

Rock Mass Classification of Chalk Marl in the UK Channel Tunnels

by Nick Barton, Norwegian Geotechnical Institute, Oslo, Norway and
Colin Warren, Sir William Halcrow and Partners, London, United Kingdom

ABSTRACT

Although Chalk Marl is nearly at the weakest end of the strength spectrum for rock, its bedded and jointed nature make it quite amenable to classification by rock mass quality descriptors such as the NGI Q-system. Steeply dipping jointing and subhorizontal bedding was mapped and photographed in the partly flooded Beaumont (Abbots Cliff) and Terlingham Tunnels prior to analysis of core logs and core box photographs from the PB series of marine core drillings. Mean Q-values were 3, 4, 10, 6 and 12, 6 respectively. The Grey Chalk seen in the cliff exposures at Shakespeare indicated Q-values in the range 4 to 33. Jointing appears to have been similar in the slightly weaker underlying Chalk Marl, where permeabilities of about 1 to 20 Darcys in an otherwise very impermeable matrix also indicated the presence of extensive jointing. The jointed and bedded nature of Chalk Marl as experienced in the Beaumont, Terlingham and Channel Tunnels resulted in a lot of distinctly discontinuum as opposed to continuum behaviour. Overbreak was marked where joint sets, bedding joints and an unfavourable tunnel direction combined to give the necessary degrees of freedom for block release. The inevitability of block release problems was increased by the relatively smooth and planar character of the joints and by the destabilising effect of high pore pressures in the case of the sections of the Channel Tunnel having low cover and higher permeability. Trans Manche Link (TML)'s own rock mass Q-characterisation in the Marine Service Tunnel for km 20-30 was based on 250 face logs and 1, 120 side wall logs. Average Q-values were 9, 9 for km 20 to 24 where most difficulties with overbreak were experienced, and 33, 4 for km 24-30. Lower values were obtained when only face logs were analysed due to the absence of swarf. In the low cover zone between km 20, 5 to 21, 3, TML's mean Q-value was only 5, 6. The above range of mean values is similar to that obtained independently from pre-construction sources. According to Q-system case records, tunnels of 8, 4m span (Marine Running Tunnel, MRT) and 5, 3m span (Marine Service Tunnel, MST) need Q-values of 40 and 10 respectively for no support to be required. The 17 to 18m of unsupported tunnel lengths behind the MST and MRT tunnel boring machine tunnel faces made overbreak a very likely phenomenon when Q-values were in the range 1 to 10.

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1. INTRODUCTION

The Channel Tunnel was driven in Chalk Marl with the prior expectation of quite ideal tunnelling conditions in the UK side. This expectation was partly the result of little emphasis on the implications of joint structure. As a result of the difficulties and initial delays caused by overbreak in the UK sub-sea drive, the first author was requested to assess the rock quality in existing tunnels in Chalk Marl. The work was performed during 1990 and 1991 under contract to GeoEngineering who were conducting a major review for Eurotunnel. The assessment was made using the Q-system of rock mass classification (Barton *et al.*, 1974) which was also being used by Trans Manche Link (TML) in the Marine Service and Running Tunnels. The first author's classification of the Grey Chalk at Shakespeare Cliffs and of the Chalk Marl in the Beaumont and Terlingham Tunnels was performed prior to any data being provided on conditions in the Marine Service Tunnel (MST) or in the Marine Running Tunnel (MRT). The PB series of core logs and photographs for marine drill core PB1 to PB8 was also classified without prior knowledge of MST or MRT conditions.

Since rock mass classification is very much based on visual assessment and experience, it is judged to be helpful if the following Chalk and Chalk Marl classifications are illustrated by representative photographs. The starting point is logically the overlying Grey Chalk at Shakespeare Cliffs which is illustrated in Figure 1 (Plate 1).

2. Q-CLASSIFICATION OF GREY CHALK AT SHAKESPEARE CLIFFS

The strongly developed bedding and steeply dipping conjugate jointing are easily recognised at many locations along the cliffs. Figure 1 is a typical illustration of these features. Superficial (non-systematic) Q-system classification of the Grey Chalk exposed in the lower cliffs gives the following preliminary indications of potential rock mass quality, where Q is defined as:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$

1. Typical $Q = \frac{80-100}{9} \times \frac{2}{1} \times \frac{1}{1} = 18-22$ (good)

2. Range $Q = \frac{70-100}{6-12} \times \frac{1,5-2}{1-2} \times \frac{1}{1} = 4,4-33$ (fair to good)

These parameters describe RQD (rather high); J_n , number of joint sets (often three); J_r , roughness (smooth undulating); J_a , alteration (not visible); J_w , water inflow and SRF, stress/strength (assumed favourable). The degree to which the observed jointing was representative of jointing in the Chalk Marl was



Figure 1 Representative conjugate jointing and subhorizontal bedding in the grey chalk at Shakespeare Cliffs.

examined in more detail in relation to marine drill core and in underground exposures in the Beaumont and Terlingham Tunnels.

The very persistent and planar WNW-ESE trending dominant joints seen both in the cliffs and in the foreshore below the cliffs in the Chalk Marl had virtually no undulation nor small scale roughness. Measurements of amplitude/length (a/L) indicated very low values of joint roughness coefficient (JRC) (Barton and Choubey, 1977) for these dominant steeply dipping joints. Shear strength would be correspondingly low. The significance of JRC values as low as 1 to 2 (more or less non-dilatant surfaces) for the stability of blocks in the periphery of a tunnel can be readily demonstrated in distinct element models such as UDEC and UDEC-BB (Cundall, 1980; Barton *et al.*, 1986; Makurat *et al.*, 1990).

3. Q-CLASSIFICATION OF PB SERIES DRILL CORE

Potential tunnelling conditions in the Chalk Marl were assessed from Channel Tunnel marine drill core (PB series), and from direct classification of the Chalk Marl in the Beaumont and Terlingham Tunnels. In each case, the Q-parameters were logged in histogram format, to give a fair indication of the range of parameter values. An example of jointing in one of the PB1 to PB8 drill core is shown in Figure 2.

Several hundred core box photographs and corresponding core logs and "fracture logs" were studied, resulting in the extensive set of histograms shown in Figure 3. The six Q-parameters are shown on the left-hand side, with complementary estimates or measurements of joint frequency, spacing, joint roughness and joint wall strength. (See Barton *et al.*, 1992 for a fuller description of the geotechnical logging format). The jointing seen in Figure 2 was described as follows at the time:

PB7: well jointed zone at 15 to 18m in Chalk Marl. Joint surfaces are reportedly stickensided (*i.e.*, $J_r = 0.5-1.5$ depending on planarity), $J_n = 9$ (or more), RQD = 85% (logged).
The weighted mean sample for the six Q-parameters shown in Figure 3 was as follows:

$$\bar{Q} = \frac{89}{6} \times \frac{1.4}{1.1} \times \frac{0.8}{1.2} = 12.6$$

Note in particular the low estimates of joint roughness JRC at core scale: JRC = 1-2 for bedding joints, and JRC = 2-3 for steeply dipping joints, and the correspondingly small roughness amplitudes (a/L at 10 cm scale: a \approx 0.2-1.0mm).

4. Q-CLASSIFICATION IN THE TERLINGHAM AND BEAUMONT TUNNELS

Many hours were spent in Q-mapping and photography of the partly flooded Terlingham and Beaumont Tunnels. Areas of major overbreak and consistent breakage to subhorizontal bedding planes *via* steeply dipping joints were a common feature in both tunnels. Examples of minor overbreak are shown in Figure 4. In some locations, many cubic metres of collapsed roof debris had to be climbed over, and it could be reasonably claimed that the tunnel cross-section had moved upwards (and outwards) a metre or so. The subhorizontal bedding planes tended to form the new roof, and could be continuous for many tens of metres in places.

The Q-parameter histograms for the two tunnels are reproduced in Figure 5. The weighted mean samples for the six Q-parameters were as follows:

$$\text{Terlingham Tunnel} \quad \bar{Q} = \frac{90.0}{7.4} \times \frac{1.6}{1.1} \times \frac{0.9}{1.5} = 10.6$$

$$\text{typical range of } Q = 1.3 \text{ to } 50$$

$$\text{Beaumont Tunnel} \quad \bar{Q} = \frac{93.6}{4.8} \times \frac{1.4}{2.7} \times \frac{0.7}{2.1} = 3.4$$

$$\text{typical range of } Q = 0.2 \text{ to } 100$$

The mapped section of the Beaumont tunnel had an overburden increasing under the cliffs from about 50 to 120m. This resulted in some stress related failure in the haunches, which is reflected in the lower Q-values. In the case of the MST and MRT channel tunnels, the effective stress level caused by 20 to 60m of overburden and 30 to 50m of sea depth is lower than most of Beaumont. Considering the uniaxial strength of the Chalk Marl (3.5 to 11 MPa, mean 5.5 MPa) the ratio of uniaxial strength to maximum principal stress (σ_2/σ_1) lies, however, well within the Q-system data base range for σ_2/σ_1 of 1.0 to 60 (mean 8.8).

5. SYNTHESIS OF Q-PARAMETERS FROM PRECEDENT DATA

We have seen above that Q-system classification was performed at Shakespeare Cliffs, on PBI to PB8 cores (photos and logs) and in the Terlingham and Beaumont Tunnels. The individual histograms presented earlier have been combined in Figure 6. Numbers of observations are given from each mapping site so that a weighted mean can be obtained from the whole sample. The letters SH, BT, TT and PB refer respectively to Shakespeare Cliffs, Beaumont and Terlingham tunnels, and PB drill core.

Combining data from the four sites is considered important since the core logging may provide an overly optimistic picture of the joint frequency, as verti-

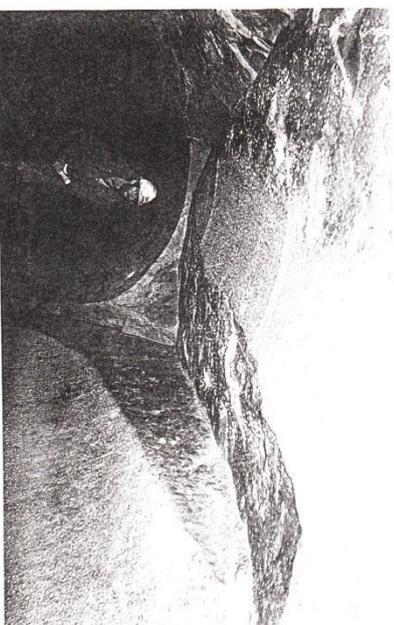
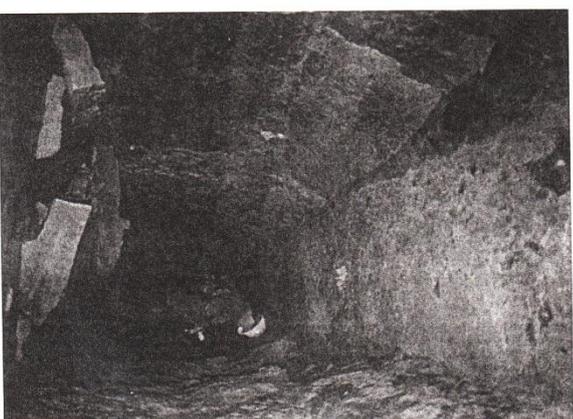
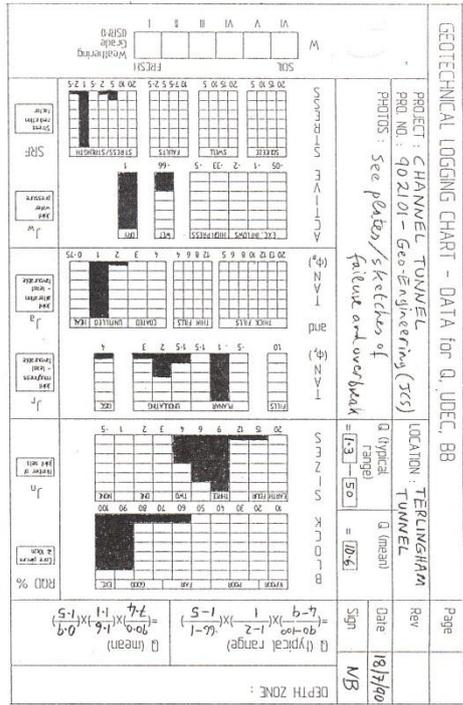


Figure 4 Joint and bedding plane controlled overbreak in a) Terlingham Tunnel, b) Beaumont Tunnel.

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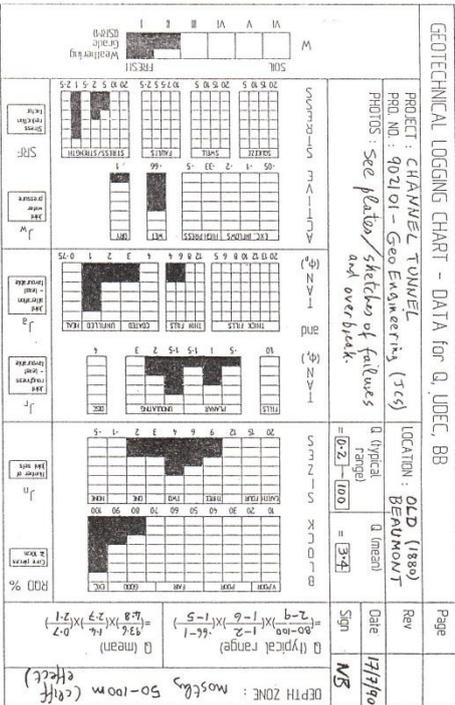
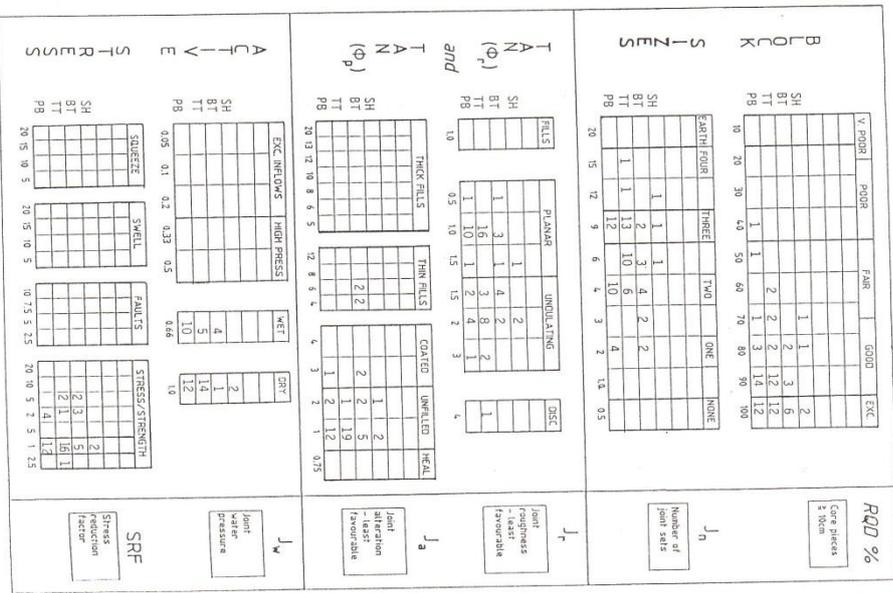


Figure 5 Q-classification of the Terlingham and Beaumont (Abbots Cliff) Tunnels.



cal holes were used. It is also valuable that the logging data could be tempered by the actual tunnelling experiences gleaned from inspection of the Beaumont and Terlingham tunnels. The addition of a very small number of observations in the Grey Chalk at Shakespeare Cliffs adds little to the data base. However, the experience of the jointing at the cliffs and along the foreshore is considered an important calibration process. With so little variation in mechanical strength, it is inconceivable that jointing observed in the Grey Chalk at the Shakespeare Cliffs does not also penetrate the Chalk Marl, as indeed observed in these precedent tunnels, and in the foreshore below the cliffs.

A synthesis of all the first author's Q-system observations from these sources resulted in the following range of properties which were expected to apply to the MST and MRT drives under the Channel:

Table 1 Synthesis of some pre-construction sources of rock quality for the Chalk Marl.

Parameter	(mean) Description	Sum./No. observations	Typical Range
RQD	= 90.1 excellent	6940/77	90-100
I_n	= 6.5 two joint sets plus random	473/73	4-9
I_r	= 1.5 rough, planar joints	93/63	1-2
I_a	= 1.5 slight alteration	79/53	1-2
I_w	= 0.9 slight water inflow	41.5/48	0.66-1.0
SRF	= 1.5 slight stress problems	73.5/48	1-2

The above rounded values (one decimal place accuracy) give a mean Q of 8.3. Rigorous multiplication and division of the whole (unrounded) sample gives a mean Q of 7.8. For practical purposes a round figure of Q = 8 can be adopted. The typical range of Q was 2 to 50 (poor to very good).

The above weighted mean of Q = 8 may not be the most typical or frequently occurring rock mass character. A glance at the histograms in Figure 6 indicates that the following are the most frequently occurring characteristics according to the classifications performed:

1. Most frequent $Q = \frac{100}{9} \times \frac{1}{1} \times \frac{1}{1} = 11.1$
2. Next most frequent $Q = \frac{90}{4} \times \frac{2}{2} \times \frac{0.66}{2} = 7.4$

A probable frequently occurring combination of the above two "classes", due to the likelihood of higher water inflows and slight joint alteration when three (as opposed to two) joint sets are present would be as follows:

3. Possible *problem ground* $Q = \frac{90}{9} \times \frac{1}{2} \times \frac{0.66}{1} = 3.3$

A recently updated version of the Q-system tunnel support diagrams, which is reproduced in Figure 7, indicates that Q-values of approximately 40, 10 and 1 are required for permanent unsupported spans (diameters) of 8.4, 5.3 and 2.1m (*i.e.*, the diameters for MRT, MST, Beaumont respectively). The fact that some 40 to 50% of the Beaumont tunnel is still standing with its final cross-section after 100 years is reasonable. The worst regions of overbreak and failure presumably have Q-values of between about 0.2 (Beaumont minimum) and 1.0. The "no support required" lower diagonal line in Figure 7 is slightly conservative (compared to mining practice) since it reflects civil engineering practice. (This diagonal line is unchanged since 1974, when the Q-system was first published.)

6. THE POSSIBILITY OF OVERBREAK PROBLEMS IN THE CHANNEL TUNNELS

The mean value of Q = 8 for the whole sample of Chalk Marl (as characterised by the first author prior to MST and MRT assessment) and the foregoing analysis of frequency of occurrence, leads to the following possible scenario for evaluating the potential for overbreak in poorer ground. We will assume the occurrence of the following (frequently observed) local conditions and their cumulative effect on the mean Q-value:

- 1 Three joint sets ($I_n = 9$)
 - 2 Smooth, planar joints ($I_r = 1.0$)
 - 3 Slightly altered joint walls ($I_a = 2.0$)
 - 4 Medium water inflow (>5 litres/min. locally) ($I_w = 0.66$)
- The successive, cumulative effects that these frequently observed conditions will have on the weighted mean Q-value of 8.0 are as follows:
- $$Q = 8.0 - 5.8 - 3.9 - 2.9 - 1.9$$

This progressive worsening moves both MST and MRT size tunnels well into the regions of the Q-support diagram that require support close to the tunnel face.

The majority of the original Q-system case records were, however, related to drill and blast tunnels and caverns. During the instant of excavation, blocks of rock that are inherently unstable due to unfavourable I_n , I_r and I_a values (perhaps combined with "external" factors like stress and water pressure) will tend to fall out in the excavation process and contribute to overbreak.

When equally unstable blocks are freed by a TBM cutter head, many of them will try to fall out on the shield or trailing fingers. However, a good percentage of them will probably remain in place since they were not disturbed as much by the TBM as by blasting which involves blast gas penetration and

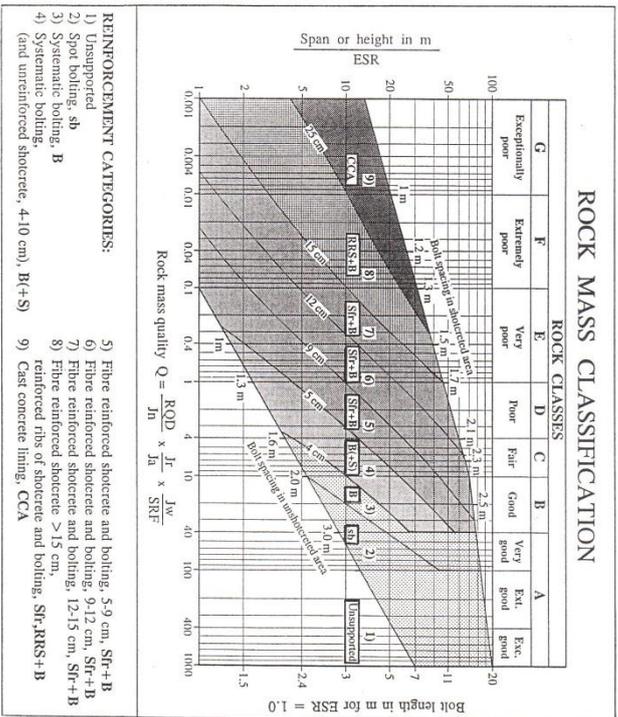


Figure 7 Q-support diagram showing the required spans for unsupported openings (lower diagonal line) and support measures when this span is exceeded. (Grinstad and Barron, 1993)

higher levels of vibration.

The lower frequency of vibration in a TBM excavation scenario may well lead to less overbreak, but overbreak that does occur is of course more problematic when trying to build pre-cast linings. The fact that the MST tunnel is "left unsupported" for 3.5 diameters (17m to first ring) and that the larger MRT tunnel is "left unsupported" for 2.0 diameters (18m to first ring) means that overbreak problems are likely when local conditions follow the worsening and likely scenario outlined above.

The MST and MRT have diameters approximately two to four times that of the Beaumont Tunnel. They are not subjected to such high rock stresses as at Beaumont, but they are in many locations subjected in many locations to several more serious factors such as more adverse joint orientations, weathering effects and high pore pressures in the subvertical joints connected to the seabed. The increased tunnel sizes induce a fundamental scale effect, where unchanged joint frequency causes greater problems with overbreak, the larger the tunnel diameter. In the Q-system, this scale effect is reflected in a greater need for immediate temporary support and final support, the larger the tunnel. The need for a certain level of support in the case of large tunnels even with "good" rock quality ($Q > 10$) is demonstrated in Figure 7.

7. PRELIMINARY REVIEW OF TML Q-LOGGING IN THE MST AND MRT

TML made extensive use of the Q-system in describing (under difficult mapping conditions) the rock mass conditions encountered. In the MST, approximately 250 face logs and approximately 1,120 sidewall logs between km 20 and 30 provide a wealth of information on TML's Q-system estimates, which, taken together with their careful descriptions of tunnelling conditions and joint characteristics, give a very useful data base from which to draw conclusions on the encountered conditions.

The conditions under which TML's observations were obviously made were not ideal, due to limited access to the rock. However, numerous and more extensive mapping results are available from cross-passages to supplement the necessarily sparser data sets from the face and sidewall logs. These have been carefully reviewed, and an assessment made of the validity of TML's estimates. Output from TML computer file print-out provided by Eurotunnel have been analysed in four different ways as follows:

- 1 distributions of Q for km 20-30
- 2 distributions of Q for km 20-24
- 3 distributions of Q for km 24-30
- 4 distributions of Q for km 20.5-21.3

In the first stage of analysis that follows, both face logs and side wall logs have been analysed. Due to the presence of swarf, the latter may be a less reliable source of data and side wall logs are subsequently excluded from our analysis in Section 8.

TML's average estimate of Q for these 10 kilometres in the MST was 22.9 (good), though most observations were in the "very good" class ($Q = 40-100$). Comparison of TML estimates for km 20 to 24 and km 24 to 30 in the MST reflect both the poorer quality of rock in the early kilometres and the more accurate description of conditions that was possible when overbreak due to jointing was frequent. Average Q-values were 9.9 (fair) and 33.4 (good) respectively. The most frequent rock class observed in km 20 to 24 was "fair" ($Q = 4$ to 10, mean 6.3, 270 observations). The most frequent rock class observed in km 24 to 30 was "very good" ($Q = 40$ to 100, mean = 50, 380 observations).

Table 2 shows the range of TML's observations in the MST for the poorer ground between 20 and 24 km.

Table 2 TML's Q estimates for km 20 to 24 from face and sidewall logging in the MST.

Range of Q	Description	No. of observations	Sum of actual Q-values	Mean Q-values
0.1-1	very poor	5	3.2	0.6
1-4	poor	191	470.4	2.5
4-10	fair	272	1,708.7	6.3
10-40	good	96	1,577.4	16.0
40-100	very good	47	2,280.0	49.0
	totals	611	6,039.7	9.9 (fair)

TML's Q-system classification of the low cover zone (km 20.5 to 21.3) in the MST gave a mean $Q = 5.6$ (fair) with most frequent observations in the "fair" ($Q = 4$ to 10) and "poor" ($Q = 1$ to 4) classes. The distribution of observed Q-values is shown in Table 3.

Table 3 TML's Q estimates for km 20.5 to 21.3 from face and side wall logging in the MST.

Range of Q	Description	No. of observations	Sum of actual Q-values	Mean Q-values
0.1-1	very poor	2	1.5	0.7
1-4	poor	42	90.6	2.2
4-10	fair	45	275.9	6.1
10-40	good	14	213.1	15.0
40-100	very good	0	0.0	0.0
	totals	103	581.1	5.6 (fair)

Maximum ranges and mean values of Q estimated by TML for the various zones mapped in the MST between km 20 and 30, fairly closely resemble the first author's independently derived predicted conditions obtained from core logging (PB series) and precedent experience (Beaumont and Terlingham Tunnels). The following list compares the two sets of data:

TML mapping during construction of the MST:	Q (range)	Q (mean)
km 20-30	Q (range) = 0.3 to 100	Q (mean) = 22.9
km 20-24	Q (range) = 0.3 to 40	Q (mean) = 9.9
km 20.5-21.3	Q (range) = 0.7 to 20	Q (mean) = 5.6

Author's estimates from pre-construction sources:

PB1 to PB8	Q (range) = 1.5 to 50	Q (mean) = 12.6
Terlingham Tunnel	Q (range) = 1.3 to 50	Q (mean) = 10.6
Beaumont Tunnel	Q (range) = 0.2 to 100	Q (mean) = 3.4

Comprehensive data packages were analysed by the first author at ten well documented chainages within km 20 to 30, in order to independently check TML's methods of Q-system application in the various qualities of rock. Some rather small but consistent errors in their application of the Q-system included a non-conservative use of $J_w = 1.0$ in many cases where significant water flow was observed and where $J_w = 0.66$ should have been used. In contrast, TML consistently used a conservative value of SRF = 2.5 in all cases, while only a limited number of the poorer, low cover tunnel sections perhaps qualify for this "low stress, near surface" characterisation.

In the poorest qualities of rock where TML's structural descriptions were quite comprehensive and accurate due to joint delineated overbreak, TML's estimates of Q were very similar to those of the author. At the ten well-documented sections between km 20 and 30, careful interpretation was made of TML's face and taluskin logs for the MST, and of their logs of cross-passages and vertical and sideways probes. The following results were obtained:

TML : range of mean $Q = 4.0$ to 28.9, overall mean $Q = 10.4$
 Author : range of mean $Q = 2.6$ to 17.6, overall mean $Q = 7.8$
 Details of this comparison are given in Table 4.

Table 4 Comparison of TML and the first author's Q-estimates at well documented MST and MRT channages between km 19.8 and km 27.2.

Channage (± 50m)	Author's estimates of Q		TML estimates of Q	
	Q (range)	Q (mean)	Q (range)	Q (mean)
1 19.824 km	7.5-50	17.6	2.4-80	28.9
2 19.925 km	0.9-25	7.5	1.7-40	11.4
3 20.651 km	1-100	7.8	1.6-40	11.7
4 21.026 km	1.7-50	9.4	4.4-13.2	7.4
5 22.151 km	1.2-17	5.7	2.4-19	7.0
6 22.526 km	1.2-17	5.6	3.0-6.7	4.9
7 22.901 km	0.5-20	2.6	1.8-10.7	4.9
8 23.276 km	0.5-25	3.7	1.1-8.2	4.0
9 23.651 km	1.5-33	5.7	2.5-13.3	7.9
10 27.025/167 km	9.9-133	12.4	3.0-80	16.2

In these well documented cases, an example of which is shown in Figure 8, there is no question about the poor quality of the ground, and the first author's estimates are in fact slightly more conservative than TML's estimates. Conditions encountered were well within the range predictable from pre-construction information if the necessary classifications had been performed.

8. DETAILED REVIEW OF TML FACE LOGGING RESULTS

In the foregoing section TML's Q-logging was analysed, using both the face logs and side wall logs. As indicated earlier, the latter might be expected to be affected by the swart. In this section we have therefore included only the results of TML's face logging. This is a significantly smaller data base as can be seen when comparing with the numbers in parentheses (from Tables 2 and 3).

Table 5 TML's MST face logging results, km 20 to 24

Range of Q	Description	No. of observations	Mean Q-values
0.1-1	very poor	0 (5)	- (0.6)
1-4	poor	18 (191)	2.2 (2.5)
4-10	fair	17 (272)	6.0 (6.3)
10-40	good	9 (96)	14.6 (16.0)
40-100	very good	5 (47)	40.0 (49.0)
	totals	49 (611)	9.7 (9.9)

(The numbers in parenthesis are from Table 2.)

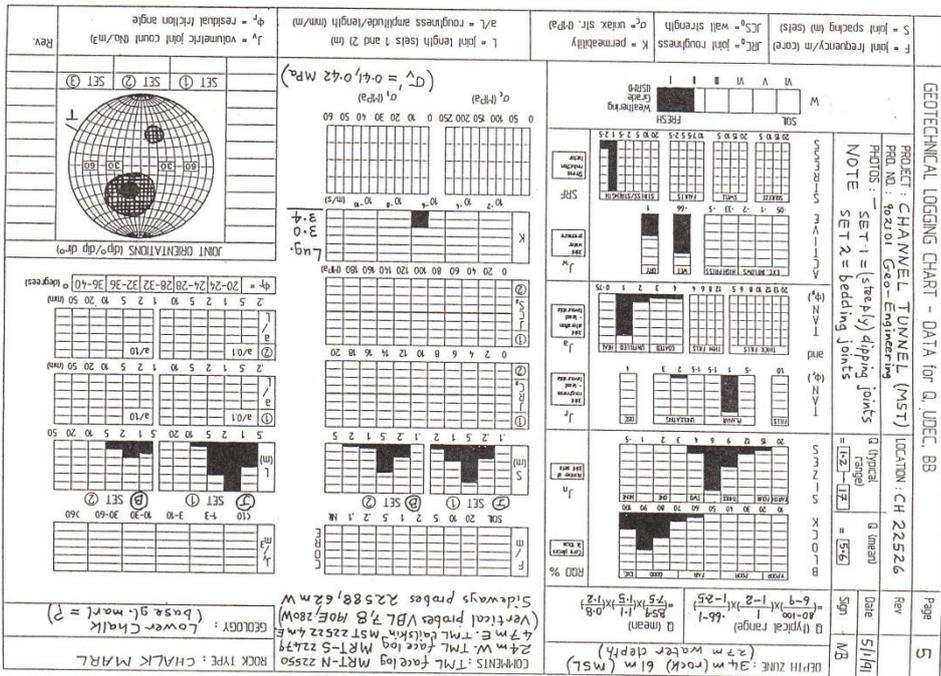


Figure 8 Example of Q and Geotechnical log at well documented section for MRT/MST at ch. 22.526m.

A similar comparison between face logging results and the full data set (from Table 3) is given for the low cover section (km 20.5 to 21.3) in Table 6.

Table 6 TML's MST face logging results, km 20.5 to 21.3

Range of Q	Description	No. of observations	Mean Q-values
0.1-1	very poor	0 (2)	- (0.7)
1-4	poor	8 (42)	2.0 (2.2)
4-10	fair	6 (45)	6.2 (6.1)
10-40	good	3 (14)	11.7 (15.0)
40-100	very good	0 (0)	- (-)
totals		17 (103)	5.2 (5.6)

(The numbers in parenthesis are from Table 3.)

If we again analyse just the face logging results, but include both the MST and two MRT tunnels, we obtain the following results:

Table 7 TML's face logging of all tunnels, km 20 to 24.

Range of Q	Description	No. of observations	Mean Q-values
0.1-1	very poor	0	-
1-4	poor	51	2.8
4-10	fair	52	6.2
10-40	good	24	15.5
40-100	very good	10	40.0
totals		137	9.0

Table 8 TML's face logging of all tunnels, km 20.5 to 21.3.

Range of Q	Description	No. of observations	Mean Q-values
0.1-1	very poor	0	-
1-4	poor	13	2.3
4-10	fair	22	6.0
10-40	good	12	13.4
40-100	very good	5	40.0
totals		50	10.5

Summarising the above analyses of potential differences between face logs and side wall logs, we can observe that TML's average estimate of Q for km 20-30 was 22.9 for all logs, but only 10.3 considering solely the face logs within the Chalk Marl. For MST km 20-24, TML's average Q-values were 9.9 (for all logs) and 9.7 (for face logs). In this case the difference is rather small, as also seen in Table 5. For MST km 24-30, TML's average Q-values were 33.4 (for all logs) but only 17.3 (for face logs).

The most frequent rock class observed for MST km 20-24 was "fair" (all 272 logs, mean Q = 6.3), and "poor" to "fair" (35 face logs, mean Q = 4.1). The most frequent rock class observed for MST km 24-30 was "very good" (all 380 logs, mean Q = 50), and "fair" to "good" (30 face logs, mean Q = 11.7), in this case a marked reduction. It is therefore seen that the face log interpretation generally gave a somewhat lower value of Q than the combined face and side wall logs. This is to be expected due to the problem of swarf smearing over joint traces.

An important point to be noted in the above analyses is that the MST entered Glauconite Marl and then Gault/6A material between chainage km 26.2 and 29.1 as it deviated to the north and below the level of the adjacent running tunnels past the UK Crossover. Therefore ideally this section of MST values should be ignored if considering solely Chalk Marl. This point has been taken into consideration when computing the face logs results given in Table 9. Here we compare MST, MRTN and MRTS face log results, giving mean Q-values (and numbers of logs in parenthesis).

Table 9 Comparison of mean Q-values obtained by TML from MST, MRTN and MRTS face logs only.

Scenario	MST Face	MRTN Face	MRTS Face	All
1. Q km 20-30	15.28 (179)	8.23 (63)	10.85 (82)	12.78 (324)
1A. above without km 26.2-29.1	11.38 (88)	same	same	10.34 (232)
2. Q km 20-24	10.06 (49)	6.95 (40)	9.77 (48)	9.05 (137)
3. Q km 24-30	17.25 (130)	10.44 (23)	12.37 (34)	15.52 (187)
3A. above without km 26.2-29.1	14.95 (34)	same	same	12.83 (91)
4. Q km 20.5-21.3	5.53 (16)	7.28 (21)	17.17 (13) *too high?	9.29 (50)

(Number of logs given in parentheses)

The poor conditions encountered between km 20 to 24, and in particular from the low cover section between km 20.5 to 21.3 indicate mean Q-values virtually identical to the weighted mean value Q = 8 obtained from the precedent

experience described earlier (PB1 to PB8 drill core, Terlingham and Beaumont tunnels).

9. USE OF PRECEDENT DATA IN PREDICTING TUNNELLING PROBLEMS

In Figure 6 the Q-system histograms for the combined observations of Terlingham and Beaumont Tunnels, of the PB series core and of Shakespeare Cliffs and foreshore, gave the following "most frequent" and "next most frequent" occurrences:

1. Most frequent $Q = \frac{100}{9} \times \frac{1}{1} \times \frac{1}{1} = 11.1$
2. Next most frequent $Q = \frac{90}{4} \times \frac{2}{2} \times \frac{0.66}{2} = 7.4$

From these two theoretical cases, it was reasonable to surmise that higher water inflows and slight joint alteration were more likely to be present with three joint sets ($J_n = 9$) than with two joint sets. A third class was therefore predicted as follows:

3. Possible *problem ground* $Q = \frac{90}{9} \times \frac{1}{2} \times \frac{0.66}{1} = 3.3$

Each of these six parameter values were frequently observed (*i.e.*, most frequently or next most frequently) and they combine to form a logical physical reality. If an SRF value of 2.5 had been used (shallow siting assumption made by TML), an even poorer quality ($Q = 1.7$) could have been reasonably predicted.

The same procedure of histogram analysis will now be followed for seven detailed structural data packages within the chaignage km 20-24. The most frequently and next most frequently estimated Q parameters for the seven relevant data packages within this chaignage were as follows:

1. Most frequent $Q = \frac{90}{5.8} \times \frac{1}{1} \times \frac{0.9}{1} = 13.2$
2. Next most frequent $Q = \frac{88}{6.6} \times \frac{1.4}{2.5} \times \frac{0.8}{2.3} = 2.6$

If we proceed as before and combine the most frequently and next most frequently observed parameters in the generally least favourable manner, we arrive at the third category:

3. Possible *problem ground* $Q = \frac{88}{6.6} \times \frac{1}{2.5} \times \frac{0.8}{1} = 4.3$

If TML's SRF value of 2.5 had been used as before, an even poorer quality ($Q = 1.8$) is obtained. This range of problem ground ($Q = 1.8-4.3$) is remarkably similar to that deduced earlier from pre-construction data ($Q = 1.7-3.3$) and suggests that poor ground conditions were predictable.

It has to be admitted that users of a rock mass classification system such as the Q-system will be on the lookout for joints and unfavourable features (in outcrops, tunnels and drill core) and may arrive at an overly pessimistic classification of the ground even when less jointed conditions are represented in their logging. The above analysis and comparison of pre-construction *predictable* conditions and post-construction *observable* conditions may therefore be a little unfair. If it turns out that one is comparing the generally poorer zones observed in the Terlingham and Beaumont Tunnels and in the PB core, with more average conditions in the chaignage interval km 20-24.

As a concession to this possible bias, the conditions encountered at the five poorest sections (of the ten data packages analysed) will now be reviewed. The five worst sections (of the ten analysed) were chosen based on TML's own Q data. The five worst sections received the lowest Q-values according to TML logging (refer to Table 4). The first author's mean Q-estimates at the same chaignages, based on the extensive documentation given by TML are given on the right hand side of TML's estimates:

Table 10 Comparison of TML's and the first author's estimates of mean Q-values at five well-documented sections in poorer ground.

Chaignage	TML	Author
km 21.026	= 7.4	9.4
km 22.151	= 7.0	5.7
km 22.526	= 4.9	5.6
km 22.901	= 4.9	2.6
km 23.276	= 4.0	3.7

A set of Q-system histograms for one of these five chaignages was given in Figure 8. An analysis of the frequency of Q-parameter observations for these five sections is given in Figure 9. From this we can derive data for the three categories "Most frequent", "next most frequent" and "possible problem ground" as before. The following results are obtained:

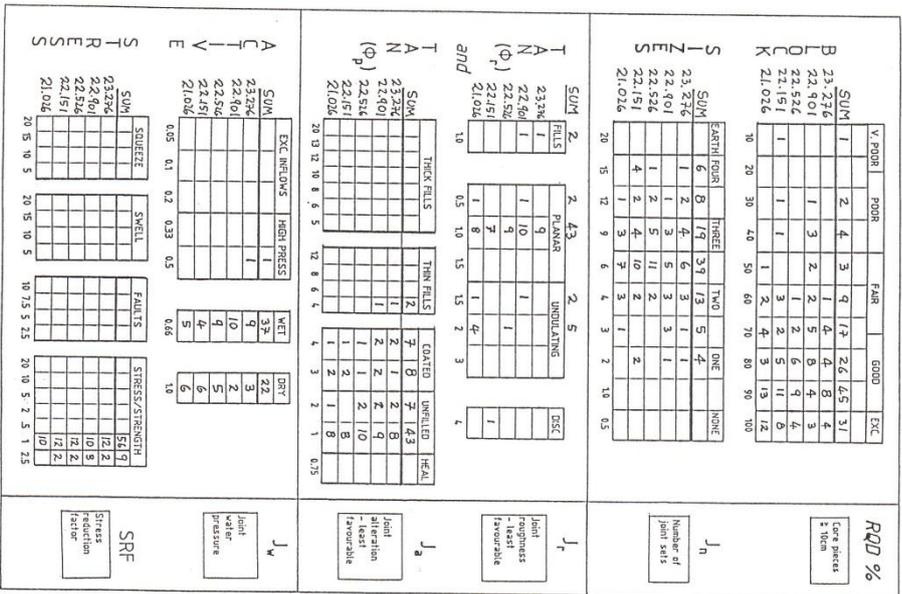


Figure 9 Analysis of five of the poorest chinnages.

1. Most frequent $Q = \frac{90}{6} \times \frac{1}{1} \times \frac{0.66}{1} = 10.0$
2. Next most frequent $Q = \frac{100}{9} \times \frac{2}{3} \times \frac{1}{2.5} = 3.0$

If we proceed as before and combine the most frequently and next most frequently observed parameters in the generally least favourable manner, we arrive at the third category:

$$3. \text{ Possible problem ground } Q = \frac{90}{9} \times \frac{1}{3} \times \frac{0.66}{1} = 2.2$$

If the above minimum value of SRF = 2.5 had also been used as before, an even poorer quality (Q = 0.9) is obtained.

Table 11 finally compares the predictable conditions with those in the 20-24 km sections, using the "most frequently" observed, the "next most frequently" observed and the "possible problem ground" categories described above.

Table 11 Comparison of predictable and encountered conditions based on Q-values calculated from the most frequently observed conditions.

Data from:	Most Frequent	Next most Frequent	Possible problem ground
Predicted Conditions	Q = 11.1	Q = 7.4	Q = 1.7-3.3
(Pre-construction data)	$\left(\frac{100}{9}\right) \times \left(\frac{1}{1}\right) \times \left(\frac{1}{1}\right)$	$\left(\frac{90}{4}\right) \times \left(\frac{2}{2}\right) \times \left(\frac{0.66}{2}\right)$	$\left(\frac{90}{9}\right) \times \left(\frac{1}{2}\right) \times \left(\frac{0.66}{1-2.5}\right)$
Encountered conditions	Q = 13.2	Q = 2.6	Q = 1.8-4.3
(typical: km 20-24)	$\left(\frac{90}{5.8}\right) \times \left(\frac{1}{1}\right) \times \left(\frac{0.9}{1}\right)$	$\left(\frac{88}{6.6}\right) \times \left(\frac{1.4}{2.5}\right) \times \left(\frac{0.8}{2.3}\right)$	$\left(\frac{88}{6.6}\right) \times \left(\frac{1}{2.5}\right) \times \left(\frac{0.8}{1-2.3}\right)$
Encountered conditions	Q = 10.0	Q = 3.0	Q = 0.9-2.2
(Poorest: km 20-24)	$\left(\frac{90}{6}\right) \times \left(\frac{1}{1}\right) \times \left(\frac{0.66}{1}\right)$	$\left(\frac{100}{9}\right) \times \left(\frac{2}{3}\right) \times \left(\frac{1}{2.5}\right)$	$\left(\frac{90}{9}\right) \times \left(\frac{1}{3}\right) \times \left(\frac{0.66}{1-2.5}\right)$

In view of the fact that the Q-system is based on logarithmic scales (minimum Q = 0.001, maximum Q = 1000) the closeness of the predicted and observed conditions is remarkable. With rock masses of such poor quality the "possible problem ground" (Q = 0.9-4.3) will inevitably have caused overbreak when excavated by TBM, and especially when left some 16 to 17m behind the face before support could be provided. The MRT has such a large span that the "next most frequent" Q range of 2.6 to 7.4 will also undoubtedly have led to overbreak.

A glance at Figure 7 indicates the level of NMT style permanent support (Barton and Grimstad, 1994) actually required in support classes 4 and 5 when Q -values are less than 10. For the case of a drill-and-blasted or road-header excavated tunnel, the necessary support would be systematic bolting and shotcrete, with steel fibre reinforcement in the poorest classes of rock. If stresses were higher, and the SRF factor became "mobilized" by unfavourable ratios of principal stress to uniaxial compression strength, then heavier support would of course be needed. A drained lining and satisfactory drainage measures are of course a pre-requisite for this NMT style of support.

10. UTILISATION OF SEISMIC MEASUREMENTS

Offshore geophysics carried out during several campaigns indicated P -wave velocities generally in the region of 2.0 to 2.6 km/s for the UK Chalk Marl. These low values reflect the low compressive strength and relatively high porosity of the Chalk Marl. Extensive laboratory testing of the Unit 2 Chalk Marl through which most of the UK tunnels were driven showed the following average values:

Table 12. Laboratory index test values for Unit 2 Chalk Marl

	Average	Min/Max
Uniaxial compressive strength	MPa 5.9 (252)	0.6/17.8
Young's modulus (vertical)	GPa 0.64 (37)	0.15/4.2
V_p (axial)	km/s 2.44 (152)	1.26/3.27
V_p (transvers)	km/s 2.62 (144)	1.37/3.58
Specific gravity	gm/cc 2.71 (72)	2.67/2.73
Dry unit weight	gm/cc 1.96 (289)	1.63/2.30
Moisture content	% 13.3 (288)	5.8/23.8
Porosity (calculated)	% 27.7 -	15.7/39.0

(No of samples in parentheses)

During the years since the Channel tunnel was completed, developments have been made in linking seismic velocity measurements with Q -value descriptions of rock mass quality. The objective has been to improve tunnel support prognoses based on refraction seismic measurements. This work was accelerated by direct calibration of core logging results with adjacent seismic tomography. This was obtained from crosshole seismic measurements. An initial calibration between Q and V_p was obtained for shallow, jointed, hard rock sites for which:

$$V_p = 3.5 + \log_{10} Q \quad (2)$$

proved to be a quite accurate method (Barton *et al.*, 1992)

Subsequently, seismic measurements and rock quality assessments from many sites around the world, including chalks, sandstones and other weak rocks, have been added to the data base, providing the opportunity to extend the correlation to weak porous rocks at variable depth.

Figure 10 shows the most recent version of these correlations. The central (thick) line gives the relationship between V_p and Q shown in Equation 2, for which $Q_e = Q$ (when uniaxial strength q_c approximates 100 MPa). The "normalising" of the Q -value by direct application of the uniaxial strength is a necessary modification for very weak rocks, and further correction is provided by the porosity and depth.

In order to illustrate the use of this seismic correlation chart, we can take the weighted mean value of $Q = 8$ from our precedent study (PBI to PB8 core, Terlingham and Beaumont tunnels). This value is very close to the TML mean of $Q = 9$ for km 20 to 24 obtained from all the face logs in the MST, MRTN and MRTS. (See Table 10.)

$$\bar{Q} = 8 \quad \bar{\sigma}_c = 6 \text{ MPa} \quad \bar{Q}_c = \frac{8 \times 6}{100} = 0.48$$

This Q_c value intersects the reference diagonal line (Equation 2) at $V_p = 3.2$ km/s. Correction for average porosity ($n = 27.7\%$, Table 12) results in a reduction of 1.6 km/s giving 1.6 km/s. Tunnel depths of, for example, 40m (see Figure 11) bring this value up to about 2.0 or 2.1 km/s. Even lower values appear likely in the shallow cover (20m) zone between km 20.5 and 21.3.

In those sections of the tunnel with markedly higher Q -values, *i.e.*, $\bar{Q} = 15$ (approximately) for the MST between km 24 and 30 (excluding 26.2-29.1) we have the following:

$$\bar{Q} = 15 \quad \bar{\sigma}_c = 6 \text{ MPa} \quad \bar{Q}_c = \frac{15 \times 6}{100} = 0.90$$

This Q_c -value intersects the reference diagonal line (Equation 2) at $V_p = 3.4$ km/s. Correction for average porosity ($n = 27.7\%$) results in a reduction of 1.4 km/s, giving 2.0 km/s. Tunnel depths of up to 40m (approximately) bring this value up to about 2.5 km/s.

An uncertainty in the above correlations which potentially show excellent agreement with the offshore geophysics (typically 2.0 to 2.6 km/s) is the effect of water depth and effective stress. Where permeability is very low due to less interconnected structure, the water could perhaps be considered as an additional load thereby potentially increasing the velocity and modulus of deformation. In permeable sections with a lot of structure, the water pressure giving significantly reduced effective stress would presumably have resulted in velocities of between 1.5 and 2.0 km/s according to the trends exhibited in Figure 10. Due to the changed tunnel location in relation to earlier offshore boreholes and seismic lines, it is not clear whether these potentially lower velocity values have been registered.

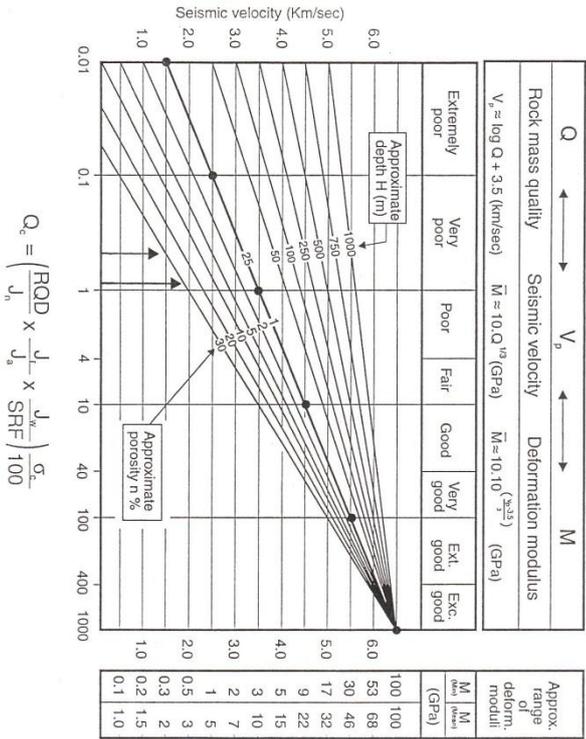


Figure 10 Seismic correlations chart for interrelating Q, V_p and M. (Barton, 1995)

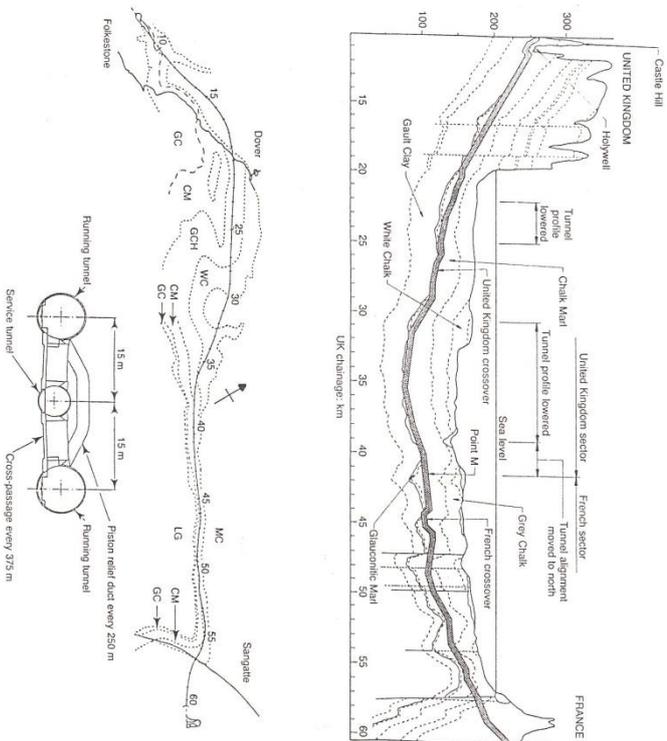


Figure 11 Channel tunnel stratigraphy, depths and dimensions (Fugeman et al., 1992; Varley et al., 1992)

The third parameter illustrated in Figure 9 - deformation modulus M_v is subject to considerable uncertainty due to stress effects, and disturbance effects when measured *in situ*, and due to the EDZ or excavation disturbed zone effect of reduced modulus immediately surrounding the tunnel. An undisturbed, fully *confined* modulus prediction of 3 to 5 GPa (M_1 mean) and a fully *disturbed* modulus prediction of 0.5 to 1 GPa may well be in line with the assumed near-tunnel values of about 0.8 to 1.4 GPa (Eves and Curtis, 1992) that were derived by back-analysis of deformation measurements.

11. CONCLUSION

In a drill and blasted tunnel, overbreak occurs as part of the excavation cycle and support can be applied right to the face if need be, which was not feasible with the chosen TBM method. Unsupported tunnel lengths of 17 and 18m for the MST and MRT represent approximately 3.5 and 2 diameters of unsupported rock in a rock mass with an average predicted rock mass quality Q of about 8, but with a quality range of at least 1 to 50. Tunnels of 8.4 and 5.3m span require rock mass qualities of 40 and 10 (respectively) for no support to be required according to case record analysis in the Q -system. Since rock mass quality between km 20-24 was generally below 10 and much below 40, problems with stability (overbreak) were predictable and inevitable. Sensitivity studies using the most frequently observed Q -system parameters indicate that several combinations of events (three joint sets, smooth joints, high water pressure, locally weathered joints) would in fact lead to overbreak and need for immediate support. Overbreak onto the trailing fingers was therefore inevitable in many locations in this chertage. Recently developed correlations between seismic velocity and Q -value using porosity, depth and uniaxial compressive strength appear to be promising ways of improving prognoses of rock quality and tunnelling problems in future projects.

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REFERENCES

- Barton, N., R. Lien and J. Lunde, 1974, "Engineering classification of rock masses for the design of tunnel support", *Rock Mechanics*, Vol. 6, No. 4, pp. 189-236.
- Barton, N. and V. Choubey, 1977, "The shear strength of rock joints in theory and practice", *Rock Mechanics*, Springer, Vienna, No. 1/2, pp. 1-54. Also NGI-Publ. 119, 1978.
- Barton, N., R. Lien, F. Løser, T. Løken, E. Grimstad, H. Hansteen, L. Hårvik and M. Christiansson, 1986, "Methods for Selecting Support in Sub-Sea Rock Tunnels", Proc. of Int. Symp. "Strat. Crossings", Stavanger, Norway.
- Barton, N., E. Grimstad, G. Aas, O.A. Opsahl, A. Bakken, L. Pedersen and E.D. Johansen, 1992, "Norwegian Method of Tunneling", WT Focus on Norway, World Tunneling, June/August 1992.
- Barton, N., F. Løser, A. Smallwood, G. Vik, C. Rawlings, P. Chryssanthakis, H. Hansteen and T. Ireland, 1992, "Geotechnical Core Characterisation for the UK Radioactive Waste Repository Design". 1992 Proc. of ISRM Symp. EUROCK, Chester, UK.
- Barton, N. and E. Grimstad, 1994, "Rock mass conditions dictate choice between NMT and NATM", *Tunnels & Tunneling*, October 1994, pp. 39-42.
- Cundall, P., 1980, "A generalized distinct element program for modelling jointed rock." Report PCAR-1-80, Contract DAMA37-79-C-0548, European Research Office, US Army. Peter Cundall Associates.
- Eves, R.C.W. and D.J. Curtis, 1992, "Tunnel lining design and procurement", *Proceedings of the Institution of Civil Engineers, The Channel Tunnel, Part 1: Tunnels*, pp 127-143.
- Fugeman, I.C.D., J. Hawley and A.G. Meyers, 1992, "Major underground structures", *Proceedings of the Institution of Civil Engineers, The Channel Tunnel, Part 1: Tunnels*, pp 87-102.
- Grimstad, E. and N. Barton, 1993, "Updating of the Q -System for NMT", Proceedings of the International Symposium on Sprayed Concrete - Modern Use of Wet Mix Sprayed Concrete for Underground Support, Fagernes, 1993, (Eds Konpen, Opsahl and Berg. Norwegian Concrete Association, Oslo.
- Makurat, A., N. Barton, G. Vik, P. Chryssanthakis and K. Monsen, 1990, "Jointed rock mass modelling", International Symposium on Rock Joints. Løen 1990. Proceedings, pp. 647-656, 1990.
- Varley, P., A. Darby and E. Radcliffe, 1992, "Geology, alignment and survey", *Proceedings of the Institution of Civil Engineers, The Channel Tunnel, Part 1: Tunnels*, pp 43-54.