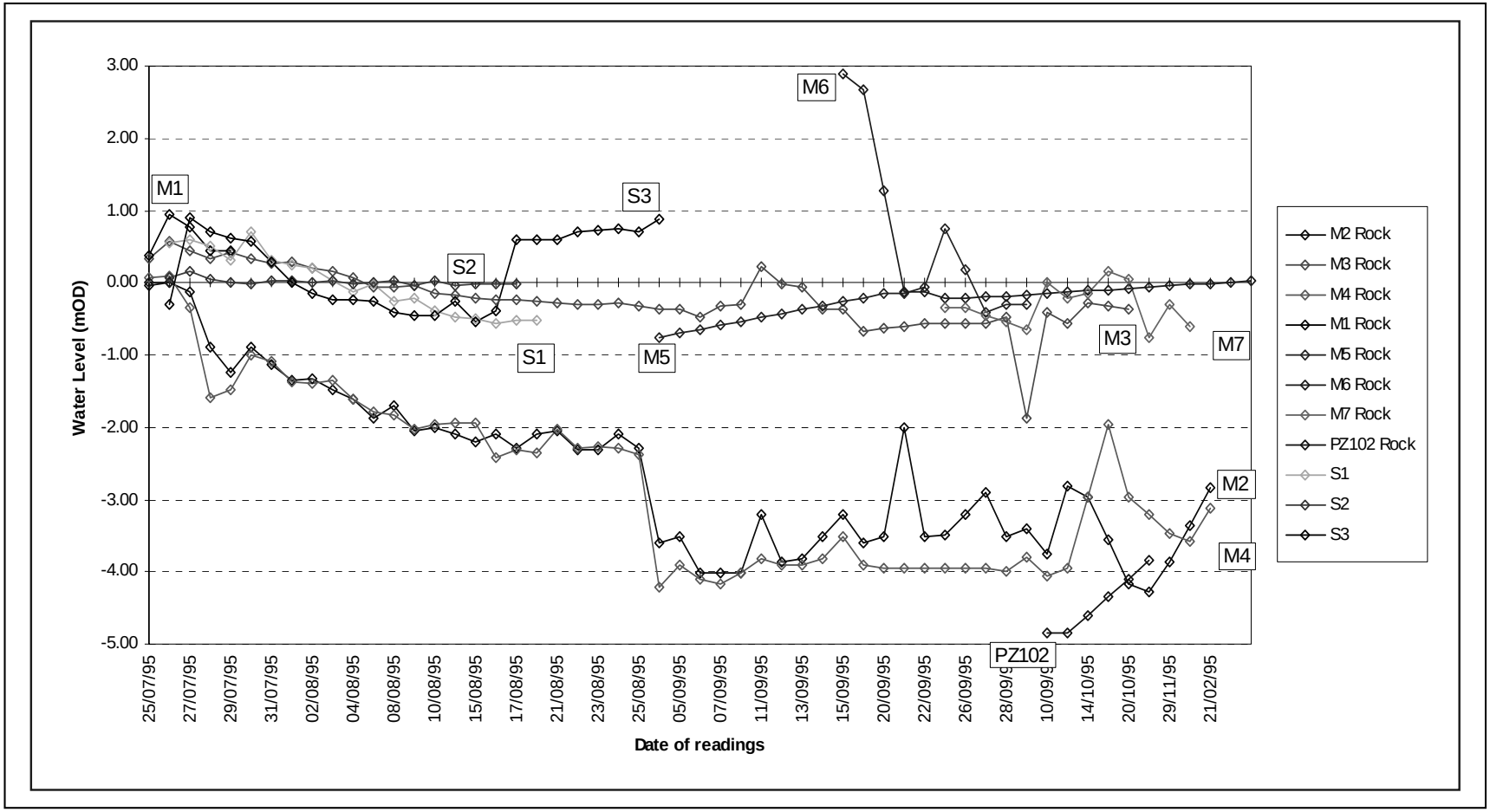

FIG 8b. *Piezometer readings in bedrock*

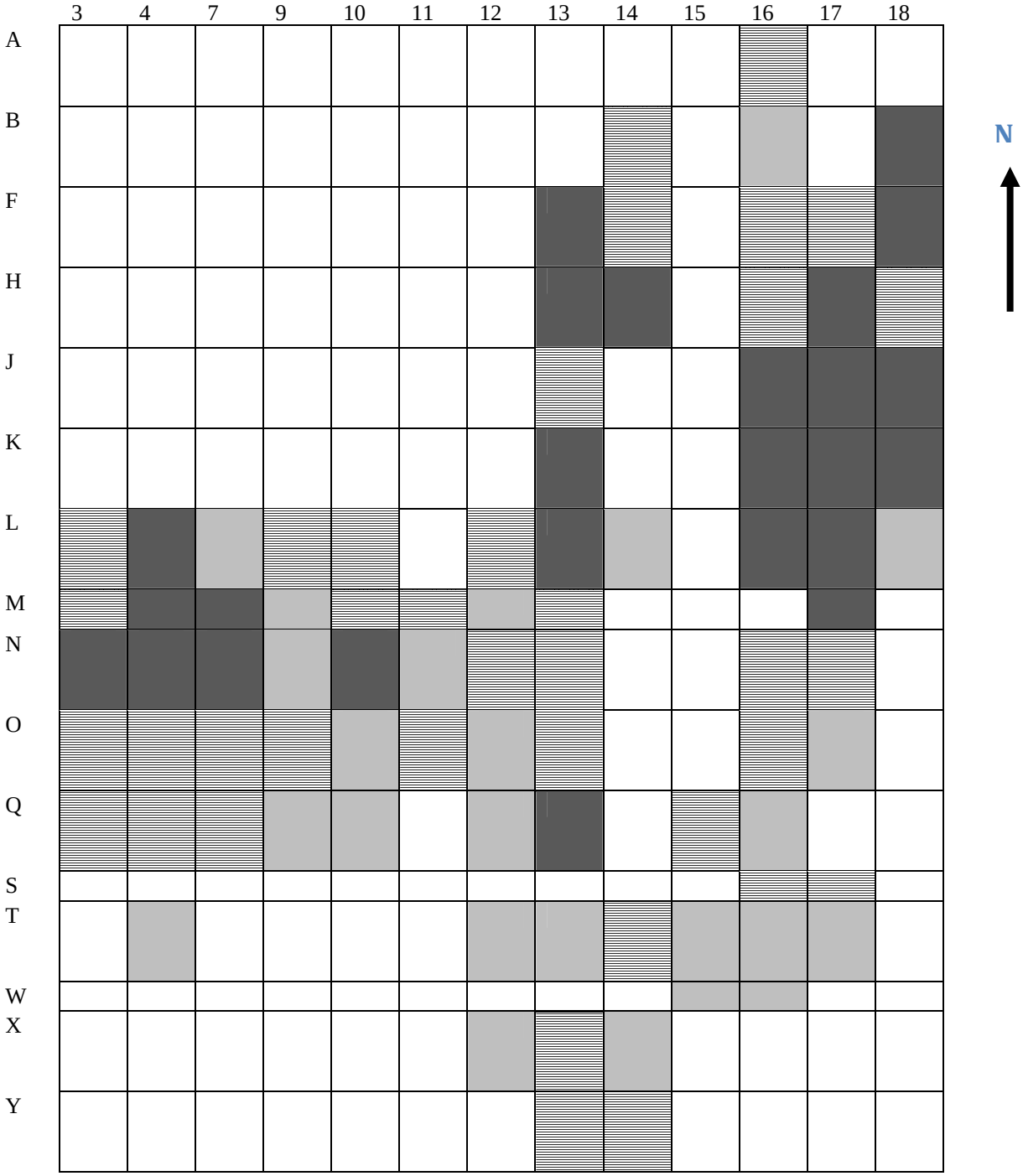

FIG 9a. Average time to drill hole

	90+ minutes
	60 – 90 minutes
	0 – 60 minutes

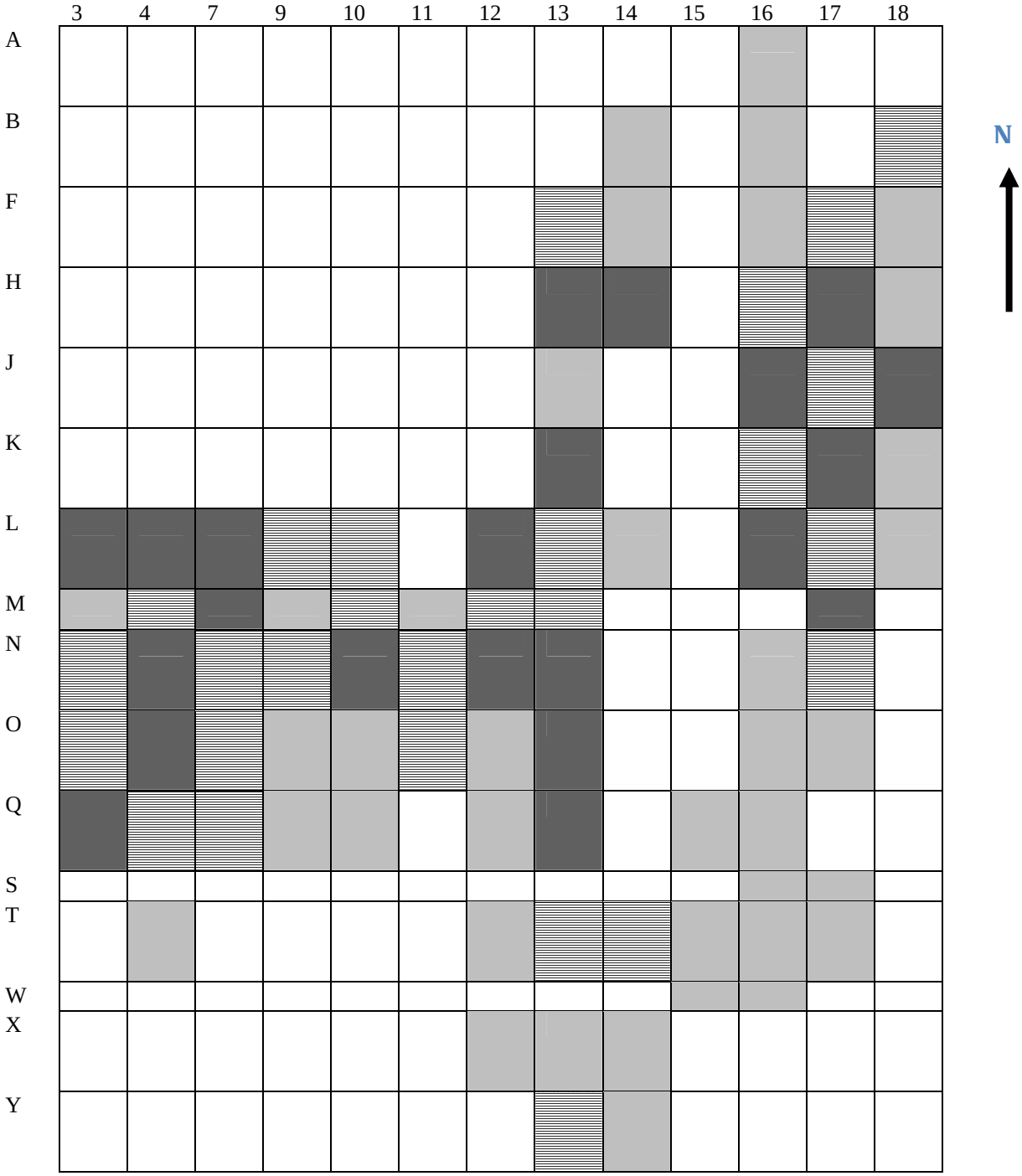

FIG 9b. Cased hole or equivalent depth of collapsed hole

	40%+ collapsed hole
	20% – 40%
	0 – 20%

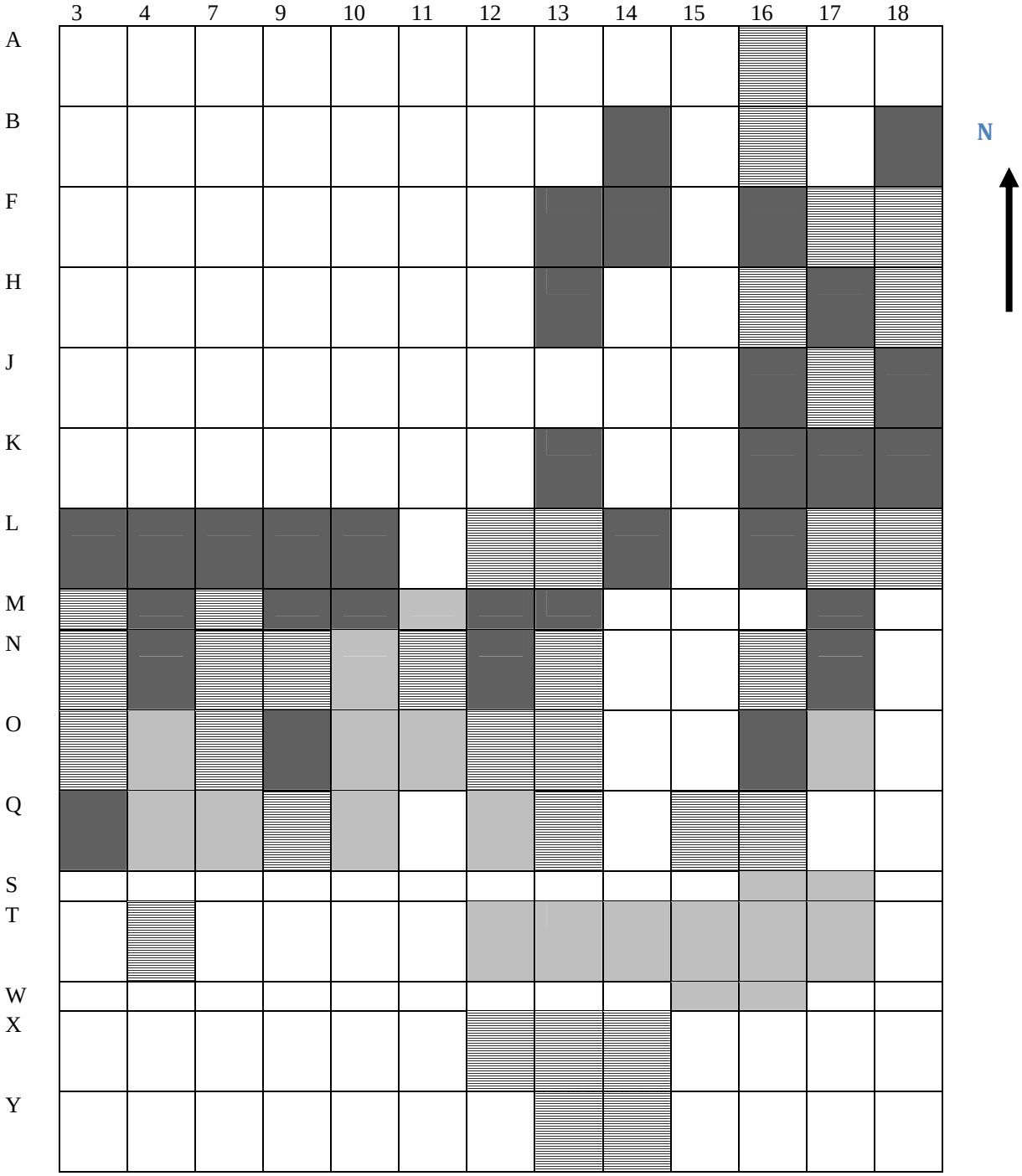

FIG 9c. Average volume of grout per hole (anchorage installation only)

	0.38 + m^3
	0.19 - 0.38 m^3
	0 – 0.19 m^3

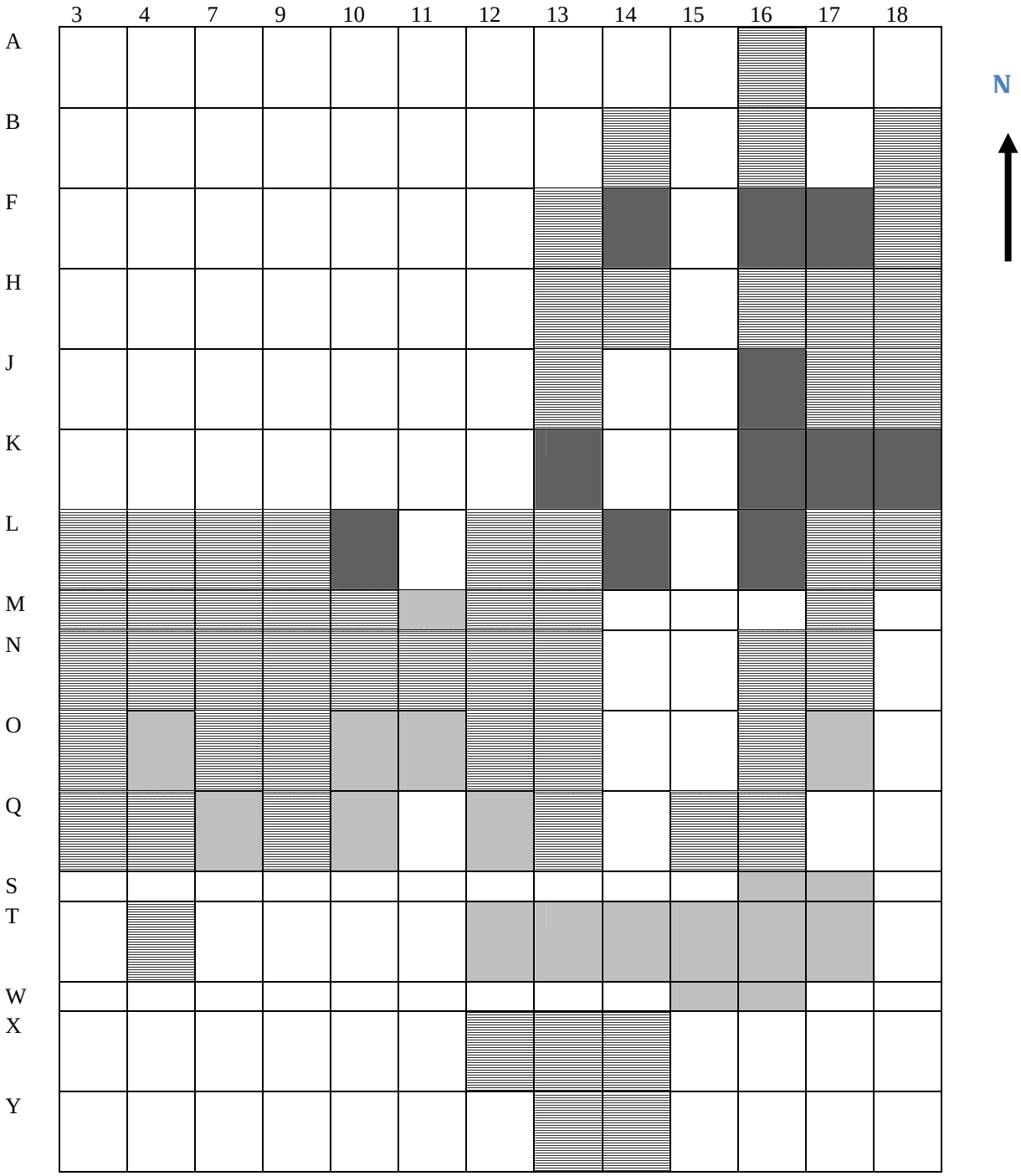

FIG 9d. Total grout takes

	> 500% budgeted take
	100 – 500% take
	0 – 100%

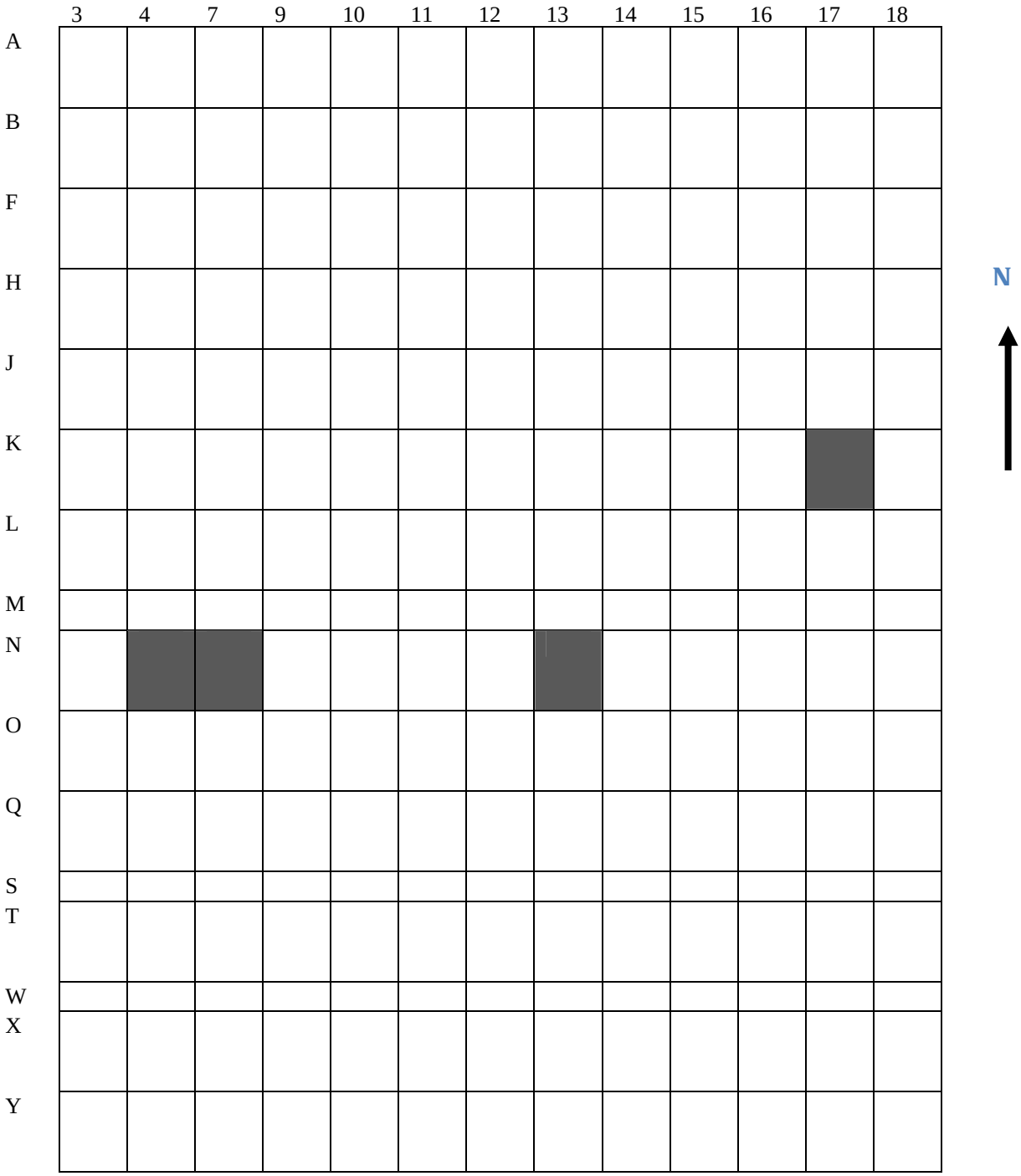

FIG 9e. Location of voids

Location of voids

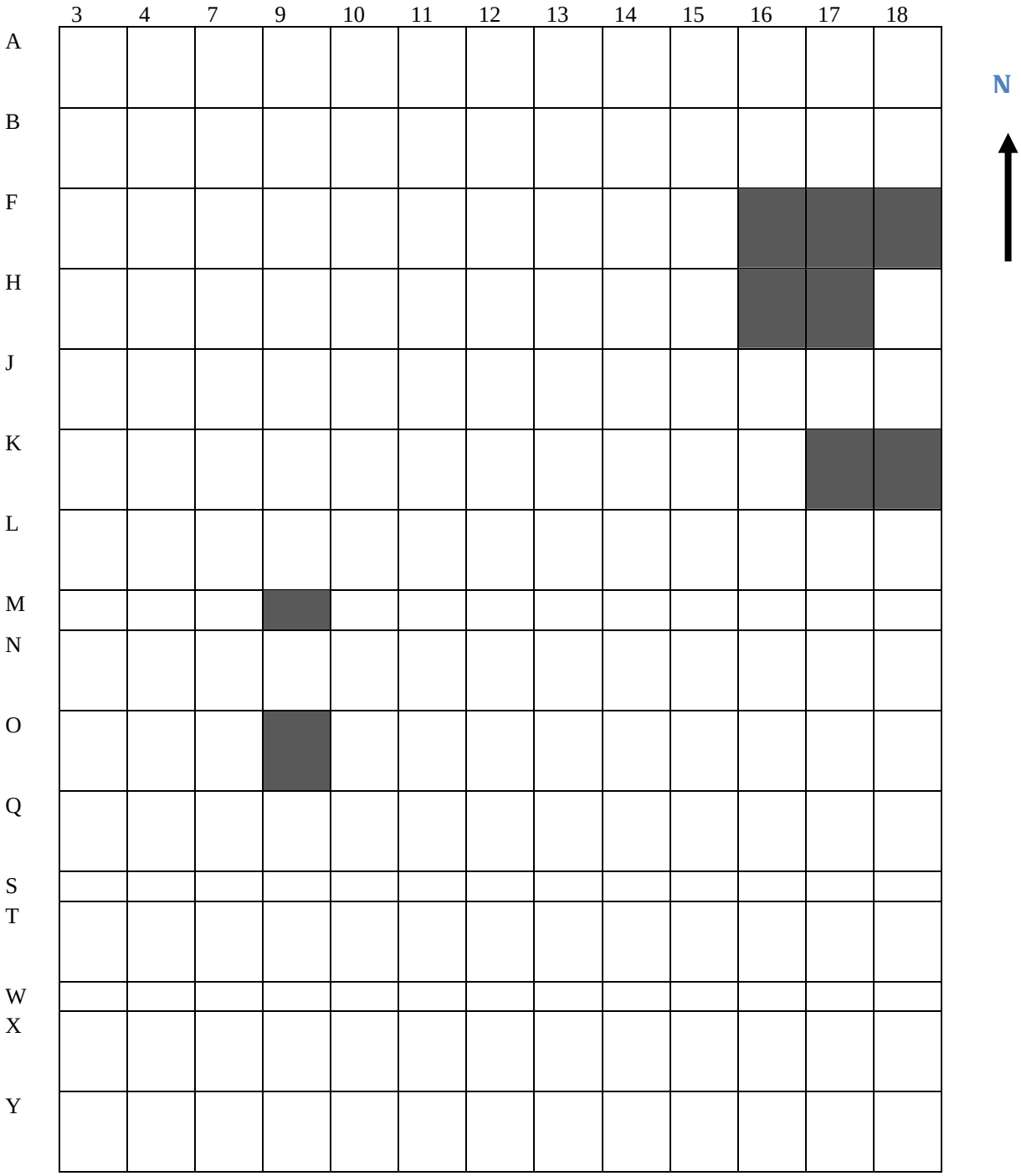

FIG 9f. Location of cobbles and gravels

	Location of cobbles and gravels

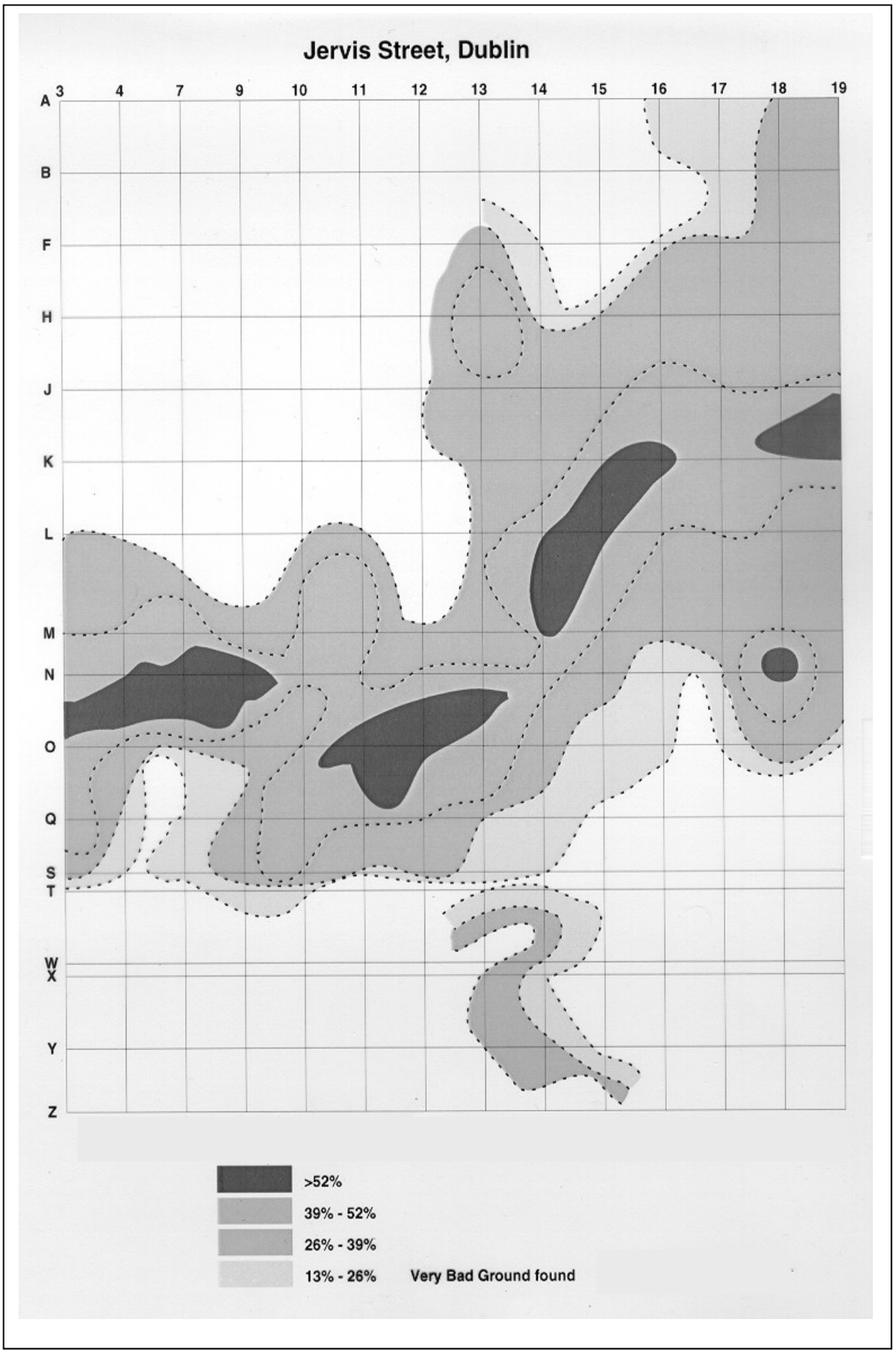

FIG 10. Percentage of very poor ground encountered during drilling