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THE APPLICATION OF ENGINEERING GEOLOGY TO THE CONSTRUCTION OF DAMS IN THE UNITED KINGDOM

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ABSTRACT

There are over four hundred large dams in the United Kingdom and these have been constructed over the past two centuries. Although the highest dam built to date is only 91 m the spectrum of dam-type is considerable and includes gravity, arch, various buttress and composite designs, rockfill and a wide range of earthfill structures. This variety in dam-type is a direct consequence of the regional geology of the country extending over effectively every geological system. Foundation problems associated with rock have occurred in the metamorphic rocks of the Scottish Highlands, the lower Palaeozoic of Scotland, Wales and England, and igneous rocks distributed throughout much of the country. Overconsolidated clays of Mesozoic and Tertiary age form the foundations to embankment dams in central and southern England. Most of the United Kingdom was glaciated and those areas which were not have been affected by periglacial activity. In consequence, glacial erosion and deposition leading, for example, to the formation of deep buried channels, periglacial freeze-and-thaw, cambering and valley-bulging have contributed significantly to difficult foundation conditions and different technical solutions.

INTRODUCTION

There are well over four hundred large dams in the United Kingdom which have been built over a period of two centuries. The earliest dams are small embankments which impound water originally used as storage for canals or mills. During the nineteenth century a large number of relatively small reservoirs were constructed for public water supply in the industrial midlands and north of England, and Scotland. Towards the end of the nineteenth century, the first mass gravity masonry dams were constructed to form major impounding reservoirs supplying large cities such as Liverpool, Manchester and Birmingham by aqueduct. This pattern of development has continued throughout the country until recent times as progressively larger reservoirs have been built for both regional and local supply. However, within the last few years there has been an introduction of a new mode of reservoir

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operation by the use of the river channel as the aqueduct so that the reservoir adopts a river regulating, rather than impounding, function. Localised hydro-electric development took place in a number of areas within Scotland and Wales between 1900 and 1940. However, in the period after 1945 extensive construction of conventional hydro-electric projects occurred in the Scottish Highlands and, to a lesser extent, in Wales; three large pumped storage schemes have also been built.

Geologically, the United Kingdom is underlain by rocks ranging in age from Pre-Cambrian gneisses to recent alluvial sediments. Inevitably dams have been constructed on a wide variety of geological materials covering this complete range. A wide range of foundation conditions and materials problem have been encountered during the design and construction of dams in these diverse geological environments. As a consequence there is considerable variation in the types of dam that have been constructed, differences of detailed design between dams of similar type and contrasts in method of foundation and cutoff treatment. Although matters other than geology have influenced such differences, there is little doubt that geological factors have played an important role.

There is a marked relationship between the type of dam foundation conditions encountered and the regional geology and stratigraphy of the country (Walters, 1962; Morton, 1973). It is possible to divide these main types of foundation condition into a series of groups which may be summarized as follows:

(*i*) Pre-Cambian and early Lower Palaeozoic metamorphic and granitic rocks: These dams are predominantly located in the Scottish Highlands where they are associated with hydro-electric schemes. There are a variety of dam types involving particularly buttress and gravity dams, and also, arch and embankment structures. In the main, sound rock is present close to or at the ground surface and the cover of overburden is limited.

(*ii*) Lower Palaeozoic slates, slaty mudstones and greywackes: the dams founded on these rock materials are mainly located in Wales, the Lake District and the Southern Uplands of Scotland, and include arch, gravity, buttress and embankment structures. The more slaty rocks have commonly been affected by glacial and periglacial action resulting in a disintegration and loosening of the rock mass.

(*iii*) Carboniferous cyclothemic sediments: These dams include the very large number of relatively small structures which have been constructed in the industrial midlands and north of England, and central Scotland. The dam foundations are composed of alternations of sandstone-shale/mudstone with some subsidiary coals and limestones. Most of the dams are embankments although several gravity dams have been built on more competent strata. The valleys in which these dams were built have often been extensively influenced by glacial erosion and deposition. Valley-side cambering, and valley bulging are commonly present.

(*iv*) Igneous rocks: Igneous rocks are distributed throughout the northern and western parts of the country. A number of dams, primarily of gravity type, have been constructed on a variety of rock types including granites dolerites and lavas some of which have been influenced by hydrothermal alteration.

(*v*) Mesozoic and tertiary clays: Overconsolidated marine and lacustrine clays are extensively developed in southern England. Several embankment dams formed from these plastic clays have been built on such relatively weak and deformable materials.

(*vi*) Glacial sediments: Glaciers extended as far south as a line between the Thames and the Bristol Channel during the Pleistocene. North of this line most dam foundations have been influenced to some extent by the consequences of this glaciation. In some cases, embankments have been constructed on deep buried channels.

(*vii*) Recent sediments: These weak, compressible post-glacial sediments are restricted to flood plains and the coastal margin. Low embankments have been constructed on such materials for the purpose of creating lagoons for the retention of water and waste such as fly ash.

The paper will draw upon five of these groups to illustrate the range of foundation conditions encountered in dam construction in the United Kingdom (fig. 1).

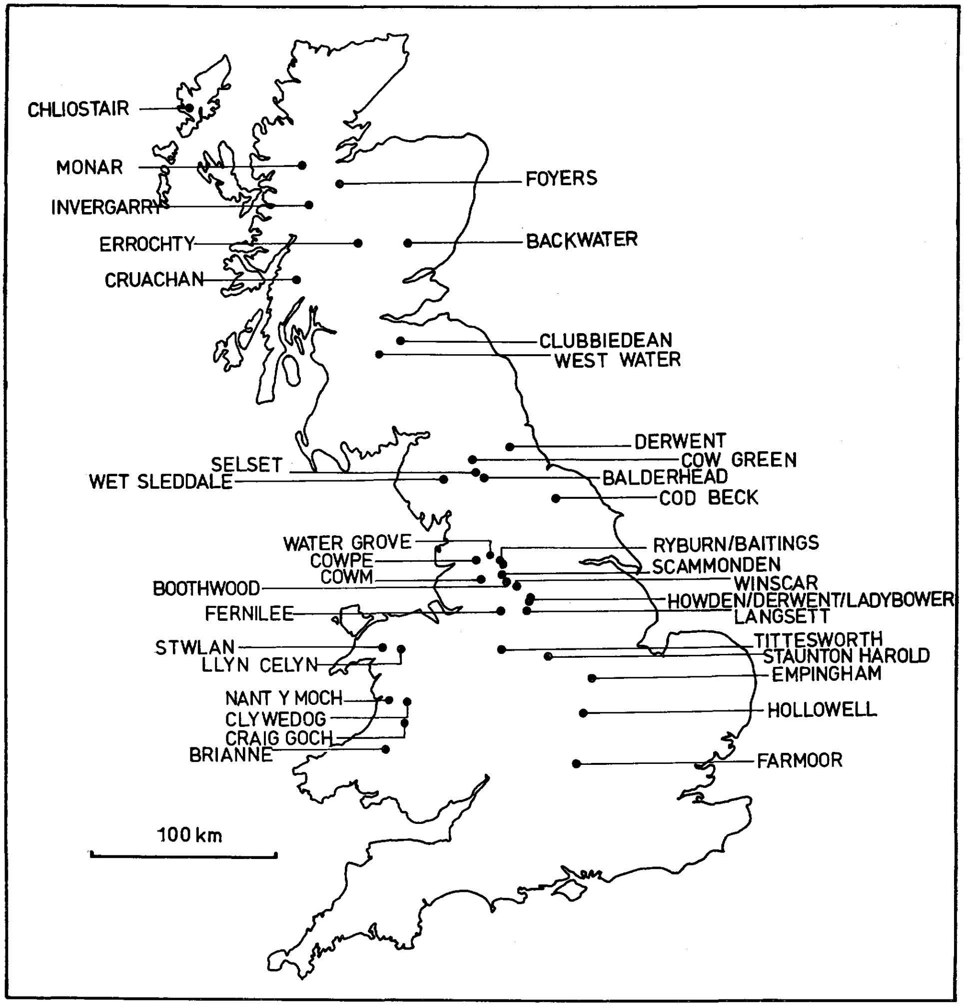

FIG. 1. — *Map of United Kingdom with localities of main dams referred to in text.*

FOUNDATIONS ON METAMORPHIC AND GRANITIC ROCKS OF THE SCOTTISH HIGHLANDS

The Scottish Highlands form a part of the Caledonian mountain chain and are composed of rocks ranging in age from Pre-Cambrian (Lewisian) pyroxene and hornblende granulites more than 2 600 ma old to late Pre-Cambrian (Moinian and Lower Dalradian) and early Palaeozoic (Upper Dalradian) rocks. These metamorphic rocks have been highly deformed and contain major granitic bodies of both migmatitic and intrusive type. The region has been extensively glaciated and, in consequence, fresh or slightly weathered rock is commonly exposed or present at shallow depth. In the main the rivers flows from higher ground near the western seaboard, or the Great Glen, along valleys falling in elevation towards the east (Johnstone and Crichton, 1966). These valleys are typically wide with a flat floor and gentle side-lopes rising towards steeper rocky hill-sides; bouldery glacial moraine, outwash gravels and alluvium is present. Narrow gorges are relatively unusual and, in consequence, most of the dams constructed have been of gravity or buttress type. However, in the case of the Invergarry dam, the diversion tunnel was deliberately constructed from the reservoir past a gorge infilled to a depth of at least 13 m by the debris of major rock falls in view of the potential risk of erosion triggering off further instability. Although the valleys are generally topographically and geologically suitable for the construction of embankment dams, only three embankments and two composite structures have been built.

By far the majority of the dams built in the Highlands have been constructed for the purposes of developing hydro-electric power (Fulton and Dickerson, 1964). In the period prior to World War II, five projects were constructed in connection with public supply and aluminium smelting having an aggregate installed capacity of 194 MW. Since 1945, a major programme of construction has resulted in the building of 53 large dams, 52 power stations and the driving of over 320 km of tunnel. These civil engineering works have been associated with the commissioning of 965 MW of conventional hydro-electric plant and 400 MW associated with the Cruachan pumped storage scheme (Knill, 1973); a further 300 MW pumped storage scheme has recently been completed at Foyers.

A variety of factors, apart from geology, determined the selection of dam-type. In all cases arrangements had to be provided for flood discharge and the most convenient arrangement frequently took the form of an overfall crest spillway thus supporting the use of a concrete gravity section. Despite the local availability of rock and earthfill, alternative tenders generally gave preference to concrete as distinct from embankment designs. A consequence of this situation was that a combination of good bedrock foundations, sources of good quality concrete aggregate, valley shape (length to height ratios typically in the range 7 to 15) and restrictions in the availability of cement led to the construction of a relatively high proportion of buttress dams. It was established that for dams in excess of about 35 m a buttress design was more economical than a gravity section. Buttress dams also carry the advantage that uplift pressures are minimised and there is ready access for installation of drainage measures. In the case of the embankment dams constructed for hydro-electric purposes, the lack of the suitable natural cohesive materials resulted in concrete being used either as a core or an upstream membrane.

However, in the case of the Backwater Dam (described later) the micaceous schist bedrock contribued to the formation of a cohesive glacial till suitable for placing as a core.

The foundations upon which dams have been constructed cover a range of rock-types including gneisses, psammitic granulites, mica and hornblende schists, schistose grits, quartzites, phyllites, metadolerites, and granites. In general terms, a suitable rock foundation could be created by excavation 2 to 4 m in depth. An investigation (Knill, 1970) has been made into the in situ properties of the bedrock by seismic methods at forty concrete dams in the Highlands. The mean longitudinal wave velocities characteristic of the bedrock and intact saturated specimens are summarized in figure 2. The ratio between these two observations is the Fracture

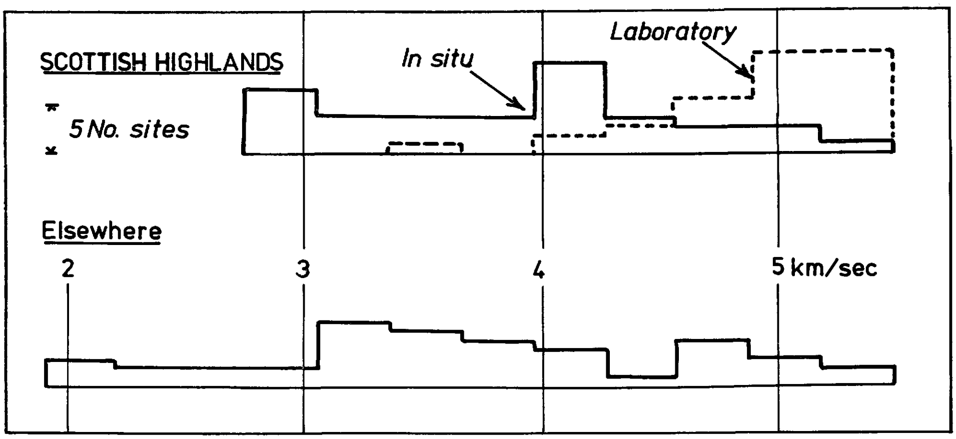

FIG. 2. — *Histograms of longitudinal wave velocity in dam foundations in Scottish Highlands and elsewhere in the United Kingdom.*

Index (Knill and Jones, 1965) which provides a measure of the natural fracture state of the rock mass. It will be observed from these measurements that satisfactory dam foundations for large dams can be created by natural rock masses with in situ velocities in the range of 3 000 to 5 500 m/sec. In comparison, similar data from 43 other concrete dam foundations formed by a wide variety of rock types in the United Kingdom are also presented in figure 2. It will be observed both that there is a wider range of velocity, and that lower velocities are encountered outside the Highlands; this is presumably a response to differences in rock type and depth of weathering. Despite these contrasts, the range in Fracture Index observed in the metamorphic rock foundations of the Highlands and those elsewhere is broadly similar. In the case of 9 sites, out of the 83 considered, the Fracture Index is in excess of 1 which can only imply that the rock mass is in a state of stress and relief is required before the fractures dilate. Such conditions appear characteristic of areas that have been strongly glaciated leading to the partial removal of rock cover. A further example of such regional differences in foundation properties can be drawn from a contrast between granites in the Scottish Highlands and South-West England (Knill, 1972):—

	No. of dam foundations	Mean in situ longitudinal wave velocity m/sec.	Mean Fracture Index
Scottish Highlands	6	4 360	·90
South-West England	7	2 760	·57

This situation is a further illustration of the contrast between the influence of glaciation in Scotland as compared to South-West England where periglacial action was important, weathered rocks were not removed by glaciation and hydrothermal influences are present.

By virtue of the satisfactory or good quality of the dam foundations available in the Scottish Highlands, there have been relatively few major foundation problems. In those cases where fractured or faulted rock was present, the foundations were generally treated, following additional excavation, by consolidation grouting. The Cruachan dam crosses a major fault zone cutting through granodioritic rocks and containing chlorite-covered shear surfaces. The bases of the individual buttress webs founded on the fault were splayed out in order to reduce the contact pressures. Glaciation has resulted in significant localised loosening of near-surface rock and the emplacement of sand and silt into open fractures. In the case of the Chliostair dam, a 15 m arch dam founded on micaceous gneiss, the rock foundations had been loosened by glacial plucking. The foundations were treated by flushing the fissures with pressurized water, rock bolting and final consolidation grouting. The major example, however occured at Errochty dam a 49 m high buttress dam founded on thinly foliated quartzitic schists dipping gently downstream. Thick seams of silt had been emplaced into the rock mass by a combination of glacial and periglacial action to a depth of up to 10 m below foundation level. The foundation area was divided into a series of "boxes" into which 13 holes were drilled and then injected with water to wash out the seams. Grout was injected once clear water was flushed from the holes. The total grout take in this operation was equivalent to a voids infilling of about 1 %.

TABLE 1. — *Summary of* in situ *and laboratory modulus tests in* $MN/m^2 \times 10^3$ *at Monar Dam*

Test method	Concrete in dam	Ungrouted rock	Grouted rock
In situ seismic	2.2	3.4	–
Laboratory dynamic	3.6	–	7.1
In situ plate bearing	–	2.2	–
In situ borehole jack	4.2	4.4	4.3
Laboratory static	6.3	9.0	8.4

The only major double curvature arch dam in the country is at Monar (Plate I) 50 km west of Inverness. The dam is 38 m in maximum height and has a theoretical chord to height ratio of 3.3. The gorge at Monar is eroded within micaceous psammitic granulites with thin micaceous layers (Henkel, Knill, Lloyd and Skempton, 1964; Roberts, Wilson and Wiltshire, 1965). The basis of design of the arch was that the modulus of deformation of the concrete and rock founda-

PLATE 1. — *General view of right bank of Monar dam during construction.*

tion were equivalent. This proposition was examined by means of a programme of in situ plate loading tests in an adit, borehole jacking tests using the Centex cell and seismic measurements; the Centex cell was also used in the concrete of the arch. The results of this programme are summarized in table 1 from which it can be concluded that the modulus of deformation of the rock mass is, in probability, rather higher than that of the arch. However, during construction a chloritic lamproschist dyke, 1.8 m in thickness, was identified dipping below the dam foundation at an average inclination of 38.5° (fig. 3). Up to 0.6 m of the dyke was altered to a green clay but subsequent calculations indicated that a maximum settlement of 3 mm could be anticipated. Most of this limited settlement would be expected to take place during construction. It was also recognised that a prominent (Plate 2) set of joints dipping upstream at an average angle of 31°. Consideration was given to the possibility that sliding of the dam might occur on a wedge of rock if the structure was founded at too shallow a depth. Shear box tests demonstrated that the peak and residual angles of shearing resistance unweathered fractures ranged between 48-55° and 41-44.5° respectively; clay-covered or micaceous foliation surfaces were associated with lower strengths. A stability analysis (Henkel, Knill, Lloyd and Skempton, 1964) revealed that the minimum factor of safety for an individual block of the dam was 2.2 without drainage and generally the factor of safety was 30 % higher allowing for some drainage effective at the dam toe. Overall, the dam has a factor of safety with regard to sliding of at least 3 assuming an effective drainage system. An extensive system of instrumentation was

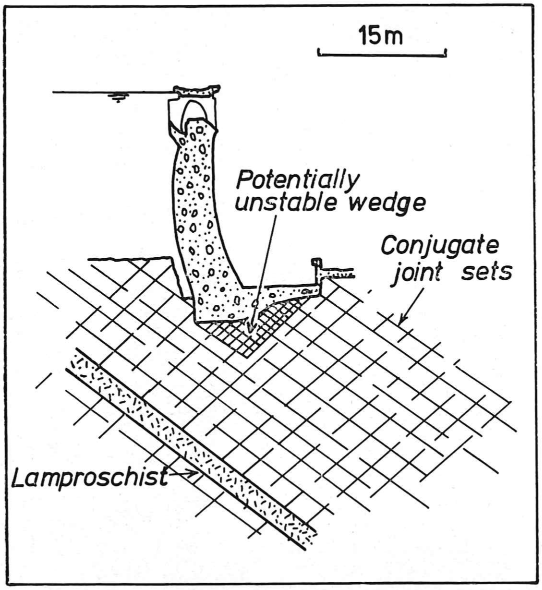

FIG. 3. — *Section through foundations of Monar Dam illustrating conjugate joint pattern and lamproschist dyke.*

PLATE 2. — *Detail of rock structure on right bank with upstream-dipping joints (dipping towards camera).*

installed, the greatest settlement observed being about 2 mm, and the greatest relative downstream movement of the dam being about 1 mm. These measurements confirm the rigidity of the foundations relative to the dam and the static stability of the structure.

FOUNDATIONS ON LOWER PALAEOZOIC SLATES AND SLATY MUDSTONES

Extensive outcrops of Lower Palaeozoic rocks occur in the more mountainous regions formed by Wales, the Lake District and the Southern Uplands of Scotland. In each of these regions, major water supply schemes for cities have been constructed by the progressive development of a series of reservoirs over a total period of about 80 years. For example, Birmingham is supplied by aqueduct from mid-Wales by four reservoirs (Caban Gôch, Pen-y-Gareg, Craig Gôch, Claerwen) in the Elan Valley of the River Wye catchment and by the Clywedog river regulating reservoir in the River Severn catchment. It is likely that future developments will centre on replacing the Craig Gôch dam by a new, higher structure discharging by tunnel into the Severn catchment. Much of this mountainous or hilly country is underlain by rocks of geosynclinal facies in a state of low grade regional metamorphism including greywackes, rare conglomerates and abundant argillaceous rocks transitional between slates and shales generally referred to as slaty mudstones. Considerable experience has now been obtained in the construction of dams and tunnels in such terrain.

The slaty mudstones are poorly cleaved slates, the rock tending to break readily along the cleavage into lenticular or lensoid masses and fragments. The rock material is composed of finegrained quartz with an illite-sericite matrix and minor accessory minerals. Typically the fold structures are upright and, in consequence, the slaty cleavage is normally vertical or steeply dipping. The slaty mudstone is a strong, durable rock and when tested normal to the cleavage has an unconfined strength of about 100 MN/m^2; the strength is about one third this value when loaded parallel to the cleavage. One unusual feature of this rock-type (Knill, 1970, p. 65) is that it, contrary to typical rock behaviour, has a higher longitudinal wave velocity when dry as compared to the saturated condition; this may be a reflection of the influence of swelling of the phyllosilicate minerals. The anisotropy in the rock material is also reflected in the engineering behaviour of the rock mass. In situ seismic velocity measurements are typically 10-15 % higher when measured parallel to, as distinct from across, the cleavage.

Marked surface disintegration of the rock occurs along the direction of the cleavage so that surface outcrops commonly occur as ribs of harder rock extending parallel to the cleavage. Surface outcrops often have a shattered appearance as a result of the penetration of the effects of mechanical weathering along the cleavage, faults and zones of fracturing. It is apparent that this weathering is largely a relict of glacial and periglacial action. During glaciation (Knill, 1968) near-surface shears are developed as a consequence of glacial drag and this can lead to a general loosening of the rock mass. These shears tend to form a crude lensoid pattern roughly paralleling the ground surface similar to stress relief fracturing. During, or following glaciation, fine grained silty or clayey material can be introduced into these fractures, eventually forming weak seams in the rock mass. Observations in tunnels have demonstrated that such effects can commonly extend to depths of at least 30 m. Presumably once the glacial shears have been developed in bedrock, the increase in secondary permeability will result in extensive freeze-and-thaw action in a periglacial environment, which would permit the ready emplacement of finegrained detritus. At Wet Sleddale some of the clay-silt seams were layered indicating the influence of aqueous deposition in their origin. An engineering consequence of this rock condition has been that additional excavation, beyond that anticipated, has been required at a number of dam sites. In the case of the Nant-y-Moch Dam (Anderson, 1963), a 51 m high buttress dam in mid-Wales, the eastern flank of the valley was extensively disturbed and rock blocks near the surface were found to be separated from the main rock mass by seams of clay. Deeper excavation was required beyond that predicted on this flank of the dam and the foundations were generally more difficult than on the western side of the valley. At the Stwlan Dam (Anderson, 1970), a 36 m high buttress dam in North Wales, large blocks of fresh rhyolite were found to be separated from one another, and the in situ bedrock, by clay-filled fissures. In consequence, the cutoff trench was carried to sound rock at a depth of 12 m and the stability of the dam improved by the construction of a continuous mass concrete footing under the higher buttresses. More extensive frost-shattering of slaty bedrock had occurred at the Wet Sleddale Dam (Knill, 1968), a 23 m high gravity dam in the Lake District. This shattering, taken together with the occurrence (Plate 3) of deep, clay-filled glacial shears resulted in broad foundation excavations up to 12 m in depth and the provision of an additional structural cutoff in the dam profile. One linking feature between

PLATE 3. — *Deep glacial shears at 10 m depth in slaty tuffs at Wet Sleddale dam.*

several sites is that the periglacial effects of rock shattering and clay-silt seams are most prominent on south or west facing slopes. Presumably this situation results from the exposure of such slopes to the extremes of diurmal and seasonal changes in air temperature contributing to the greatest opportunity for intensive freeze-and-thaw action. In contrast the northerly or easterly facing slopes, being in shadow, remained in a frozen state and did not suffer extensive disintegration.

Within the past decade two major dams have been built in mid-Wales for the purposes of water supply. The Clywedog dam, a 71 m buttress dam was constructed in 1964-1968 to be followed by the Brianne dam, a rockfill dam 91 m in height built in the period 1968-1972. These two dams, which clearly illustrate the range of foundation and material problems encountered in the region, will be discussed at greater length.

Three alternative sites were originally considered for the Clywedog dam (Fordham, Cochrane, Kretschmer and Baxter, 1970) but these were resolved into a single site in the lower part of the Bryntail Gorge which is a glaciated valley eroded in Ordovician slaty mustones, siltstones and slates with some thick bands of greywacke. There was a general absence of earthfill materials suitble for a dam core in the area although rockfill could have been quarried locally. The valley profile (width/height of 3/1) was appropriate to an arch design but uncertainty with regard to the condition of the rock and the dip of greywacke layers could have given rise to foundation difficulties. As the estimated cost of a mass gravity dam was 24 % higher than that of a buttress dam it was decided to adopt a buttress design. The river was diverted through the buttresses during construction and slabbing between the buttresses has provided an overflow spillway for virtually the complete crest length of the dam.

The Bryntail Gorge is crossed by a ridge formed by a 36 m band of massive greywacke dipping downstream at 40-50° (fig. 4). On the right bank of the valley there is a sharp fold in the greywacke which results in an inward dip into the hillside of 45°; the greywacke is intensely fractured and quartz-veined at the fold hinge. Slaty mudstones outcrop upstream of the greywacke, the cleavage generally dipping steeply upstream and striking normal to the valley. The cleavage is vertical or locally dips downstream close to the greywacke band. A series of slaty shales, siltstones and sandstone bands outcrops downstream of the greywacke. The dam was located so that the buttress toes were founded on the greywacke which would thus take the form of a structural member carrying the foundation stresses to depth. The bulk of the dam foundation however, would be composed of the slaty mud-stones and the question arose as to the relative deformability of these two rock types. In situ jacking and seismic tests (Knill and Price, 1972) revealed a consistent relationship between the static (3rd cycle) and dynamic moduli of deformation for the in situ fresh rock. A seismic investigation prior to excavation revealed in addition that the modulus of deformation at constant depth decreased with increase in elevation, the modulus at top water level being about 15 % of that below the valley floor (Knill, in discussion, Fordham *et al.*, 1970). On general grounds, it might have been expected that the greywacke would have the higher modulus but in the event proved the reverse to be the case, with the slaty mudstone and greywacke having moduli of about 30 and 18×10^3 MN/m^2 respectively. A finite element analyses of the dam and its foundation was carried out in which the moduli were varied as follows:—

$$E_{concrete} = 2\,E_{greywacke} = 1, 2 \text{ or } 4\,E_{mudstone}\,.$$

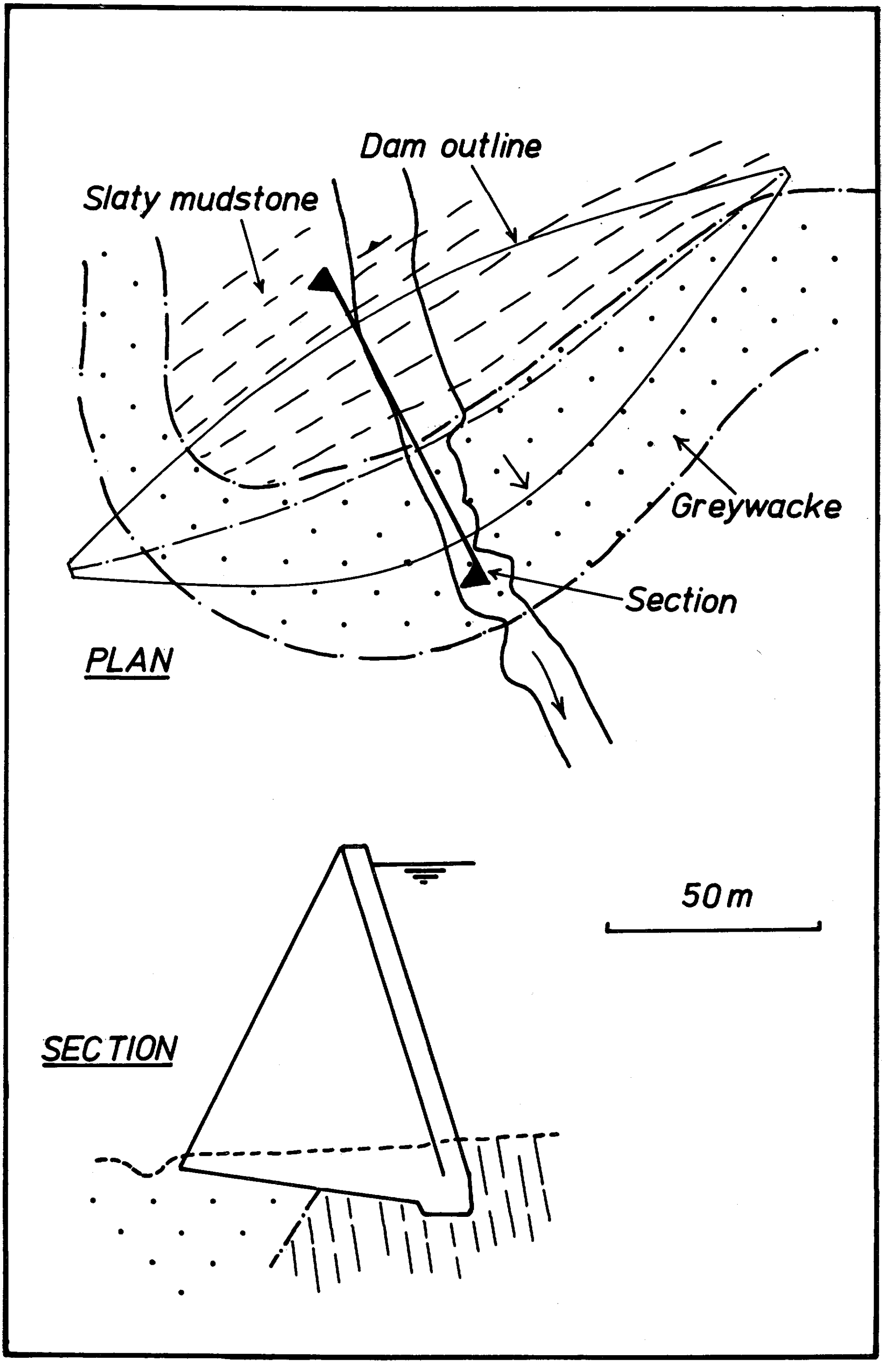

FIG. 4. — *Geological plan and section of Clywedog Dam.*

An important feature of this analysis was a concern as to how the stresses would be transmitted downstream of the dam. The Van Lode a mineralized vein which had been previously worked for galena and barytes, crosses the valley about 130 m downstream of the dam (Knill, 1971). The vein is 14-15 m in width and althought some 19th Century plans were available, the detailed location of the many mined-out voids was unknown. It had to be assumed, for structural purposes, that stresses could not be transferred across the Lode; it was established that this lack of support reaction would not influence dam stability.

During construction it was established that the depth of mechanical disintegration of the slaty mudstones was greater than anticipated. In particular, glacially-induced fractures roughly parallel to the ground surface were identified on the valley sides. This necessitated a general lowering of the foundations by some 4.5 m to a total depth of about 8-10 m on the valley sides. In retrospect, it is clear that this condition was predicted from the decrease in modulus of the slaty mudstone with increase in elevation. An additional factor of importance was that, with the stepped profile of the excavated valley side, there was risk of lateral failure from one buttress to the adjacent buttress immediately below. This situation was coped with by providing mass concrete in each buttress plinth to ensure adequate toe weight. The foundations were subjected to intensive consolidation grouting, the general take being between 0.1 and 0.2 % voids infilling. In addition a grout curtain was constructed below the broad structural cutoff at the dam heel to a maximum depth of 20 m reducing to 9 m on the upper parts of the valley sides. The aggregate used in the dam was won from a local quarry, opened for the purpose, in a band of greywacke downstream of the dam.

The Brianne dam is located in a narrow V-shaped gorge eroded in Silurian slaty mudstones 19 km north of Llandovery (Carlyle and Owen, 1973). The River Towy is vigorously downcutting into the river bed and rock is exposed over much of the site. The left side of the gorge, which faces south-west, has been severely eroded by weathering and clearly demonstrates the more intense effects of periglacial freeze-and-thaw action referred to previously. Preliminary designs were prepared for four dam types, the ratio of the estimated costs being as follows:—

Gravity	1.44
Buttress	1.23
Arch	1.14
Rockfill	1.00

As at Clywedog, there was uncertainty as to the extent of remedial work which might be required on the dam abutments to form arch dam foundations. Suitable core material was available from a glacial moraine 11 km upstream in the reservoir area, but there were no suitable sources of concrete aggregate locally available in the quantities required for a major concrete dam. At the dam site the River Towy changes sharply in direction and this permitted an overflow chute spillway to be cut into rock adjacent to the dam. In view of these site circumstances, a central core rockfill design was selected for the site (Plate 4).

Excavation of the site was relatively straightforward, detailed foundation treatment being limited to the core-contact zone. The more open-jointed rock was removed and then the exposed bedrock was coated with pneumatically applied mortar 50 mm in thickness. Faulted or fractured zones were also covered by 100 mm 9 gauge steel wire mesh pegged to the rock face. The area was then

PLATE 4. — *General view of Brianne rock fill dam under construction.*

contact grouted to a depth of 10-15 m and a central cutoff between 45-75 m in depth formed. The only geological problem encountered was the location of deeply eroded, steep-sided fault zones up to 15 m deep and infilled with gravel and covered by scree (Rawlings, 1970) ; they were excavated out and backfilled by concrete. It is probable that the gorges represent sub-glacial channels eroded by glacial melt water spilling over the left bank spur when main valley was blocked by ice.

An extensive investigation was carried out into the properties of the slaty mudstone as rockfill material. It was found that adequate compaction with limited breakdown of the oversize blocks occurred during placement of the fill by a D8 bulldozer in 0.5 m layers and subsequent compaction of 1 m layers with a 13.5 tonne vibrating roller; dry densities were of the order of 2.1 tonne/m^3. Rockfill was tested for shear strength in a 250 mm diam. triaxial cell. ϕ' in terms of effective stresses varied from 50° at a major principal stress of 7 MN/m^2 to 44° at 21 MN/m^2. Tests with the cleavage fragments orientated parallel to the plane of failure gave results about 4° lower than for a random orientation. The moraine, used for the core, was a stoney-silty clay; the compaction and in situ water contents of which were in practice the same, apart from fine weather when light watering was used. The rockfill embankment contructed has an upstream slope of 1 on 2 and a downstream slope of 1 on 1.75 ; the sideslope of the central core is 4 on 1 with a minimum width of 6 m.

In summary it is of interest to contrast the two dams at Clywedog and Brianne in view of the very different designs which were adopted in similar geological environments:

	Clywedog	Brianne
Maximum dam height, m	71	91
Reservoir storage, Ml	49 900	61 000
Excavation for dam, m^3	114 680	225 000
Dam volume, m^3	145 250 (concrete)	2 040 000 (fill)
Cost of dam, £m	3.95 ([a])	5.18
Time from start of construction to impounding, years	2.7	3.2

([a]) Includes power generation and a minor dam.

The contract price for Brianne dam was £3.54 m, a 46 % rise in anticipated final cost occurring during construction overtaking the very marked estimated cost differential between the buttress and rockfill designs. It is also of relevance to note that by virtue of the very much larger foundation area involved, excavation for the Brianne dam was much greater than the deeper but more localised excavation at Clywedog. The proposed new dam for the existing Craig Gôch site, which will be of comparable size to that at Brianne, is likely to be a rockfill structure. In this case, also, there are no potential sources of concrete aggregate nearby and, in addition, lack of suitable core materials may result in the use of an artificial upstream membrane.

FOUNDATION AND MATERIAL PROBLEMS OF DAMS SITED ON CARBONIFEROUS CYCLOTHEMIC SEDIMENTS

The Carboniferous system of England and Wales may be conveniently grouped into three units which in upward sequence, are represented by the Carboniferous Limestone Series, Millstone Grit and Coal Measures. Apart from those areas where all or part of the Carboniferous Limestone Series is represented by massive limestones, cyclothemic sedimentation is characteristic of much of the British Carboniferous. Typically the cyclothems are represented by limestone-shale-sandstone, shale-sandstone and shale-sandstone-seatearth-coal as on passes upwards through the System. The Carboniferous Limestone Series and Millstone Grit outcrop extensively in the hilly and mountainous Pennines of northern and north-central England, western England and South Wales; rocks of broadly similar facies occur in central Scotland. These elevated areas are adjacent to low-lying coal field regions. In view of the location of industry within coalfields during the nineteenth and twentieth centuries it is inevitable that the upland areas should have provided a potential source of water particularly bearing in mind the higher precipitation in such areas. Initially, the sandstone and limestone layers would have provided adequate local sources directly from springs or by ensuring adequate maintenance of groundwater discharge during dry weather. However, sufficient storage was guaranteed through the creation of impounding reservoirs supplying domestic and industrial requirements by aqueduct. The coalfields were, at that time, less suitable for reservoir construction being at lower elevations and subject to potential leakage as a consequence of subsidence fracturing (Knill, 1971*b*). In addition, the rivers in the coalfields were (and still are) used as drainage channels for mine waters, industrial waste products and sewage. It was natural to look at the uplands for reservoir sites and, by virtue of the prevalence of limestone layers in the Lower Carboniferous, most reservoirs have been developed on the shale-sandstone rhythms of the Middle Carboniferous Millstone Grit.

It is possibly surprising, in retrospect, that the feasibility of the construction of reservoirs on the limestone-bearing rocks of the Lower Carboniferous was never seriously questioned for many years. The unsuitability of such sites for reservoir construction has become virtually dogma despite the fact that, up to the mid-1960's, five dams had been successfully constructed on limestone-bearing Carboniferous rocks in the British Isles and there was only one case of a reservoir failure directly due to leakage through limestone (which occured in 1912). As a consequence the proposal to construct a major reservoir at Cow Green (Kennard and Knill, 1969; Knill, 1971*b*) on an alternating limestone-shale-sandstone sequence met with some criticism. The site had previously been rejected on geological grounds but re-evaluation proved these fears to be groundless. The reservoir has been completed and was impounded in 1970; no leakage has been observed.

Valleys eroded in the Middle Carboniferous are characteristically associated with a set of superficial disturbances which can have a significant influence on dam design and construction. In their simplest form these disturbances are represented by valley bulging with valley-side cambering (Hollingworth, Taylor and Kellaway, 1944; Knill, 1971*b*). The features specifically linked with the valley bulging process may be summarised as follows:—

(i) a single anticlinal fold dying out with depth and whose axis is essentially parallel to valley. Such folds typically occur in valleys occupied by shales interbedded with sandstone layers and bands; the sandstone bands are generally open-jointed. In the case of the Fernilee Dam, the amplitude of the main anticlinal fold formed in shales and siltstones, was about 35 m, the wavelength being about 20 m. At the 21 m Staunton Harold dam, the anticlinal fold in shales interbedded with sandstones and a coal is more open, having an amplitude of 16 m and a wavelength of 330 m (fig. 5*a*).

(ii) small-scale folds are commonly present in the valley floor and they may adopt a parasitic relationship to the main fold, if present. Such small folds have an amplitude of a few metres, terminating rapidly with depth in the manner of cylindroidal folds (fig. 5*b*). The folds may occur as separate sharp-crested anticlines merging into reverse faults or as groups of folds extending upwards from the same stratum. The bedding surfaces in these circumstances are commonly polished and slickensided providing evidence of horizontal displacement in the formation of the folds. In the cutoff trench for the Langsett dam the shales were described as being "folded and wrinkled by lateral thrust" and that fissures and cracks in the shale were filled with silty clay (Lapworth, 1911).

(iii) faulting is frequently present in the dam foundations although uncertainty does exist as to the age of the faulting and whether it is a direct response to a valley bulging process. However, the fault gouge may incorporate materials of superficial origin indicating that at least part of the displacement occurred below a shallow cover of overburden. The relative fault movements can indicate both a thrusting of the valley side over the valley floor or a horst-like displacement of the valley floor analogous to a bulge-like structure (fig. 5*c*). Apart from discrete faults, highly disturbed fractured rock is commonly present.

(iv) several cases of extreme disturbance, interpreted as deep-seated movement of the bedrock into the valley floor have been identified (fig. 5*d*), in some dam foundations. It is, however, not clear whether such rock slides are associated with valley side movements generated as part of the valley bulging process.

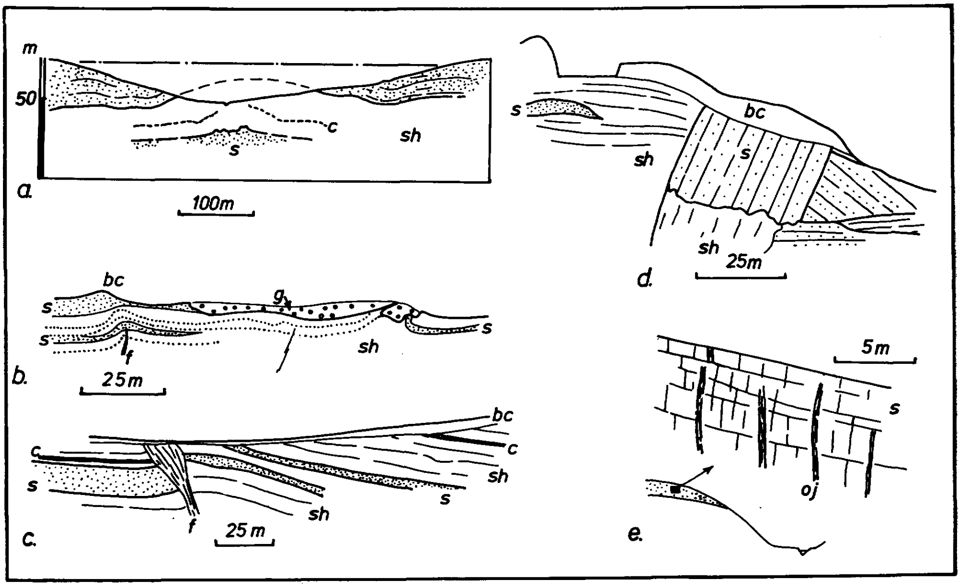

FIG. 5. — *Structures encountered in dam foundations associated with valley bulging. a) valley bulge at Staunton Harold Dam; b) small-scale crumpling in Langsett Dam; c) fault in valley floor at Underbank Dam; d) severely disturbed sandstones and shales at Woodhead Dam; e) open fractures generated by cambering on valley side at Boothwood Dam. s, sandstone; sh, shale; c, coal; bc, boulder clay; oj, open joint; g, gravel.*

Little information is available on the reasons for the development of the different forms of valley bulging at specific localities. It is apparent that sandstone layers, 15 m or more in thickness, can provide effective struts across a valley floor reducing the bulging to either a minor arching, or encouraging fault development (fig. 6). For example, in the case of the Langsett and Underbank dams, constructed in the same valley west of Sheffield two quite distinct responses have been identifield. At Langsett, the shales are fractured and folded to a depth of 30-35 m. However, at Underbank, a 10 m thick sandstone layer acts as a protective cover to the underlying shales and the reaction to valley bulging appears to have taken the form of reverse faulting. The most marked effects of valley bulging

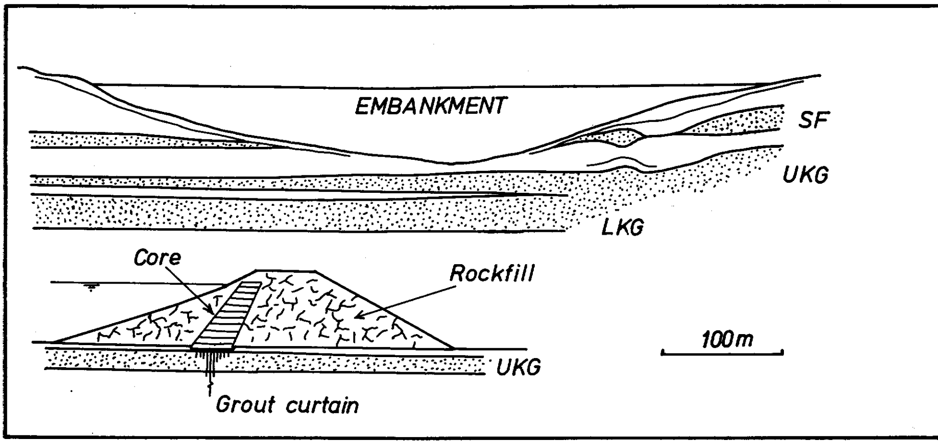

FIG. 6. — *Longitudinal (viewed from upstream) and cross profile of Scammonden Dam illustrating lack of valley bulging apart from minor fold on right bank. SF, Scotland Flags; UKG, Upper Kinderscout Grit; LKG, Lower Kinderscout Grit.*

occur in those situations where thin sandstone layers are interbedded with shales. In consequence, it appears likely that both the lower strength of the shales and the aquifer-influence of sandstones can be important factors in locating and determining the scale of valley bulging. No totally consistent origin has been proposed to explain the valley bulging process but the following factors could be contributory:—

(i) artesian groundwater pressures at depth;
(ii) glacial and/or periglacial freeze-and-thaw;
(iii) release of in situ (overburden and tectonic) stresses due to erosion (Ferguson, 1967);
(iv) valley-bottom release of vertical stress under thick ice cover resulting from subglacial drainage;
(v) deformation and creep of valley bottom due to valley side loading and
(vi) swelling of shale materials.

Valley bulges occur elsewhere in the country and their implication on dam foundations in Mesozoic clays will be discussed in the next section. The limited evidence available indicates that most valley bulging is of glacial or post-glacial age; there are slight indications that some valley bulging may still be active. It is somewhat curious that valley bulging should not have been identifield in dam foundations elsewhere in the world, particularly in the Carboniferous of the Appalachians. Ferguson's (1967) descriptions of valley bottom structures in dam foundations in the Allegheny Plateau near Pittsburgh in the U.S.A. are comparable, but small-scale, to those in the British Carboniferous. The Ghrib Dam (Cheylan, 1952), in Algeria, appears to be associated with a small valley bulge in interbedded Miocene grits and marls.

Massive sandstones on the valley-sides have almost universally been influenced by cambering, leading to a tensile separation of the rock mass into individual joint blocks (fig. 5*e*). The open fractures become infilled by overburden materials.

The engineering consequences of valley bulging and cambering are that the rock mass is affected by dilation, thus increasing the secondary (fracture) permeability and porosity. The effects of valley bulging and cambering, when present, generally extend to depths of up to 30-50 m below the valley floor and at least 150-250 m into the valley sides. In the case of the earliest dams constructed in the Pennine hills of north-central England the occurence if valley bulging was not recognised. In the case of the Cowm embankment built and reconstructed over the period 1867-1886 the occurrence of tension fractures in the arched sandstones were not recognised until the reservoir failed to fill in 1870. Excavation through the clay core of the dam exposed the fractures, initially unsuccessfully filled with clay, and eventually replaced by concrete. Sandstone layers have acted as important sources of leakage unless adequately protected by a blanket of overlying superficial deposits and shales. The Cowpe reservoir (1897-1902) leaked through a 6 m bed of sandstone, the clay core of the embankment having been terminated in a shallow cutoff within a thin overlying layer of argillaceous seatearth. Subsequent leakage resulted in the construction of a 15 m deep cutoff trench in adit and backfilled with concrete. The cutoff was excavated from the base of the clay core, through a horizontal 14 m sandstone layer which was the cause of the leakage into underlying shales.

Experiences of this type, which were often accompanied by the significant water inflows into relatively shallow cutoff excavations during construction, led to the development of the concept of the "Pennine cutoff". An impressive record of the early history of the construction of such cutoffs is presented in Lapworth's (1911) paper on "The geology of dam trenches", one of the first classics of dam geology.

The cutoff generally took the form of a 2 m wide trench excavated through overburden and to a sufficient depth in rock considered necessary to achieve a "watertight reservoir". The trench was backfilled in the early years, and at least up to the 1950's, by puddled clay; however, the use of clay was progressively replaced by concrete. Although grouting had been used by the early 1900's and was being applied systematically in the 1920's it was not until the late 1960's that reliance was being placed primarily upon grouting in creating an adequate cutoff. There is little doubt that the "Pennine cutoff" design became almost indiscriminately applied to the solution of any apparently adverse set of geological conditions whether in the Middle Carboniferous of the Pennines or not. It is possibly only in the last decade that the question of cutoff design has undergone serious re-evaluation in the light of bedrock and overburden geology, groundwater conditions and reservoir operation in relation to downstream discharge requirements.

Possibly the largest cutoff constructed in the Pennines was that at the Ladybower dam (1940-1942) which is an embankment 42 m in height. The cutoff was excavated to a maximum depth of 72 m and 9×10^6 litres/day were pumped from the trench. Lateral wing trenches were carried out in adit to distances of 150 m away from the ends of the dam. Grouting was carried out over an area of about 32,000 m^2 below the main cutoff and the wing trenches and this resulted in a mean grout take of 50 kg cement/m^2 of curtain which is an indication of the massive disturbance which had extensively influenced the rock. There were frequent descriptions of large fractures, up to 0.5 m in width, present in valley-bulged sandstones below Pennine valleys. In the case of early gravity dams such as those at Howden and Derwent (Derbyshire), which are 33 m in height, major excavation to a depth of 21 m was required to carry the foundations below the influence of bulging. The wing trenches, formed in tunnel at Derwent, are 180 and 240 m in length respectively although in more recent gravity dams, such as those at Ryburn and Baitings, the treatment of the tensile fractures resulting from valley-side cambering has been effected by a fanned series of grout holes. There is little doubt that grouting has been very effective in the reduction of loss from these reservoirs. For example, at Fernilee reservoir (1936) a leakage of 6×10^6 litres/day was reduced by 90 % by grouting and a 25 m sandstone layer below the Water Grove dam (1930-1935) was effectively treated, prior to impounding, by a grout curtain. Nevertheless, there has continued to be some reticence in relying fully upon grouting.

At Selset dam, a 38 m embankment, constructed in 1955-1960, a conventional concrete cutoff was constructed to a maximum depth of about 30 m (Kennard and Kennard, 1962) into a series of shale layers interbedded with gently dipping sandstones and thin limestone bands. A grout curtain was taken to a maximum depth of 60 m below the base of the cutoff, the average take being 75 kg cement and sand/m^2 of cutoff, being well in excess of that recorded at Ladybower. However, at Selset the grouting pressures used, which achieved a maximum of 15 MN/m^2, were well in excess of those required to cause hydraulic fracture of the shales. It, therefore, appears likely that this relatively high grout take was a consequence of injudiciously high grouting pressures rather than a response to an open-jointed rock mass (Morgenstern and Vaughan, 1963). During impounding of the reservoir there was a significant rise in groundwater pressures below, and on the south flank of, the dam (Bishop, Kennard and Vaughan, 1963). The pressures below the dam were reduced by installation of relief wells. However, in the case of the groundwater in the valley sides, it was apparent that the increase in pressure was a response to the diversion of the natural hillside groundwater consequential on the creation of the reservoir. This condition is by no means unusual, as during the impounding of the Tittesworth reservoir, new springs discharging 450,000 litres/day appeared 250 m downstream of the dam; tests demonstrated that the water had not been derived from the reservoir (Arden, 1964). At the Balderhead dam, a 47 m embankment constructed on shales in the valley immediately to the south of, and after, Selset, the cutoff trench remained in the design although the actual cross-sectional area completed is only a proportion of that originally intended. A grout curtain was constructed below the cutoff and as a consequence of careful restriction of grouting pressures the unit grout take was reduced by about 80 % compared to Selset. Uplift pressures were, as at Selset, also generated downstream of the dam which were dissipated into relief wells. Some new seepages developed from hillside groundwater diverted as a result of the new hydrological regime created by the reservoir. In situ testing of the shales indicated a permeability in the range of $10^{-4} - 10^{-5}$ cm/sec which was reduced to $5 \times 10^{-5} - 5 \times 10^{-6}$ cm/sec by grouting. Although the permeability of a grouted cutoff might be two or three orders of magnitude less than that of a concrete of clay cutoff, the total quantities of water retained may be quite uneconomic in the context of the additional delay caused by, and the cost of, the construction of a cutoff trench. At the Scammonden dam, a 76 m rockfill dam built between 1966-1969, the foundations consist of an alternation of sandstones and shales which dip gently downstream (fig. 6) (Plate 5). Although the flaggy sandstones on the valley sides are fractured, cambered and to an extent landslipped, there is only minor valley bulging presumably as a result of the beam-like influence

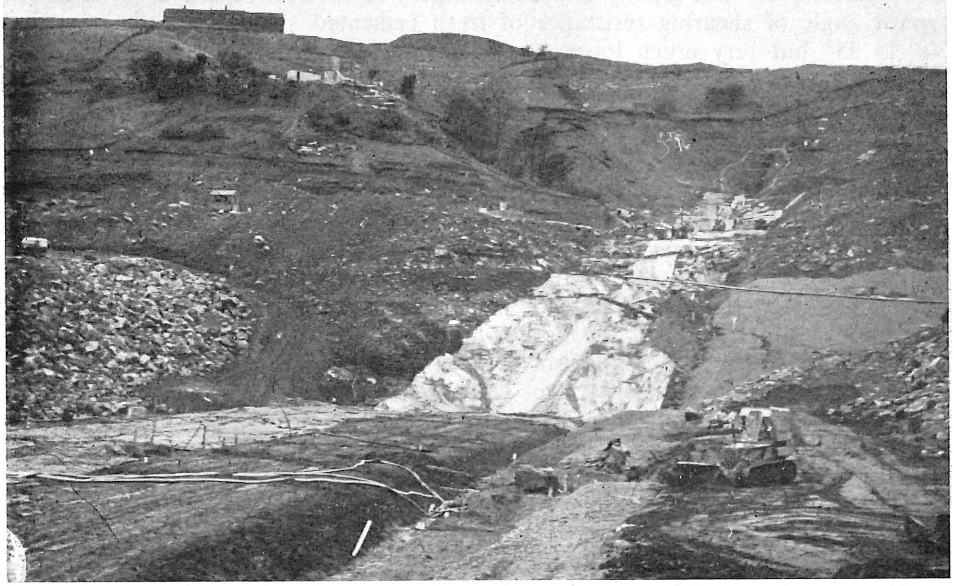

PLATE 5. — *Left bank of Scammonden dam during construction illustrating core, filters and rockfill in foreground and cutoff zone in middle distance.*

of the combined 48 m thickness of the Upper and Lower Kinderscout grits which underlie the valley. Blanket grouting was carried out over the exposed contact area between the base of the inclined clay core and the rock foundation; the rock surface was shotcreted. A shallow concrete-filled cutoff trench was constructed primarily to provide a grout cap but this was excavated to a greater depth in the fractured flaggy sandstones on the valley sides. It was originally intended to rely on a single line of grout holes but, as the take increased with depth, it was decided to use a three-line curtain, the central and flanking lines penetrating to the base of the Lower and Upper Kinderscout Grits respectively. The overall grout take was 130 kg cement and sand/m^2 of cutoff. A similar form of grouted cutoff with a vestigial cutoff trench is being installed at the Winscar dam, a 53 m rockfill embankment, currently under construction.

Satisfactory foundation conditions for gravity dams occur in those valleys where thick sandstone layers predominate. However even in such conditions, the sandstones can be faulted and are open-jointed on the valley sides as a result of cambering. This situation gave rise to a toppling failure of a sandstone block at Thruscross dam on shales dipping into the excavation. In general terms, fracturing associated with arching and cambering is coped at such dams with additional excavation of unsuitable materials and grouting, as at the Boothwood dam a 57 m gravity structure completed recently. Deep cutoff trenches have also been used with gravity dams despite the fact that downstream rotation of the dam crest would most certainly give rise to tensile fracture of the join between the dam heel and the top of the cutoff. The typical gentle dip of the bedding, together with the presence of shales, has occasionally given rise to some uncertainty as to stability of both gravity and embankment dams with regard to sliding. The typical angle of shearing resistance of fresh cemented shales is in the range of 30° to 35° but very much lower angles, of the order of 16° to 20° have been encountered in less well cemented or more fissile clay-shales. Typically, the stability problem has been overcome by the provision of a widened, deeper structural cutoff and additional excavation in order to activate a larger passive wedge in the rock dowstream.

The natural materials available in the Pennine valleys for construction purposes include boulder clay (suitable for core and earth fill), localised sands and gravels, sandstone (suitable as aggregate, rip-rap and rockfill) and shales (suitable as earthfill and, in a processed form, as a core). Most of the earlier embankments were constructed from boulder clay which was acceptable where the structures were moderately small and the limited rate of construction permitted the dissipation of construction pore pressures over a period of non-working in winter. However, with an increase in size of embankments constructed, boulder clay has become a less attractive material particularly in view of the implications of summer rainfall on the rate of placing suitable fill. Within recent years, there has also been a trend towards the use of sandstone as rockfill, as in the case of Scammonden and Winscar, and shale, as at Balderhead as fill materials. Providing these materials can be excavated economically, when placed they prove to be free-draining, durable materials more appropriate to modern techniques of excavation and compaction than boulder clays. An extensive investigation was carried out at Balderhead into the suitability of the local shales (Kennard, Knill and Vaughan, 1967). It was established that, below a cover of weathered and more fractured shale, the material was inherently stable and not subject to excessive fragmentation during handling. The placed shale had a permeability of the order of 10^{-3} cm/sec and the embankment was designed on the basis of $\phi' = 33°$. Extensive rock trials were carried out prior to the construction of the Scammonden dam (Williams and Stothard, 1967) which also forms an embankment on the M62 moterway, the rockfill having been won from sandstones and shales excavated from two major cutting. During construction it was established that greater fragmentation of the rockfill occurred than predicted possibly as a result of the rock having been quarried from a hill-top situation where the influence of weathering could have been greater. Inadequate clay occurred in the reservoir area to form the inclined core and suitable material had to be imported to the dam site.

It will be appreciated from the preceding discussion that, although there are a number of modern dams in the Pennines, a large number are relatively old, several having been constructed well over a century ago. In many cases, records are minimal or non-existent (Kennard, 1972). This situation frequently has implications on geology, particularly to provide assistance in identifying possible sources of construction materials used and predicting potential sites of leakage. Deterioration of the conditions of such dams is commonly identified initially by subsidence in the embankment and development of new, or increased, seepages downstream. At the Clubbidean dam, a 17 m embankment in central Scotland built in 1850, a subsidence in 1964 was related to the solution of bedrock causing the creation of cavities in calcareous sandstones. The leakage was intercepted by a new grout curtain, the seepage loss being reduced from 1×10^6 litres/day to 1 % of this figure (Sivasubramaniam and Carter, 1969). This single example provides one illustration of many dams that, if full knowledge were available, would require remedial treatment to bring them up to modern standards.

FOUNDATIONS ON MESOZOIC AND TERTIARY CLAYS

Stiff overconsolidated, uniform clays of marine or lacustrine origin form a characteristic part of the Mesozoic and Tertiary sequence in south-east England; the liquid limits of such clays characteristically range from 45-90 %. Enbankment dams have been constructed, in particular, on the Lias and Oxford Clays of the Jurassic, the Weald and Wadhurst Clays of the Cretaceous and the Tertiary London Clay. In general terms, the construction of embankment dams from clay on such foundations presents no insuperable difficulties providing the embankments slopes are at a sufficiently flat angle and appropriate drainage measures are incorporated in the design. The clay foundation to the dam provides a natural blanket preventing or reducing seepage below the dam. However, if the foundation is plastic then excessive deformation could develop. Most of the dams of this type which have been constructed are relatively low, with maximum heights of about 20-25 m. However, the recently completed Empingham dam is 40 m in height and new questions arose as to the scale of the dam relative to the foundation properties; this site is to be discussed at length.

The most common problem in dam sites on these clays is that weathering has resulted in a reduction of strength of the near-surface material. For exemple, the undrained shear strength of the Oxford Clay at the Marmoor reservoir, near Oxford was reduced to 30 kN/m^2, about 25 % of the average value. Apart from such effects of softening, valley bulging and the associated structures such as flat shears are common and such situations can give rise to either a general reduction in strength or the development of localised principle shear surfaces at the residual strength. In the case of the Hollowell dam (Terzaghi, 1948) a well-developed series of folds was identified during excavation of the cutoff. This dam slipped after construction and this may well have been a response to internal shearing of the clay. The strength of these clays is extremely sensitive to moisture content and this can give rise to questions of design and problems during construction. Careful moisture content control is required during construction and drainage blankets are normally included to ensure adequate dissipation of pore water pressures. Where possible, locally won sandstone or granular alluvium is incorporated either as blankets or shoulder material. If the dam is to be sited on thick cohesive alluvium or softened clays, sand drains are normally installed early in construction to accelerate the gain of strength of the foundations with consolidation. Broad cutoffs, cutoff trenches and diaphragm walls have all been used to reduce or prevent seepage.

The Empingham Reservoir Project is a pumped storage scheme due for completion in 1974 which will involve the extraction of water during peak flow from the Rivers Welland and Nene, in eastern England, and then pumping through a tunnel into the reservoir. The yield of the scheme will be about 300 Ml/day (65 million gallons per day) and should provide for increased demand in this region until the late 1980's. The complete reservoir will be very large forming the second largest sheet of inland water in England and Wales. The powers were obtained in 1970 and construction then began with the driving of a Diversion Tunnel to take the flow of the River Gwash during the construction period.

The site of the dam which is 40 m in height is underlain by rocks of Jurassic age with a thin cover of glacial drift and some valley alluvium; the sequence present is as follows (fig. 7):—

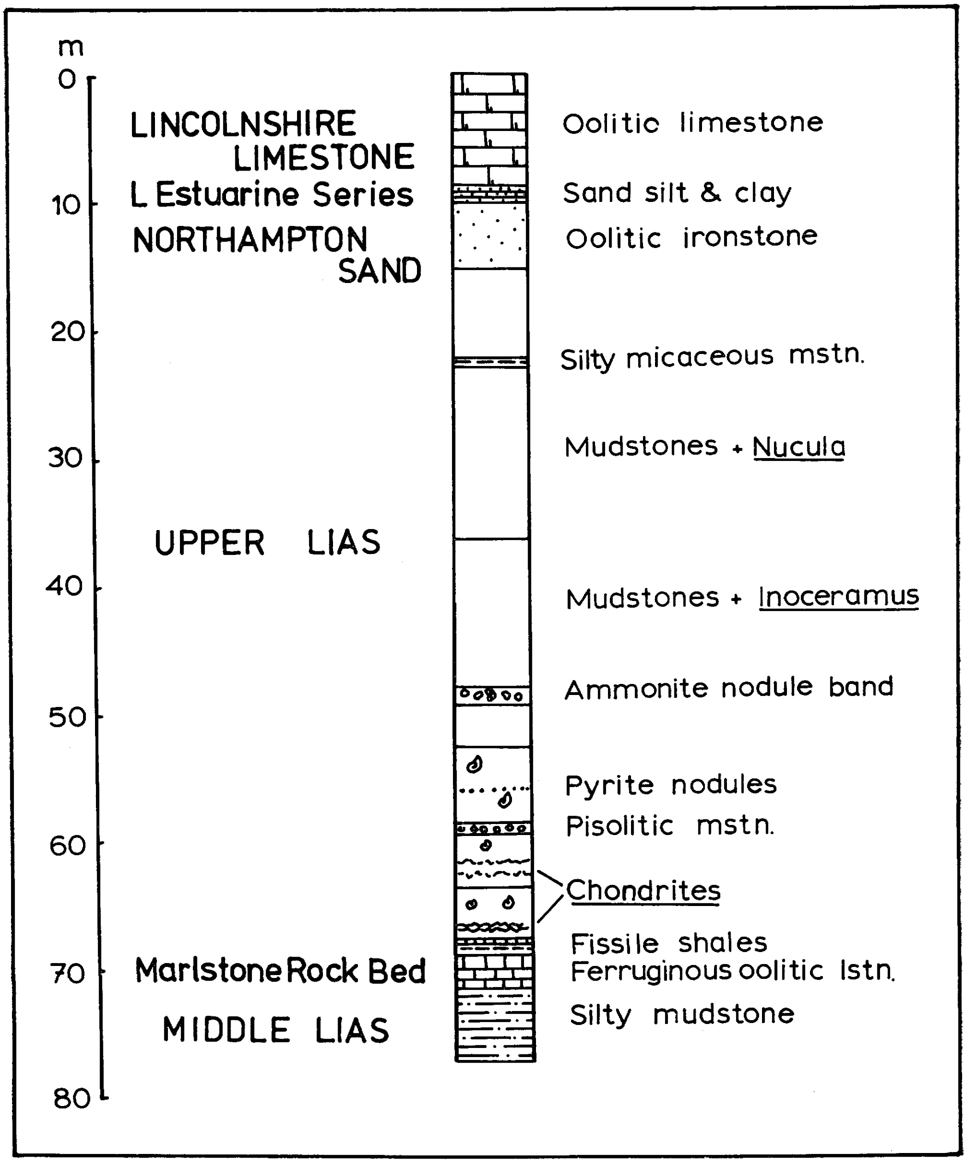

FIG. 7. — *Geological sequence at Empingham Dam.*

Lower Lincolnshire Limestone: up to 12 m encountered
Lower Estuarine Series: 0.5 to 2.5 m
Northampton Sand: 4.5 to 7 m
Upper Lias Clay: up to 70 m
Marlstone Rock: 0.5 to 2 m
Lower and Middle Lias Silts and Clays: 25 m+

The Upper Lias Clay outcrops in the valley floor and the overlying materials are cambered over the valley sides resulting in the formation of gulls and dip-and-fault structures. The Lias Clay is overlain by up to 4 m of soft alluvium together with some colluvial brown silty clay with fragments of ironstone and limestone. The valley bottom has been subjected to bulging but the Marlstone Rock Bed (a ferruginous limestone and calcareous limestone) which is 27 m below the original valley floor only rises by about 2 m above this elevation. The main bulge is a single fold, with a width of 40 m and an amplitude of about 20 m, associated with steep reverse faults. It was not until the formation was being stripped that the details of the bulge were revealed. Two marker beds, the Ammonite Beds and the Pisolitic Band, were identified occurring in undisturbed sequences at 10 and 20 m respectively above the top of the Marlstone. Detailed investigation revealed that the bulge had occurred above the Marlstone and marked thickening of the Upper Lias Clay has occurred in the centre of the valley. A detailed micro-palaeontological study of boreholes has confirmed the litho-stratigraphic sub-division of the Upper Lias Clay. In addition this study has shown that individual micro-fossil defined bands thin consistently towards the valley centre. For example, a sequence of Upper Lias Clay 46 m in thickness below the valley sides thins to 37 m below the valley floor over a horizontal distance of 440 m (fig. 8). At the same time, this study has confirmed the very marked thickening of the lowest micro-palaeontological faunules within the valley bulge. Extensive shearing and frost shattering has influenced the structure of the Lias Clay. The mechanism by which the sequence on the valley side became thinned and the bulge created is not understood mechanically. Various factors may have contributed to the situation:— Freeze-and-thaw of the clays during and after glaciation, differential loading during and after glaciation, artesian pressures in the Marlstone Rock Bed and swelling of the clays. The only satisfactory explanation mechanically is that high normal load on the valley sides and hill tops during glaciation was compensated by active movement in the valleys floor where the normal loads were less. It is possible that there is a "passive" double wedge in the Lias clays above the Marlstone Rock Bed arising as consequence of sub-glacial drainage causing valley bottom unloading.

These observations of mass structure together with recognisable shearing in the clays give rise to a fundamental problem with regard to the assessment of foundation strength. The dam will be about 36 m in maximum height above its foundation and in consequence some care has been adopted in investigating the mechanical properties of the foundation materials. Test data is a follows:

$LL = 53\ \%$ average, 82 maximum, $PL = 24\ \%$ average, natural moisture content = 15-25 % at depth, activity = *c.* 0.7.

$C_U = 35\text{-}140\ \text{kN/m}^2$ in upper 5 m and $105\text{-}280\ \text{kN/m}^2$ below this level.

$\phi_p' = 22.5^\circ$, $\phi_r' = 11^\circ$.

In order that the dam could be satisfactorily designed, it was necessary to know both the strength of the foundation and the likely pore water distribution.

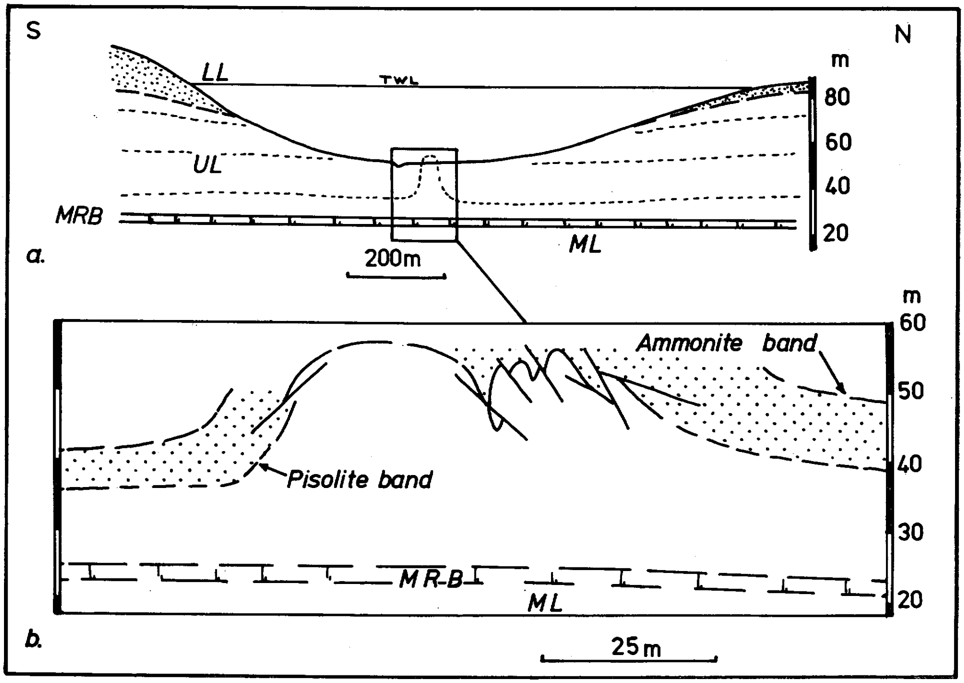

FIG. 8. — *Longitudinal section of Empingham Dam (a) with true-to-scale enlargement (b) of detail of valley bulge. LL, rocks above Upper Lias; UL, Upper Lias; MRB, Marlstone Rock Bed; ML, Middle Lias.*

Clearly reliance could not be placed on peak strength as the foundation rocks show evidence of both shear and there is evidence of larger scale movements. On the other hand the structural, stratigraphic and palaeontological studies show there is no evidence for a single continuous shear plane at the residual strength which would justify adopting the most pessimistic solution. There will be pore water drainage from the Marlstone Rock Bed below the dam into the lower parts of the Upper Lias Clay. Sand drains have been installed immediately below the dam to maximum depths of about 18 m to assist consolidation of the Clay (Plate 6). This will still leave a central zone which will have to rely on natural drainage properties of the clay (2×10^{-9} cm/sec in laboratory tests). The design of the dam provides very flat shoulders to the upper part of the structure (1:5) and very much flatter toes to cope with the relatively low foundation strengths. Relatively deep cutoffs have been provided below lower dams in the same general geological situation as Empingham, in view of the disturbed condition of the clays within the valley bulge. Observations have demonstrated that the permeability of the clays within the valley bulge can be very high and may approach 1 cm/sec locally. On the other hand, loading by the dam will tend to consolidate the clay and so reduce the in situ permeability. There will be a broad shallow cutoff provided at Empingham but no special measures have been anticipated. On the right bank the camber will bring down the permeable materials above the water of the Upper Lias Clays to an elevation underneath that of the top water level.

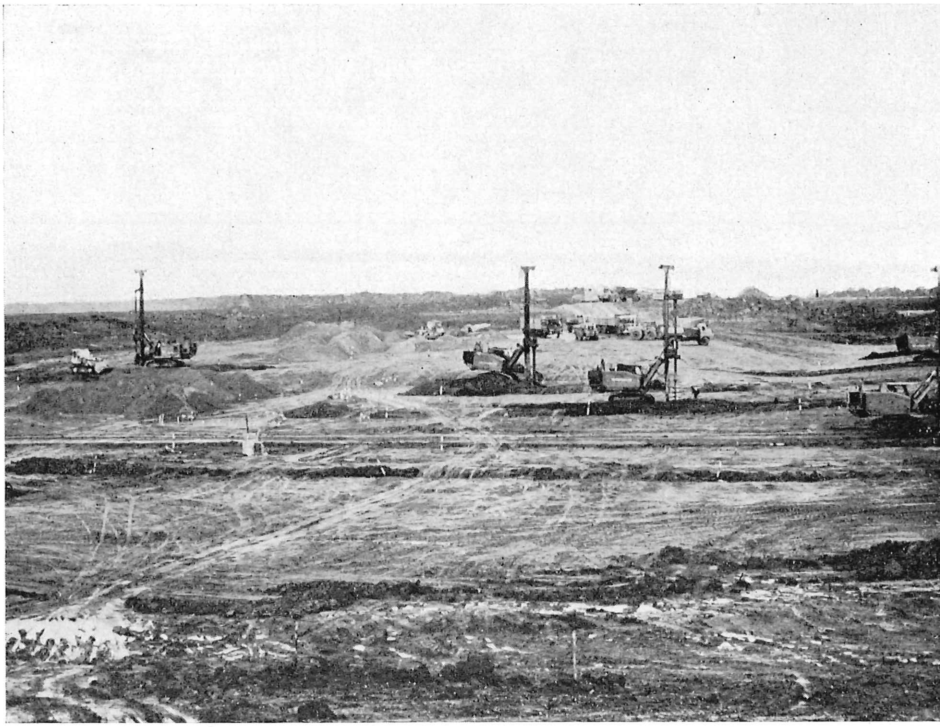

PLATE 6. — *Installation of sand drains in Upper Lias Clay foundation at Empingham.*

It is intended to provide a physical cutoff in this region extending to such a point as the level of the Lias rises above the reservoir level (Plate 7).

The sequence dips gently eastwards so that the Marlstone Rock Bed rises into the reservoir floor at the head of the valley. This horizon is relatively thin and only of value locally as an aquifer. It has been recognised that the groundwater in the Rock Bed is artesian at the dam site but in situ tests revealed a relatively low permeability (10^{-5} cm/sec) correlating well with the appearance of the massive cores obtained. However, two shafts excavated downstream of the dam for the start of tunnelling operations led to a blow-in of the clay plug left in the shaft floors. Dewatering was carried out and a yield of 20 Ml/day was obtained from eighteen boreholes drilled into the Marlstone. This yield was unprecedented in relation to the formation. The piezometric head in the Marlstone is very uniform over the area investigated and it is clear that a large area is being dewatered by the current pumping operations. This would suggest a permeability of about 1 cm/sec for the Marlstone in the valley bulged area. The final design of the tunnel lines and levels had been planned deliberately to avoid the Marlstone in order to prevent this eventuality. It appears possible that disturbance to the Marlstone in the valley floor has caused fracturing and so a significant flow towards the wells; in this case, the water is being drained from the area upstream of the dam rather than catchments to the north and south. An additional factor arises with regard to the influence that this situation can have on the

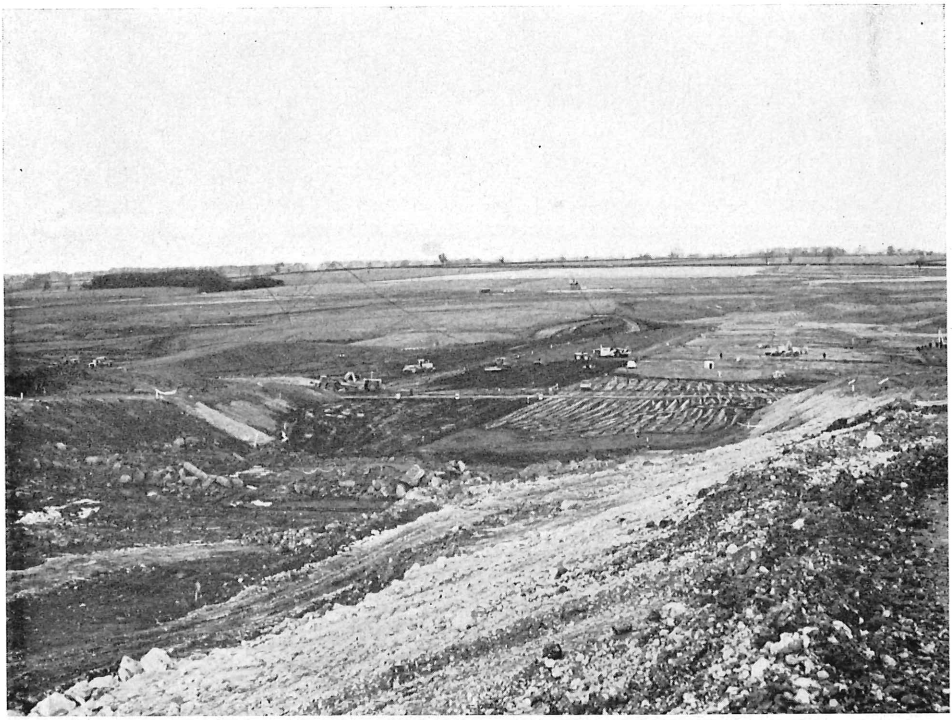

PLATE 7. — *View of Empingham dam during construction from broad cutoff trench on right bank.*

reservoir which will flood over the Marlstone outcrops. There are only very limited outcrops, separated from the reservoir area by faults, of the Marlstone down-dip towards the east and so there is no obvious drain which will allow water to flow from the reservoir into the Marlstone. The water being drained at present has a high iron and H_2S content.

The total fill in the embankment is estimated to be about 4.5 million cubic metres which will be virtually entirely won from borrow pits in the Upper Lias Clay in the reservoir area. There is a sloping upstream core with an undrained shear strength of 60 to 90 kN/m^2. This clay is excavated from the upper, more weathered parts of the borrow pits and then artificially watered. The fill downstream of the core has a higher undrained shear strength with a minimum value of 90 kN/m^2 and has been excavated from the drier, deeper parts of the borrow areas. The shoulders are composed of weaker materials including head, weathered clay and material obtained from site stripping and general excavation.

FOUNDATIONS ON GLACIAL SEDIMENTS

Most of the United Kingdom was covered by ice at some stage during the Pleistocene glaciation and the previous examples have illustrated the influence of glacial and periglacial action on bedrock. Inevitably the dams constructed wholly or partly on glacial sediments are embankments themselves formed from local sources of material of glacial origin. It is common practice for a central cutoff to be extended down from the dam core through the potentially variable glacial sequence into bedrock; if the overburden cover is sufficiently thin, then the whole core is founded upon bedrock. Where the glacial materials, usually represented by stiff till, have adequate strength, the embankment shoulders are founded directly on the till after removal of superficial and softened materials.

The Llyn Celyn dam (Crann, 1968) provides a typical example of a dam constructed in the mountainous, glaciated terrain of North Wales. The reservoir is impounded by a 45 m high embankment and has a yield of 290 Ml/day. The bedrock is composed of Ordovician volcanic rocks with near surface fracturing presumably induced by glaciation. The rock is covered by some 6-8 m of glacial till which, at a depth of about 2 m, was a stiff clay with gravel, cobbles and boulders suitable as a foundation. Local areas of sandy gravel and laminated blue silts were exposed, the latter being mainly excavated prior to placement of fill. In one case, beneath the upstream central section of the toe berm, sand drains were installed through the laminated silts. The rolled clay core was formed from till and the gravel shoulders won from a glacio-fluvial deposit within the reservoir basin. The rock was exposed to the width of the core and a cutoff cons'ructed was composed of a concrete grout cap, surface treatment of the rock with dental concrete and both curtain and consolidation grouting. The complete rock surface was covered with concrete or cement slurry to ensure a tight contact between the foundation and the overlying clay. Fifty five pressure relief wells were drilled to a depth of up to 11 m at the dam toe. Other dams built at a similar time as Llyn Celyn have relied upon a central cutoff in rock supplemented by grouting. For example, the West Water dam, in the Southern Uplands of Scotland, has a concrete cut-off with a maximum depth of 21 m with a grouted cutoff below with an additional average depth of 23 m (Cuthbertson, 1966).

At those sites where the glacial till is weathered or softened, the strength is commonly inadequate to support the weight of the embankment. Relatively limited volumes of such material can be excavated but deep or extensive layers of soft clays are normally treated by means of sand drains to provide for drainage leading to an increase in strength. At the Selset dam (Kennard and Kennard, 1962) an area of about 24 000 m^2 of soft clay with a maximum thickness of 9 m was identified in the valley floor. This soft clay with an average undrained shear strength of 28 kN/m^2 and moisture content of 18 % passed down into stiff boulder clay with equivalent properties of 210 kN/m^2 and 12.5 % respectively. The origin of these remoulded, softened clays is not known for certainty but they may represent the products of mass movement processes during valley formation. The required undrained shear strength in the foundation at the end of construction was 160 kN/m^2 and it was in consequence clear that either the material would have to be removed or it would be necessary to rely on a substantial increase in strength with consolidation. In view of the low coefficient of consolidation (about 1 m^2/year) the only feasible method of improving the strength in situ would be to rely on sand drains to dissipate the high excess pore pressures anticipated during construction. There would have been practical difficulties in excavating the soft clay in view of high artesian water pressures in the underlying rock, and the river diversion programme. It was decided to install about 4 000 drains at 0,45 diam. and 3 m centres; the drains were backfilled with crushed aggregate.

The response of the foundation to the sand drains during construction exceeded expectations and the calculated factor of safety for the end of construction, based on the most critical slip circle through the sand drains, was 2.4.

A similar problem was encountered at the Cow Green dam which is a composite structure consisting of a concrete gravity section 27 m in maximum height and an embankment 215 m in length. This choice of design was determined by the presence of a buried channel 45 m in maximum depth and infilled with till, on the right bank of the River Tees; the concrete dam is founded on dolerite and the embankment on till. Under the concrete dam and under the first part of the earth dam, until there was a cover of 15 m of till, grouting of the rock foundation was carried out to a depth of about 15 m below the cutoff trench. Cement-PFA-bentonite grout was used in the top 4 m and in areas of high permeability, and in other areas a resin chemical grout was used; the amount of grout injected was relatively low. The earth embankment comprises a rolled clay central core, which is 24 m wide at its lowest level and 7 m wide near the top of the dam, with filters on the downstream side. Relatively little increase in shear strength with depth was found in the till. Values of C_u of less than 100 kN/m^2 were found at depths of 21 m. In consequence, the final design incorporated 150 mm diam. sand drains up to 24 m in depth and at 2.3 m centres.

The major problems associated with glacial sediments, however, are those related to the occurrence of deep pre-glacial channels now infilled by thick successions involving glacial lake and outwash deposits. There are three cases of particular note, the Derwent dam (Ruffle, 1965, 1970), the Backwater dam (Scrimgeour and Rocke, 1966) and the Cod Beck dam, each of which has been associated with slightly different aspects of the control of groundwater flow through deep buried channels.

The Derwent dam is an embankment 36 m in height and 900 m in length located on one of the tributaries to the River Tyne in north-east England. The valley is underlain by a thick sequence of glacial lake deposits and three alternative sites were considered in 1954 for the location of the dam. The bedrock below the buried channel is composed of the Middle Carboniferous sandstone-shale alternation. The channel has a maximum identified depth of 54 m and the most complete sequence, with approximate thicknesses, consists of:—

3 m	Sandy gravel	Alluvium
3 m	Sandy clay with stones	Slopewash
4 m	Silty sands and silt	
3 m	Till	
14 m	Laminated clay	Glacial lake deposits
3 m	Sand and gravel (Upper aquifer)	
4 m	Silt	
7 m	Varved Clay	
3 m	Sand and gravel with local till (Lower aquifer)	
10 m	Till	
	Bedrock	

Further investigations were initiated in 1957 and it was recognised that there were two main foundation questions at the site:

(i) the occurrence of a layer of laminated clay with a minimum undrained shear strength of about 50 kN/m^2, and

(ii) the existence of thick, permeable layers of sand and gravel containing artesian ground water within the lake deposits.

The original design for the cutoff involved a 2 m wide trench excavated through the channel into rock. It was recognised that a requirement of safely excavating such a trench would be the adequate dewatering of the glacial sequence. To this end, a pumping test and additional exploration were carried out early in construction. This investigation revealed that the glacial sequence was more orderly than had been previously recognised and that there were three aquifers, one in the rock and two in the overlying glacial sands and gravels (fig. 9). The

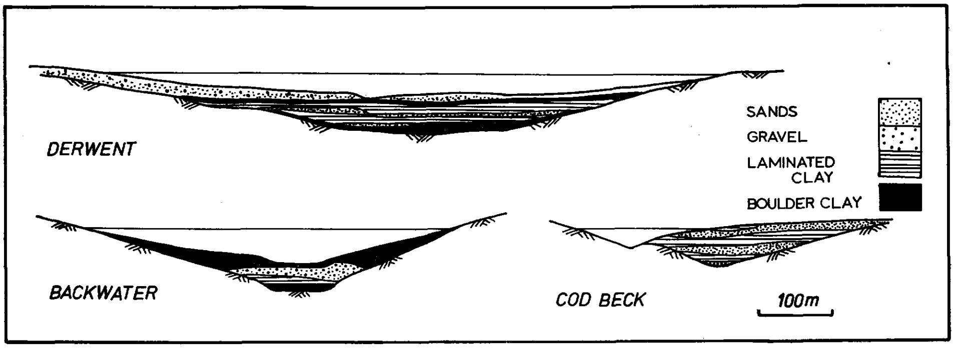

FIG. 9. — *Longitudinal sections through the buried channels at the Derwent, Backwater and Cod Beck Dams viewed from downstream.*

pumping test demonstrated that the uppermost of these aquifers was not in significant contact with the rock aquifer. It also demonstrated that groundwater loss due to seepage would be of the order of 1.7 Ml/day, through the upper aquifer ($k = 10^{-2}$ cm/sec). In these circumstances, quite apart from the continuing uncertainties as to the likely degree of pore pressure dissipation from the laminated clays during excavation of the trench, there was cause to re-examine the question of the need for a complete cutoff. The new investigations had demonstrated the layer of laminated clay to be continuous and so this could be relief upon as a natural upstream blanket. The design was accordingly modified to provide for a partial cutoff into the upper till and laminated clay joined to the vertical core by a horizontal blanket forming the base of the dam. Drainage wells were, inevitably, required downstream of the dam to cope with ground water flow in the rock and upper aquifers. There was an extensive programme of foundation treatment involving the construction of 4 475 sand drains to the base of the laminated clay horizon (average geotechnical properties:— PL = 22 %, LL = 59 %, m = 24 %, % < 2 μ = 50).

The Backwater dam is located in the southern part of the Scottish Highlands being an embankment 42 m in height. The maximum depth to rockhead in the valley floor is 49 m, the quartzitic grit bedrock being exposed at top water level on the two flanks of the dam (fig. 9). The site is underlain by a layer of "mountain" till, a well-graded gravelly bouldery till, which rests on top of a complex of sands and gravels with some thin silty layers. The base of the buried channel is infilled by a thin stratum of silty lake deposits resting on top of a basal till. The initial boreholes identified artesian pressure of up to 12 m in the valley floor. A pumping test demonstrated that it would be impracticable to construct a traditional cutoff trench through the buried channel and consideration was given to other alternatives which rapidly resolved themselves into the practicability of a grouted cutoff. In the event, it was decided after a trial to construct the cutoff by forming a five-line grout curtain. The inner three holes were drilled to bedrock, the central line being extended into "unweathered" rock. In practice it was established that the depth of fracturing was considerable and this had been partly anticipated by the results of a seismic survey which had over-predicted the depth of the buried valley by 60-70 m. The outer lines of holes were drilled 15 m into the glacial deposits and were intended to provide outer curtains for restricting grout flow. The order of grouting, by means of the tubes-a-manchettes method involved clay-cement, deflocculated bentonite and finally silicate. After the trial grouting a shaft was sunk into the grouted block and the inflow into the shaft was about 4.5 litres/min. The measured permeability was 1×10^{-5} cm/sec and this figure was adopted as the basic criterion for the permeability of completed cutoff; a 25 % voids infilling was allowed for. The drilling for the cutoff was carried out from an embankment in the central part of the valley in order to cope with the artesian groundwater. After construction of the curtain, a broad core trench was excavated and a core of selected cohesive till placed being supported by shoulders of the mountain till. The groundwater pressure difference across the grouted cutoff is 6-9 m out of a total head across the dam of about 42 m. In such a case the influence of the curtain in the uplift pressure will be small and there will only be a 20 % reduction in seepage. The estimated seepage with the curtain in place is about 1 Ml/day so the consequential savings are of the order of 0.2 Ml/day. Separate calculations, based on the in situ permeability tests would suggest seepage figures of the same order. It is clear that the mountain till forms an effective, although partial blanket upstream of the dam as is demonstrated by the existence of artesian water pressures.

Both the cases of Derwent and Backwater call into uncertainty the criteria upon which rational decisions as to cutoff design are based. In both these cases a complete cutoff, sealing off the whole buried channel, cannot justify itself financially in relation to the value of the water retained. However, cutoffs have additional objectives in the protection of the dam foundations and the reduction of uplift pressures. Even so, the use of partial cutoffs, with proper drainage measures, would appear to be both economically and technically more desirable.

The Cod Beck dam, the last of the three dams on buried channels to be considered, was constructed in the period 1949-1953 and impounded in 1953; the embankment is a conventional structure 30 m in height. During construction it was established that the left flank of the dam was located at the edge of a pre-glacial channel and the dam was terminated in a partial cutoff on that side (fig. 9). The Cod Beck valley contains several degraded slips, some of which contain evidence of recent activity both in the Lias shale bedrock and the glacial sediments within which the present river channel has been formed. During impounding and before filling was completed, an old slip downstream of the dam underlain by the glacial deposits was reactivated (Plate 8). Active movement occurred for the next two years and drains were installed which proved successful until they

PLATE 8. — *View of slips in glacial lake deposits in buried channel immediately downstream of the left bank of the Cod Beck dam.*

deteriorated and further movements occurred in 1961. The movements were eventually arrested in 1965 at a much flatter slope than existed prior to impounding. Further investigations were carried out in 1970 and these were supplemented by the installation of additional drains.

The sequence of glacial sediments consists of two main layers of laminated clays interbedded with horizons of sands, silts and some clay layers; there is a basal gravel layer which presumably represents a previous channel filling. The initial slip involved an essentially rotational movement in the upper part merging into a uniform flow of saturated debris over the lower parts of the slope. The slope was trimmed back and both vertical, pumped drain wells and horizontal drain holes installed. This scheme was working effectively in 1956 but with time there was a progressive reduction in yield from the drains which is presumably a reflection of progressive silting. Movements where then re-initiated resulting in a deep-seated slip in 1961. This sequence is characteristic of slope instability induced and controlled by seepage pressures. The permeable zones within the buried channel provide ready access of natural ground water and reservoir seepage into the slipped area and this has been demonstrated by piezometric observations. There is some evidence to indicate that the water pressures in both the rock and lower sand horizon have risen significantly since impounding. Observations have indicated that particularly high ground water pressures exist where the instability has been most active. The total quantities of flow have not been great and their loss has not influenced the viability of the reservoir project. The example tends to add weight to the point made earlier that partial cutoffs with appropriate drainage are more than adequate for the control or groundwater flow in buried channels filled with such glacial sediments.

CONCLUSION

The varied geological structure of the United Kingdom, together with the changing approach towards dam design and construction, and water needs and usage, has contributed to an unique national record of experience in relation to the engineering geology of dam foundations. The future water requirements will involve a doubling of the volume of water storage by 2 000 and this will inevitably lead to the construction of more inland reservoirs, together with eventual estuarial barrages. In general, therefore, the size of projects will tend to increase with a consequential increase in the scale of the geological and geotechnical problems.

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