Ten thousand anchorages in rock

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Introduction

An anchor construction and design manual of the late 1960s opened its rock anchors section with the firm pronouncement: 'Anchors installed in wet or weak rocks such as marls, mudstones, shales and sandstone, and all anchors requiring loads of greater than 50 tons working capacity, should be constructed with underreamed enlargements. The enlargements, filled with grout, form a positive lock even in soft, weak or wet strata.'

A generation earlier Andre Coyne had also considered the formation of enlargements beneficial in 1934, when designing and constructing in Algeria what were probably the world's first high capacity rock anchorages. These


provided vertical prestress through Cheurfas dam into its rock foundations (Fig 1).

But no such enlargements were considered necessary in 1987, at Carron dam in Scotland where prestressed rock anchorages were required to have working loads of 1600kN to 3400kN to allow an increase in crest level (pic 1). Tested up to 5100kN, they were designed and installed in straight shaft bores. This change was supported by recent trends in rock anchor technology which have quoted working stresses carried a number of essential safety nets. Already in the early 1970s, proof loading systems had been introduced for each and every working anchorage such that each one demonstrated an inbuilt factor of safety ranging from 1.1 to 1.5 on the required working load. The proof loading was considered a necessary bonus in order to generate confidence in the then novel technique of anchoring. This safeguard was further enhanced in 1982 by the publication of DD81, which laid down even more stringent requirements for testing of both permanent and temporary anchorages. It stipulated two or three repetitive load cycles (Fig 2) and observation of creep losses over periods up to 10 days along with detailed criteria for anchorage acceptance.

The recommendations were also extended to cover the installation and testing of preliminary proving anchorages to confirm adequacy of the system in unproven strata. It was expected that a substantial body of information on ultimate rock/grout bond stress would be generated, supplementing the original data published in the state of the art

Fig. 1. Rock anchorages at Cheurfas dam enhancing the anchorage capacity by use of 'enlargements' (1934).

Undoubtedly, this design approach was strongly influenced by the 1977 publication of Rock anchors - state of the art by Littlejohn and Bruce3. This review presented extensive tabulated data, drawn from worldwide sources, summarising rock/grout bond stresses 'recommended for design' and 'employed in practice'. Design guidelines were given for straight shafted rock anchorages founded in weak to strong rocks from the argillaceous to the arenaceous grading, and in sedimentary, metamorphic and igneous formations.

However, this apparent free licence simply to design anchorages based on

Fig. 2. DD81 recommended that production anchorages are subjected to two or three repetitive load cycles to 1.25Ttw (temporary anchorage) or 1.5Ttw (permanent anchorage).

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of the rock is more compatible with that of the anchor tendon. Borehole diameters in the fixed length have generally ranged from 87mm to 127mm, where overburden drilling involving casing has been required, and 102mm to 204mm where direct access to bedrock allowed open hole drilling.

Where underreaming techniques have been employed, a system allowing the simultaneous opening of two axially spaced flights has cut two enlargements during continual flushing operations (pic 3). Enlargement diameters from the rock underreaming system have been up to 300mm formed from a 105mm borehole (Fig 3). Clay underreaming, initially developed to produce pairs of 400mm diameter enlargements from a 105mm bore in marls, has been continually refined and improved to allow the simultaneous formation of up to five 700mm diameter enlargements from a 180mm bore.

**Drilling and flushing methods**

In the rather infrequent cases where bedrock has been exposed and allowed direct open hole drilling, down the hole hammer air flush techniques have generally been employed. Operators have developed a preference for button as opposed to cross bits for down the hole hammer work although DTHH manufacturers are always forthcoming.

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**Pic. 2. All specialists have implemented considerable improvement in safety standards since 1972.**

presentation and analysis of the data in this paper. The information has been collected from anchorages installed and tested by Universal Anchorage Company (1970-78), Universal Anchorage Contractors (1978-1983), Colcrete (1983-1986) and GKN Colcrete (1986-1988).

Drilling, installation, and grouting techniques were developed throughout by these firms but may not be consistent with those which have been employed in similar conditions by other specialists (pic 2).

**Simple approach to evaluation of data**

Although it is known that the load transfer mechanism from a steel tendon through the cementitious borehole grout to the rock stratum is complex, the simple approach originally derived for estimating pile capacity is still universally adopted for the design of rock anchorages:

\[
T_f = \frac{C}{D} L
\]

where

- \( T_f \) = estimated effective load per unit length
- \( D \) = bore diameter in fixed length
- \( L \) = fixed length
- \( C \) = ultimate rock/grout bond stress.

The derivation of the formula is based on the following assumptions:

- Transfer of the load from the fixed anchor to the rock occurs by a uniformly distributed stress acting over the whole of the curved perimeter of the fixed anchor.
- The diameter of the borehole and the fixed anchor are identical.
- Failure takes place by sliding at the rock/grout interface (smooth borehole), or by shearing adjacent to the rock/grout interface in the weaker medium (rough borehole).
- There are no discontinuities or inherent weakness planes along which failure can be induced.
- There is no local debonding at the rock/grout interface.

The majority of bond stresses presented in publications have been established from test anchorage results using this approach and adjusted to:

\[
T = \frac{T_f}{D.L.}
\]

For consistency, evaluation of all bond stress values tabulated in this paper is on the same basis.

**Fixed anchor dimensions**

In the vast majority of cases minimum fixed anchor lengths in rock are approximately 4.5m. Only in a limited number of situations have shorter fixed lengths been installed in order to ensure anchorage failure and established values of ultimate rock/grout bond stress.

Maximum fixed anchor lengths of 10m to 12m have been considered useful only in particularly weak rocks such as mudstones or shales, where the elasticity of the rock is more compatible with that of the anchor tendon. Borehole diameters in the fixed length have generally ranged from 87mm to 127mm, where overburden drilling involving casing has been required, and 102mm to 204mm where direct access to bedrock allowed open hole drilling.

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**Pic. 3. The rock underreaming tool.**
with bit recommendations for each rock stratum. Rotary drilling using a rock roller bit with air or water flush have been chosen where there was a risk of borehole collapse and loss of the DTHH. But the majority of anchorages required penetration of overburden, necessitating use of drill casing and water flush. On reaching hard bedrock, rotary drilling with rock rollers has been most common. In weak rocks such as mudstone, shales and chalk, drag bits have sometimes been

**Borehole alignment**

In the majority of anchorages, the effect of deviation by 1° or 2° from the intended line would rarely be detrimental to either the structure or the adjacent anchorages. However, there are occasions when only a limited deviation can be tolerated: In very closely spaced multi row anchorages; in areas of ground services and tunnels; and when initial drilling is within existing structures. Two situations have recently arisen where alignment checks on a number, or all of the bores, were specified.

In the first, a 35° inclined preliminary anchorage was being installed through a guide duct in a reinforced concrete diaphragm wall. Some 20m of rotary percussive overburden casing was needed through clays and sands, followed by 10m of sandstone bedrock penetration. Reflex Fotobor equipment was advanced down the hole in 3m increments for alignment checking. Alignment of the 6m length beyond each Fotobor location was photo recorded at preset time intervals. The 3m overlap in the recording system allowed computation of the alignment of the entire bore and presentation in print out form (Fig 4).

Deviations of 0.3° and 0.5° were recorded at the end of the 30m bore in the horizontal and vertical planes respectively. Deviation over the cased length in the overburden was almost negligible, up to 8mm and the majority of the deviation occurred with the openhole bedrock drilling. This bore alignment proved to be considerably better than would normally be envisaged for an inclined bore in the prevailing conditions.

In the second instance when detailed checks were specified, near vertical anchorages were being installed to allow an increase in crest level of an existing gravity dam. Alignment checks were specified on all anchorage bores prior to tendon installation. Boreholes which deviated would have penetrated the outer dam wall or decreased the effectiveness of the anchorage force in resisting overturning or sliding. Drilling started with maximum accuracy by checking verticality with a drill orientated device (Matheson 1984)°. DTHH were employed for the full depth to give maximum accuracy. On completion, each bore was checked for alignment using a servo accelerometer type inclinometer (Fig 5). Only 30% of the bores exhibited deviation worse than 1 in 100 the worst being 1 in 56 (Fig 6) in an area of particularly weathered rock where the inflow of water was high. But 60% of the holes deviated to a maximum within the bore length then closed back on the intended line towards the bottom (Fig 6). Clearly this is more likely to occur in vertical or off vertical boreholes than in inclined boreholes. It was not possible to identify any operator controlled actions which influenced the deviation. However, since the greatest deviations were generally observed in the areas with the highest degree of weathering it appears that deviation was

![Fig. 3. Underreamed enlargements from the 'rock' underreaming system (mm).](image)

![Pic. 4. Difficult overburden drilling through scree in 1986 using rotary percussion casing equipment mounted on a lightweight mast-off drilling system.](image)
related largely to the bedrock conditions and prevailing fissure and bedding planes.

This data indicates that reasonable drilling accuracy can be achieved if attention is paid to control. But it is important to note that had the boreholes deviated beyond the acceptable limit due to prevailing ground conditions, or for other reasons, then no remedial measures such as grouting or redrilling at the same location could have guaranteed a conforming solution. Thus it is important that tolerances in difficult ground should not just maintain a norm but should be no more stringent than necessary for an acceptable engineering solution.

Methods of ensuring grout tightness of the fixed anchor length

In the early days of anchoring some problems were encountered in rock anchorages due to leakage or washout of cement grout from the fixed anchor length. This resulted in failure during testing or, in instances where such partial grouting of the fixed length could not be detected, it lead to a high risk of corrosion failure of the exposed, unprotected steel tendon. In the late 1960s and early 1970s cement grout alone was often considered to be adequate for corrosion protection and use of corrugated plastic ducts and encapsulations was at an infant stage of development in the UK.

In 1975 Littlejohn published Acceptable water flows for rock anchor grouting. Calculations were based on the permeability of a single fissure of adequate width (160 microns) to allow the penetration of normal size cement grout particles at low pressure. It was realised that a multitude of narrow fissures, each impermeable to cement grout particles, could also exhibit the same or higher water losses and that it would be possible to identify such a situation from flow characteristics over each section of a bore. But an ultra safe water loss criterion was recommended in order that serious misinterpretation of data should not occur. Water flow rate from a borehole of 3 litres/min/atmosphere pressure was considered permissible.

Fig. 4. Borehole alignment data monitored by reflex Fotobar equipment presented in printout form.
instances the attainment of a steady level of cement grout was observed in boreholes which had repeatedly failed the water test. This simple operation proved the boreholes to be grout tight and confirmed that water loss in excess of the permissible amount could result from the presence of a network of fissures, too fine to allow passage of cement particles.

Thus, an alternative method of testing the tightness of the bore was evolved using cement grout as fluid in lieu of water and was approved in DD81. However, the approach likely to be taken in the new British Standard for ground anchorages (BS 8081), due for publication late this year, will be somewhat more flexible. Other alternatives will be permitted since 'on completion of grouting, it is necessary to ensure that the loss of grout over the fixed anchor length is insignificant'. In current practice this can be achieved by water testing, pregrouting, by grout testing of the borehole over the fixed anchor length, or by the use of pressure injection techniques during routine grouting of the anchorage. Pressure injection, generally applied during a single phase grouting operation, is reasonably consistent with the method introduced in the 1960s for constructing pressure grouted anchorages in sands and gravels.

In weak, weathered bedrocks it is used primarily to seal the ground immediately adjacent to the fixed anchor, thereby limiting the loss of grout and preventing grout dilution by water inflow. It also verifies the grout tightness of the bore, by the attainment of a back pressure. Ground immediately adjacent to the fixed anchor is strengthened and so anchorage capacity is enhanced. This pressure grouting system is frequently proving to be economic in reducing the number of operations and programme time, in addition to providing an increase in capacity. There are, however, still situations when water testing or grout testing are more suitable.

**Grout and grout mixing**

Over two decades of anchor construction, grout mixing equipment and water/cement ratios have changed considerably. Until the early 1970s, grout was mixed in high speed paddle mixers complete with baffle plates and slotted blades to induce high shearing action and 'efficient' mixing. Water/cement ratios, by weight, ranged from 0.5 to 0.6, and 28 day strengths from 20 N/mm² to 30 N/mm². Present day machinery induces a high speed colloidal mixing action, with greatly increased efficiency in cement particle hydration. Water/cement ratios from 0.45 to 0.5 result in 28 day strength of 40 N/mm² to 70 N/mm². While this improved technique has resulted in a noticeable increase in strength of the anchor grout, its effect on rock/grout bond capacity in the same type of rock has not been found to be of similar significance. Research work involving grouts of water/cement ratio 0.45 and 0.6 in moderately strong bedrock gave no indication of significant variation in ultimate rock bond capacity. But a noticeable increase in permanent displacement was detected at the tendon/grout interface when the higher w/c ratio grout was used (Fig 7).

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**Fig. 6. Results of inclinometer test indicate worst deviation of 1 in 56, and majority of bores closing back to the intended line towards the base.**

**Fig. 7. Results of test anchorages installed with 0.45 and 0.6 water/cement ratio grouts.**

Grouts in the majority of anchorages have contained either rapid hardening or sulphate resisting Portland cement in neat form. In a number of referenced cases polyester resins have been employed, generally to allow 24 hour testing. More
recently, angular sands have been added to the cement grouts to limit and control grout flow into fissured or broken rocks, either for pregrouting or as an integral part of the anchorage installation. This has allowed anchorage construction in rock conditions which might previously have been considered unsuitable.

**Anchor tendons**

Over 90% of tendons incorporated in the anchorages have consisted of a multiple of pre stressing strands installed in varying configurations. In early days, the seven wire strands (0.6 inch diameter normal, super stabilised or Dyform) were unravelled and rewound to form a number of long bushes in each strand. The strands were banded together such that the deformation and bond length in both cement and polyester grouts were extensively investigated under laboratory and full scale site conditions. Results of this research work influenced the tendon configurations, spacing and density recommended in DD81 and also led to the adoption of a controlled strand deformation system in all the referenced strand anchorages installed after 1978.

In temporary anchorages strand bushes were reduced to lengths of approximately 500mm and located at staggered positions on adjacent strands. Strands were separated by plastic spacer/centraliser units secured at 2m to 3m centres to provide 5mm strand spacing at the unit and banded back in contact between the units to form tendon nodes. The controlled bushes were nodes and limited in overall diameter in order to prevent contact with the borehole wall. The plastic centraliser ensured a minimum of 8mm cover throughout the bond length of the tendon.

Over the anchor free length the temporary anchor tendons were debonded from the grout by either individual strand or group strand plastic sheathing. Generally this sheathing was extended for a metre or more into the upper part of the fixed anchor length to prevent the concentration of loading near the bedrock surface. This strand system for temporary anchorage has, in principle, been continued with minor refinements.

**Pic. 5. Fabricated anchor tendon showing bushes and cage effect used without spacers or centralisers in 1974.**

bushes interwound to form a continuous cage within the fixed anchor length (pic 5). When installed, the outer wires of the cage were in contact with the borehole wall so that the bulk of the tendon was isolated from the wall and from possible contamination by smearing in chalks, marls and weak mudstones. The caging also allowed the full penetration of the grout into the tendon configuration and mechanical interlocking ensured mobilisation of the full capacity of the tendon.

During the mid 1970's the effect on bond capacity of varying the strand configuration, spacing, density,
Permanent anchorage tendons have, in contrast, changed enormously over the time period considered. In 1974, to allow safe use of permanent corrosion proof anchorage tendons for support of the raised bank walls downstream of the Thames barrier, Greater London Council investigated worldwide systems of tendon corrosion protection. The resulting contract anchorage specification initially favoured protected bar systems developed by continental anchorage specialists, the Bauer/Stompf compression tube. But after a short period of bar coupling and coupling protection problems, protected multistrand systems were also approved. Arrival of this specification to control the installation of several thousand anchorages between the barrier and the Thames estuary within a three year period, caused a flurry of activity, research and development.

Some UK specialist companies had basic tendon protection systems available but up to that date the majority of permanent anchorages installed had consisted of bare strand or bar cast directly into the fixed anchor grout. The alkaline environment of the cement grout alone had been considered adequate protection against corrosion over the tendon bond length. Within the tendon free length, the strand system incorporated an individual strand sheath surrounding each greased strand. The grease allowed free movement of the strand within the sheath and provided a supplementary corrosion protective layer. A similar free length sheathing system was available on bar tendons for permanent anchorages.

GLC's specification, eventually followed by DD81, demanded double protection against corrosion throughout the entire tendon length (pic 6). The term double protection was translated and introduced from continental practice. It called for a continuous physical barrier around the steel tendon, in addition to the cement grout cover, in order to prevent penetration of corrosive fluids. Initial development and marketing of tendon bond encapsulation systems manufactured from galvanised corrugated steel duct was soon to be followed by the almost universal use of plastic corrugated ducts as the physical barrier over the tendon bond length. Prefabrication of such encapsulated tendon bonds lengths, cast into polyester resins or high quality cement grouts, has become the norm for both bar and multistrand tendons. Permanent tendon prefabrication, once carried out in open site locations, is now implemented under controlled factory conditions in the UK.

More stringent requirements of the imminent BS 8081 will require considerable improvements in the material properties and component configurations. Tendon free length protection of grease and single strand sheathing is to be supplemented by a group plastic sheath. Bar systems follow suit with double plastic sheathing.

The tendon/grout load transfer system for multistrand tendons is similar to temporary anchorages. The upper end of the tendon bond length is generally located some distance into the designed fixed anchor length. Thus, encapsulation lengths are generally some metres shorter than the fixed anchor length to ensure that load is not concentrated near the bedrock surface.

The majority of encapsulation systems, whether bar or multistrand, tend to use a

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**Fig. 9. Encapsulation systems and stress distribution.**
The eductor dewatering system

By Eric Miller, MSc, FICE

An eductor, or ejector, is a jet pump configured for raising groundwater from a borehole. It is particularly well adapted to being operated as a group forming a dewatering system and used for this purpose has certain advantages over other methods:

- The eductor system is not limited in its lift, to something less than an atmospheric head, as are vacuum suction self-priming centrifugal pumps used in well point dewatering.
- The pump has no mechanical moving parts and is therefore free of mechanical breakdown problems. It is simple and reliable.
- An individual eductor remains operational in a borehole whether or not it is intaking groundwater, i.e., the system continues to work when individual boreholes become dry.

Site personnel

The necessity to provide a reliable, experienced, and coordinated anchor team cannot be overemphasized. Attention to engineering control is vital throughout all anchorage operations. Such control and organization may supplement but cannot replace the art and expertise of a capable anchor driller and anchor grouter. Working conditions are often cold and wet, exposed and severe (pic 7) making personnel reliability an essential characteristic of any team.

Recent anchor contracts have generally employed foremen, drillers and grouters with a minimum of five years anchoring experience and have frequently involved crew members who have worked together for 10 years or more.

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REFERENCES


Pic. 7. Working conditions can be cold, wet, exposed and severe on the Thames in winter (1977).

Each individual double protected strand of the multiple anchor transfers load to the grout and rock from its own encapsulation over its own fixed anchor length. Individual strand encapsulations are located at staggered depths in the borehole and are decoupled from other encapsulations above or below (Fig 9). The system has already proved valuable in mixed ground conditions of cohesive and granular soils, and is at present being fully investigated in weak highly weathered mudstones.
Eductor form
The simplest form of eductor is shown diagramatically in Fig 1. The downpipe carries a pumped water supply to the vertically upward facing nozzle, the jet of which enters the venturi section of the eductor. The jet induces the otherwise static groundwater present at the entry to the venturi section to flow into the venturi. This induced flow combines with the pumped flow in its movement to the surface up the return pipe. At the surface the pumped and induced flows are received into a tank. Water to sustain the operation is recirculated from this tank while the excess is disposed of (Fig 4).

The fact that the eductor can operate in so small a well pipe diameter can be a useful economic advantage in favour of the system. It is necessary to incorporate cup seals below the single pipe eductor to provide an end stop to the annular down pipe and prevent pumped flow discharging into the aquifer. A non return valve is also needed to prevent draining of the down and riser pipes on cessation of pumping. At the ground surface a well head fitting has standard pumped inflow and return outflow connections, normally at about 25mm diameter (Fig 3).

The process by which flow is induced has traditionally been attributed to the low pressure created at the venturi throat by normal venturi action. However the high velocity of the jet, 20m/s to 35m/s for nozzle inlet heads of 30mwg to 100m wg, almost certainly precludes the venturi acting as such in the section upstream of the throat. A more probable explanation of the induced flow is entrainment of the surrounding groundwater with momentum transfer in the highly turbulent perimeter of the jet. The process is undoubtedly complex however the flow is induced. Relationships between induced flow and the other parameters must be obtained by physical testing at prototype scale. It appears obvious that if the groundwater level is at or below the intake level of the eductor, then air will be induced to flow into the eductor throat. When air is not available then subatmospheric pressure conditions will develop.

Fig. 2. ‘Single pipe’ eductor form
This two pipe form is commercially available and most commonly used for rural domestic groundwater supply purposes. A useful modification of the two pipe form is shown in Fig 2. The return flow pipe is contained concentrically within the pumped supply downpipe, which becomes an annulus. This single pipe eductor system is the common form for dewatering and has an eductor/down pipe diameter of approximately 60mm operating in a 80mm diameter well screen pipe.

Eductor within a borehole
The eductor will be set in a borehole at or below the depth to which the groundwater level is to be drawn down. In considering the work done to operate this eductor unit it is useful to treat the pumped flow and induced flow at if they were independent. The pumped flow fills both the down and riser pipes as in an inverted siphon and the work done in moving it is the product of the flow and the friction head. The

Fig. 3. PVC recoverable wellhead unit
Fig. 4. Eductor field installation
induced flow also incurs friction headloss in the riser pipe but in addition requires work input to lift it from the groundwater level up to the receiver tank water level. Finally there is an element of work done within the eductor itself. With a knowledge of the eductor performance derived from laboratory testing and the flow/headloss characteristics of the down and riser pipes used, the hydraulic characteristics of the eductor within a borehole may be determined.

**Eductor field group**

When dewatering it will be usual to set up a group of down the hole eductors, most often in the form of a line of equally spaced wells. This group with the associated ground surface pipework forms the complete hydraulic system as indicated in Fig 4. The pumped flow inlets to the eductor units are connected to a common pumped flow main. Likewise the individual return flow outlets join a common return flow main which will normally terminate in a water storage tank. Junctions between the mains and the individual eductor well heads will normally take the form of flexible 25mm diameter hoses, each of which will have a shut off valve and a union connector so that any eductor may be easily isolated from the rest of the system. Detail of the appropriate layout can vary considerably.

Surplus water drawn up by the system will be discharged from the tank from a constant level overflow, generally a calibrated notch.

The distal ends of the pumped and return mains will normally be connected with a smaller valved pipe to form a return loop. While not needed in theory, this valved connection gives additional flexibility of operation.

**Eductor performance test**

In view of what appeared to be the less than comprehensive performance data available for proprietary eductors, a laboratory performance test was carried out by Wimpey Laboratories on a single pipe eductor of the type indicated in Fig 2. Test results are presented as being generally indicative of the performance of this eductor type.

The test rig is shown in Fig 5, in which the main tank was approximately 1000 litres capacity. A submersible pump discharged through a horizontal 50mm diameter pipe, with a flow measuring orifice plate in line, to a short vertical length of eductor downpipe/riser terminating with the eductor under test. The riser pipe fed into a small receiver tank with a Vee notch measuring overflow back into the main tank. Although the induced flow could be calculated as the difference between the return flow and the pumped flow, both of which were measured, the induced flow was also measured directly by means of a Vee notch in a secondary tank surrounding the eductor assembly within the main tank. These level controls made the test rig very sensitive to non equilibrium conditions but once an operating method was developed it was not difficult to set an equilibrium condition. This required the volume of water in the system to be carefully adjusted for each equilibrium test pressure in order that the depth of water at the eductor intake could be kept constant, at the level of the internal nozzle. Valves in the pump discharge line and return flow line allowed the pressure head into and out of the eductor to be controlled. Thus, within the pressure head limits of the pump, the eductor could be subjected to site conditions as if in a borehole, saving only that the groundwater level was fixed relative to the eductor.

**Performance test results**

Fig 6 shows the test results for the eductor with best fit curves. The vertical cut off shown limiting the measured value of

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**Fig. 5. Eductor test rig**

**Fig. 6. Eductor test data I**
observed, being internal to the eductor, nor measured but was very distinctly audible with a sharply defined inception.

A primary relationship not shown in Fig 6 is that of the pumped flow \( Q_p \) to one or more of the pressure parameters. Fig 7 shows the pumped flow relationship to the pressure difference over the eductor, ie nozzle pressure \( P_n \) minus back pressure on the eductor \( P_v \). If the eductor were a simple, in line constrictor, a near square law relationship would be expected. It is apparent that only when the induced flow is low, is such a simple relationship established. In the cavitation zone the pumped flow is related to nozzle pressure alone and over the intermediate (normal operating) range of the eductor the relationship is transitional. A near square law relationship exists between pumped flow \( Q_p \) and nozzle pressure \( P_n \), Fig 8.

Analysis of results

Test result curves are idealised and extrapolated as dashed lines in Fig 6 and, for pressures in metres water gauge (mwg) and flow in litres per second (litres/s), the following empirical equation is derived:

\[
P_v = 4.95 + 0.385 \times P_n - 42.52 \times Q_i \text{ where the induced flow } Q_i \text{ is less than or equal to } 0.294 \text{ litres/s.}
\]

It is necessary for practical design appraisal to express the eductor performance in terms of pump pressure at ground surface \( P_p \) instead of nozzle pressure \( P_n \), and the depth of the
eductor (D) instead of the back pressure on the eductor \( P_v \).

In the test rig \( P_p = P_n \) and \( D = P_v \), since for practical purposes the eductor under test has a negligible length of downpipe/return pipe.

On site with the eductor at depth D the downpipe and return pipe give rise to friction which needs to be accounted for in calculating the flow and pressure parameters of the unit. These are simple calculations except for the difficulty of calculating the downpipe friction where the downflow is in the annulus between the downpipe and the internal riser pipe. An approximation has been made by calculating the loss in a full bore pipe

![Fig. 8. Eductor test data III](image)

**Fig. 8. Eductor test data III**

Flowing at a mean velocity equal to that in the annulus, and doubling it to represent the friction on both surfaces of the annulus. Although this is possibly not an accurate method, the friction will normally be small in relation to other losses.

Design performance curves have been calculated using the empirical performance equations of the eductor and the calculated friction in the down and return pipes (Fig 9). It is worthy of note that these design curves imply specific downpipe and return pipe dimensions, which need to be stated to complete the design information, and that the induced flow indicated was obtained for the case where the groundwater level was at the level of the nozzle of the eductor, ie the groundwater lift was maximised for the case of a submerged eductor intake.

![Fig. 9. Eductor design curves](image)

**Fig. 9. Eductor design curves**

Unfortunately the test results did not extend to relating the induced flow to other levels of groundwater. When the groundwater level falls to the level of the eductor intake, the induced flow rapidly falls towards zero as air is entrained.

**Eductor system design**

First design phase for a dewatering project is estimation of the drawdown achievable with an eductor group. This requires a knowledge of the subsoil lithology and soil properties including an assessment of the mass permeability of the aquifers, as well as the groundwater conditions. In practice eductors are generally spaced at between 3m and 15m depending on the soil properties and the drawdown requirement. The closer the eductors, the greater the overlap between the cones of drawdown and therefore the more uniform the drawdown surface. It would not normally be considered economical to space a line of eductors much closer than 3m.

Second design phase is the application of appropriate performance curves as typified by Fig 9 to the group of eductors forming the dewatering system. This requires the ground level pump pressure

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and the ground level back pressure at each eductor to be assigned. It will be noted that the data of Fig 9 is not only for particular down and return pipe sizes but also for a particular model of eductor. As a starting point it would be reasonable to assume that both the pump main and the return flow main are of sufficiently large diameter to make the flow friction loss negligible and therefore the pump pressure at each eductor downpipe inlet will be the common pump discharge pressure plus any elevation change between pump and eductor ground surface connector. To allow for the lift of the return flow pipeline from ground level to its outlet level, normally 1.5m to 2m, this head must be subtracted from the available pump pressure head. For each eductor the effective pump pressure at ground surface may be read into Fig 9 with the depth of the eductor to give the induced flow. The sum of the induced flows for the eductor group is, to a first estimate, the total pumped flow required for the system.

At this stage it is necessary to consider:

- whether the total induced flow associated with the eductor group is likely to meet the dewatering requirement.
- whether the total pumped flow calculated with its associated discharge pressure is a practical specification for a single pump and if so, the size and type; if not, how best to split the group.

When these criteria are broadly satisfied, the final design phase is the reiteration of the hydraulic calculations to match the pump flow requirement to the characteristic curve of the pump, allowing for main pipe losses and secondary losses in system fittings. All of the calculations are readily computed.

Some practical considerations

The modest flow induced in a single eductor of the size and type discussed tends to limit the practical application of the system to the less permeable soils - fine sands and silts. But since this is a difficult group of soils to dewater by other means when deep, it is a very useful field of application.

As has been stated, individual eductors in a hydraulic group continue to pass the pumped downflow when the groundwater level is lowered to the eductor intake. In this operating state there ceases to be an induced flow of groundwater but air is entrained if it is available. This air rises with the return flow and enters the common return main where it must be disposed of. As a very low pressure main, it would be difficult to operate conventional air release valves. An alternative would be open ended stand pipes. But on reasonably level ground the situation is undoubtedly best dealt with by encouraging the air to be swept along this main unimpeded, to escape by gravity at the free discharge outlet of the main. The profile of the return main should be designed with this in mind. Adequate venting direct to the atmosphere should be arranged at the discharge outlet.

It is undesirable that air or suspended matter in the return flow to the storage tank be allowed to be recirculated by the system pump. This requires that the tank should be of such dimensions as to prevent short circuiting flows between the tank supply point and the pump intake. Baffles may be required to assist the settling process. Heavy loads of suspended matter in the return flow may require separate receiving and pump tanks, but the nature and cause of a heavy suspension, other than an initial flush, would indicate a nonstable situation which would need to be investigated.

An inherent characteristic of an eductor is the subatmospheric pressure created somewhere between the nozzle outlet and the venturi outlet. One effect of this is cavitation which, as has been described, appears to limit the maximum flow rate.

Another possible effect of this sudden decrease of pressure is the release of dissolved gases and/or the triggering of chemical precipitation which can cause problems of scaling. These effects have not been studied but severe precipitation of an iron compound has been experienced. Carbonate precipitation is also a possibility depending on the quality of the groundwater.

Preventive maintenance will require that an eductor be taken out of service occasionally for inspection and possible cleaning. This is a simple operation which begins with the isolation of the eductor by closing the valves in the flexible pipes and uncoupling the well head. The shutting down of one eductor in a system will normally have an insignificant effect on the main pump performance so that the rest of the system will continue to operate normally. The well head fitting is removed, exposing the open ends of the down and return pipes and allowing the return pipe, with the eductor attached to its lower end, to be withdrawn. As the PVC pipe is lifted it begins to deflect from the vertical until the free end is within reach of an assistant who restrains it and ensures the curvature is retained until the eductor end is exposed. The pipe is laid on the ground and the eductor dismantled.

Replacement is the reverse procedure. The downpipe can also be removed in the same way but there will seldom be a need. Two men can remove and replace an eductor from depths in excess of 40m with ease. The process of withdrawing an eductor as described requires that the lower cup seals are able to slide up through the downpipe, ie against the lip of the cup seal. The downpipe must be jointed so as to allow this – sockets must face down and couplings must have no seal lip traps. If an eductor does become stuck during removal, it can be returned, the downpipe with riser pipe withdrawn together, laid on the ground and disassembled.

Another possible use for the eductor system, other than locally lowering a groundwater table, is that of creating a subatmospheric pressure within a subsoil zone resulting in an acceleration of the consolidation process. It seems unlikely that the eductors would be economic for this purpose alone, but it could be a useful additional advantage to a dewatering system.

In summary, the eductor system is seen to be a very practical method of dewatering that has much to recommend it. It could undoubtedly be used more extensively in civil engineering practice if manufacturers obtained reliable hydraulic performance data and made this available to the industry.