Clunie Dam Anchoring Works

A W GREGORY, Scottish and Southern Energy plc, UK
A E HENDERSON, Ritchies, UK
K C HENDERSON, Ritchies, UK
P NOWAK, Atkins, UK

SYNOPSIS. Following the statutory inspection of the Loch Tummel reservoir *"matters in the interest of safety"* were raised requiring a stability analysis be carried out to check the behaviour of the dam under load from a Probable Maximum Flood (PMF) event and also from a seismic event.

The paper describes the analysis; site investigation; subsequent design; and on-site construction works to implement a dam anchoring scheme which meets the requirements of the Qualified Civil Engineer.

INTRODUCTION

The analysis and subsequent works carried out to Clunie Dam were required following the 10-yearly statutory inspection under the Reservoirs Act 1975 which was carried out in June 1999 by Dr A K Hughes. Subsequent structural assessments demonstrated that the seismic stability of the dam was adequate, but that under worst case PMF conditions, the abutment sections of the dam were likely to crack becoming grossly unstable and were expected to fail. Further consequence studies and ALARP studies demonstrated justification for implementing strengthening works.

GENERAL DESCRIPTION OF DAM

Clunie Dam is located in a narrow valley at the eastern end of Loch Tummel and was constructed in the period 1949 – 1951 as one of a series of dams, of similar construction, to provide hydroelectric power. The dam is a concrete gravity structure some 21 metres high and 116m long retaining a reservoir with a capacity of 36,400,000 cubic metres.

The longitudinal axis of the dam runs in a generally north east/south west direction. The central portion contains the floating drum gates, each of which is 18m long. The left and right side gravity dams are 40.9m and 40.8m long respectively, with a maximum height of 23.8m. The gravity

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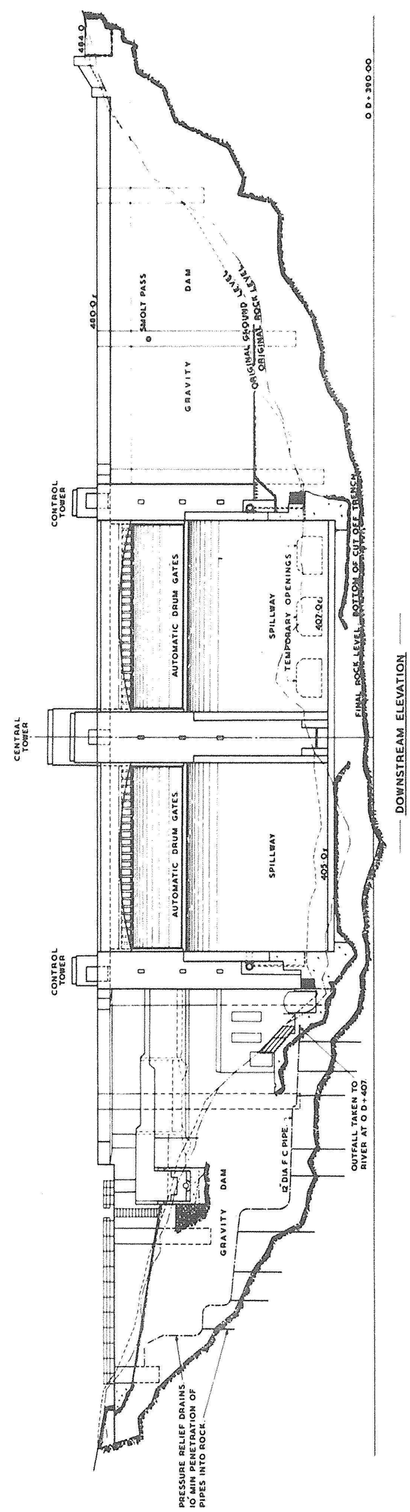

Figure 1 – Downstream Elevation on Dam

sections were both cast with a vertical upstream face and a downstream face at an inclination of 4V:3H.

The dam is founded on solid rock, with the gravity sections incorporating a cut off trench and a grout curtain drilled up to 24m into the rock.

The dam is categorised as falling within 'Category A' as defined in the publication *'Floods and Reservoir Safety'* (1996), and is placed in Category III as defined in the publication, *'An Engineering Guide to Seismic Risk to Dams in the United Kingdom'* (1991).

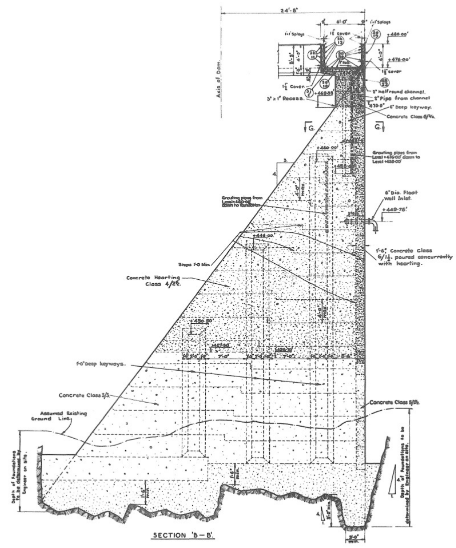

Figure 2 – Typical section through Left Bank gravity dam

SECTION 10 INSPECTION REQUIREMENTS

The Section 10 inspection report recommended that a stability analysis be carried out to check the behaviour of the dam under the loading from a PMF event and also from a seismic event.

A number of studies were carried out and concluded that

- At PMF the reservoir would rise to a level such that the parapet wall would be overtopped by 2.9m for a duration of 6 days with unit discharge over the non-spillway sections of up to 16m^3/s/m.
- The non-spillway sections of the dam have an inadequate safety factor against overturning under PMF loading

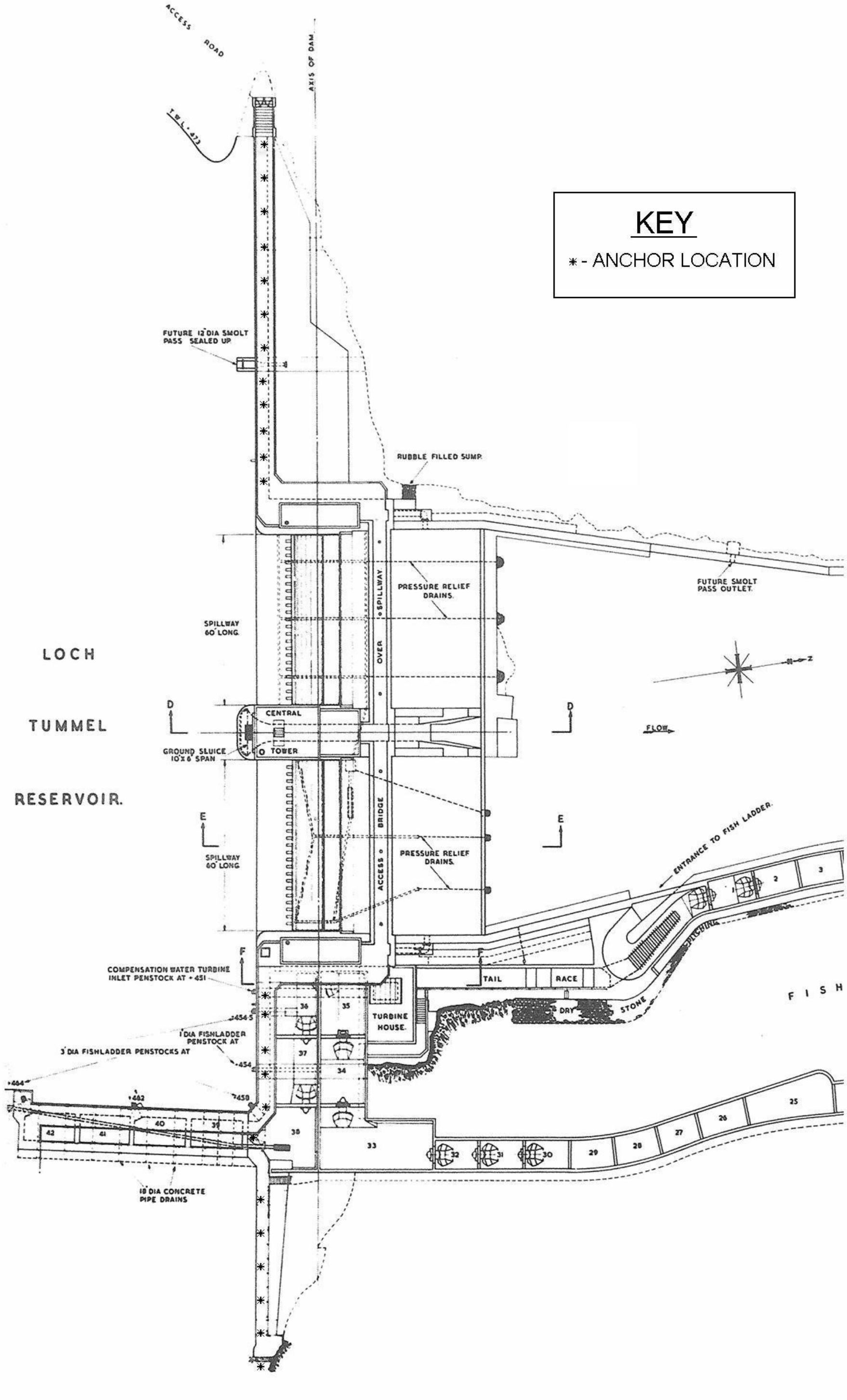

Figure 3 – Plan on Dam

- An additional failure mode, of sliding in the upper part of the rock foundation, was identified as being more critical than overturning. In this situation, the failure mode is brittle with a relatively rapid reduction in safety factor with an increase in water level, as the mode is determined by cracking at the heel, leading to both an increase in uplift pressures along the crack and reduction in shear strength along the cracked length as c' reduces to zero.
- The annual probability of failure was estimated as about 2×10^{-4}(1 in 5000 years), this being reduced to about 2×10^{-5} following anchor installation.
- The reports recommended the installation of post-tensioned anchors to prevent failure by either overturning or sliding, with an anchor load of 870kN/m required to give a factor of safety of 1.05 for the worst case PMF with a low tail water level.

Quantitative risk assessment (QRA) was used to assess whether the risk of dam failure would be reduced to "as low as reasonably practical" (ALARP) i.e. whether the cost of strengthening works would be grossly disproportionate to the reduction in risk achieved, and it was concluded that the strengthening works could be justified.

INVESTIGATION PHASE

In order to determine the nature of the ground acting as the dam foundation and confirm the nature of the dam construction a ground investigation and desk study were carried out.

Ground Investigation

Implementation

The investigation was carried out in June and July 2006, and comprised four rotary cored boreholes to a depth of between 35.24 metres and 45.0 metres below dam crest level. Subsequent to the completion of rotary coring an Optical Televiewer survey was carried out in all boreholes. On completion, all boreholes were filled with bentonite/cement grout. All rock/dam construction core was retained in core boxes of which photographs were taken and from which samples for laboratory testing were taken.

Results

The boreholes confirmed that the dam was founded on strong inter bedded schist and quartzite bedrock with little or no weathering below the concrete/bedrock interface..

The optical televiewer survey generally identified near vertical fracture orientation in the foundation bedrock. The televiewer survey showed the concrete within the dam construction to be essentially monolithic although one or two joints could be identified.

Laboratory Testing
Laboratory testing comprised the determination of unconfined compressive strength (UCS) and point load strength on cores of bedrock and dam concrete. A summary of strength test results is presented below.

Table 1. Rock Strength Summary

	UCS (MN/m2)			Point Load I_s50 (MN/m2)		
	Concrete	Schist	Quartzite	Concrete	Schist	Quartzite
No of Tests	13	8	4	17	13	3
Average	43.01	58.24	57.35	2.87	4.87	3.50
Maximum	59.99	85.69	91.51	4.96	9.69	4.39
Minimum	29.81	22.49	34.47	1.62	1.13	2.84

Desk Study
A comprehensive desk study of the construction drawings for the dam was carried out to allow the proposed anchors to be spaced such that they did not interfere with existing elements of the dam. Of particular interest was the location of penetrations through the dam for fish pass, compensation generation and control pipes, which would dictate the layout of the proposed anchors. Subsequently a full topographic survey of the gravity sections of the dam was carried out to confirm the key dimensions and physical layout.

ANCHOR DESIGN
From the inception of the project, it was intended that the successful anchoring contractor would undertake the works on a Design and Build basis. However, to allow a meaningful comparison of tenders, the number, spacing and load capacity of anchors was fixed by the Designer, and shown on the Contract Drawings. Anchors were positioned so as to avoid services and openings in the dam structure.

Design Philosophy
The philosophy of the design of rock anchors at Clunie Dam has followed the same principles as those adopted at Pitlochry dam in Sandilands *et al* (1994) with vertical anchors through the concrete gravity dams proposed to prevent failure of the abutments by sliding or overturning.

Forces for the design of anchor loads were derived from preliminary analyses taking into account changes in cross section variations across the dam. Six cross sections were analysed based on geometries provided from the As Constructed drawings. The analyses considered the beneficial effect required by anchors installed at the crest of the dam with regard, predominantly, to overturning of the dam about its downstream toe.

The width of the walkway on both the left and right abutments severely restricted the positioning of the anchor holes with respect to the geometry of the abutments. All anchors could only be positioned close to the upstream face with a clearance of some 750mm if they were to be installed in the existing walkway. This was the only practical location for installation of the anchors as alternative locations on the downstream face of the dam would involve significant temporary works and health and safety issues. The location chosen allows for verticality tolerances when drilling the anchor hole to prevent ‘daylighting’ of the anchor hole through the upstream face. Additionally, the downstream side of the walkway generally overhangs the downstream slope of the dam so the whole walkway width was not available for anchor installation.

The proposed location is advantageous when considering the contribution of the anchors with regard to overturning about the toe of the dam as it utilises the longest lever arm available to maximum the restoring action of the anchors.

Anchors were spaced at nominal 3.0 – 3.5 metre centres along the abutment crest. Anchor positions were constrained by the presence of pipework in the abutments and construction gaps from the original construction, and were been positioned to avoid these features assuming that the main concrete construction panels would act monolithically.

Proposed anchor locations are indicated in Figure 3.

Pre-Tender Anchor Design

The pre-tender design of rock anchors was carried out to BS 8081 (1989). Type A anchors were designed as advised by the Code. The anchor design requires a unit side friction which can be derived from the unconfined compressive strength (UCS) of the rock. Values of UCS were taken from laboratory testing of rock samples recovered from the ground investigation and a working friction of 1.2 N/mm2 was adopted for the pre-tender design.

All anchors were designed with a factor of safety of 3.0 as advised by BS8081. Design anchor loads have taken into account the constructed height of the abutment gravity walls and indicative design loads reduce from 2500kN per anchor close to the centre spillway to 1400kN per anchor close to the valley sides.

Additional analysis was carried out considering failure of a cone of rock around the anchor fixed length as the dam rotates in overturning after the approach after Tomlinson in BS8081, considering overlapping cones of failure for individual anchors.

This failure mechanism controls, and thus a minimum fixed anchor length of 5.0 metres into bedrock was stipulated. Individual anchors were staggered by 5.0 metre depth in order that failure cones did not significantly overlap i.e. 5 metres and 10 metres below the concrete/bedrock interface.

Load testing of anchors was specified after 9 months for three anchors to confirm that the anchors are fully stressed. A full round of load testing for anchors is scheduled for 2009 to align with other SSE dam anchor testing frequencies.

<u>Contractor's Anchor Design</u>
The Contractor-designed components comprised the tendon, the grout, the tendon/grout/ground bond interfaces, the bearing plate and the head assembly for each anchor. Design of these elements followed the requirements of BS8081 and was thus undertaken using a global factor of safety approach with a factor of 2 on the tendon ultimate capacity and a factor of 3 on all bond interfaces.

Tendon design was based on using multiples of 15.2mm dyform strand with an individual ultimate strand capacity of 300kN, supplied by Dywidag Systems International (DSI). The strands forming the tendon were noded and splayed within the fixed length to enhance the bond capacity by mechanical interlock. Double corrosion-protection (DCP) was specified and this was achieved by using DSI Type 7B anchors, comprising twin concentric corrugated plastic encapsulations over the bond length and a single corrugated plastic encapsulation enclosing individually greased and sheathed strands over the free length.

The grout design followed conventional practice for anchor grouts and used a neat CEM1 42.5N cement colloidally mixed with a water:cement ratio of 0.4:1 to give an homogenous liquid grout with a fluid density of approximately 19kN/m^3. The liquid grout displaces bore water without dilution when injected through a base-mounted tremie line and cures to give cube crushing strengths well in excess of 40N/mm^2 at 28 days. When properly mixed, the grout demonstrates 1hour bleed of between 3 and 4% and reaches initial set in approximately 45 minutes.

The bond interfaces were designed using the equivalent rectangular stress-distribution method described in the BS8081. Ultimate bond values for the grout/tendon and grout/encapsulation interfaces were drawn from the limiting value of 3N/mm^2 recommended in the code.

In common with much of Perthshire, the rock underlying the dam is high grade metamorphic, typically strong schists and quartzite. The schists demonstrate little fracturing and good mineralogical infill of the expressed schistosity. A grout/ground bond value was calculated using methods described by Rosenberg & Journeaux (1976) and Williams & Pells (1981), with a conservative unconfined compressive strength (UCS) of 35MPa (test values ranged from 29 to 87MPa) and a Rock Quality Designation (RQD) of 50. This yielded ultimate bond stresses of 1.75 and 2.275N/mm^2 respectively. A value of 1.5N/mm^2 was selected for design purposes; however, it is acknowledged that given the nature of the rock, substantially higher bond values could have been justified by intensive testing of a trial anchor.

In practice, the limiting bond interface was found to be the grout to inner encapsulation in all cases, leading to fixed lengths ranging from 5.0m to 7.3m.

Design of the head assembly bearing was based on a dam concrete strength of 35N/mm^2 (derived from UCS testing on samples recovered during the ground investigation) and an allowable bearing stress of 0.3f_{cu}. Using circular bearing plates, this gave diameters of variously 500, 550 and 600mm. The bearing plate thicknesses were calculated using published formulae for simply-supported annular plates in biaxial bending and these gave thickness of between 70 and 85mm.

The head assemblies were standard DSI components, designed in accordance with the requirements of BS8081 and BS4447 (1973). The tendon strands are held using locking serrated wedges fitted into conical sockets machined into the solid headblock. The assemblies include a short transition sheath between the tendon outer encapsulation and the bearing plate to allow the strand bundle to splay slightly before entering the headblock. Each assembly was completed with a sealed steel cap filled with corrosion-inhibiting compound.

The anchor components (tendons, encapsulations, bearing plates and head assemblies) were manufactured in the DSI anchor manufacturing facility in Southam, UK and were brought to site as complete, quality-assured anchor units.

IMPLEMENTATION

Contract Strategy

The anchor installation contract was put out to tender to four contractors, with the Ritchies Division of Edmund Nuttall Ltd. being the successful tenderer. The contract was let under the NEC3 Engineering and Construction Contract under Option A, (Priced Contract with Activity Schedule). All works were successfully completed over a 13 week period in summer 2007 at a final cost of £498,000 (including all design and SI work).

Execution

The sequence of operations required to install the anchors can be broken down as follows:

1. Coring the anchor head recesses.
2. Drilling of the anchor boreholes.
3. Installation and grouting of the anchors.
4. Preparation of the head blocks and stressing of the anchors.
5. Capping of the anchors and installation of access covers.

Coring Works

To maintain access on the dam following completion of the project, each of the installed anchors was recessed into the crest walkway. The recess comprised a 700mm diameter x 500mm deep hole formed by coring through the concrete. Circular load bearing plates were used to optimise the available bearing contact area and 50mm diameter drainage holes were cored from the downstream face of the dam using roped access techniques.

Drilling Operations

Drilling operations were carried out using a Klemm KR701-01 restricted access drilling rig using a remote hydraulic power pack and 50m hydraulic hoses. This arrangement allowed the drilling rig to operate on the narrow dam walkway while the power pack remained on land. Drilling was undertaken using a 254mm diameter drill bit on an 8” down the hole hammer using air flush and water injection for dust suppression. A centraliser was used within the starter core hole together with stabilised drill rods to ensure verticality of the borehole during drilling. Spoil from the drilling process was contained using a well head and diverted into skips placed on the dam walkway which were replaced as required. Small diameter reinforcement bars were locally encountered within the dam during the drilling works. These obstructions were overcome using a tungsten-tipped core barrel in combination with the down the hole hammer.

Anchor Installation

Strand anchors supplied by Dywidag Systems International were used during the project with installed capacities of between 1400kN and 2500kN. Anchor lengths ranged from 11.5m to 39.8m and used between 12 and 17 strands depending on the required working load.

The dam is accessed by a narrow single track road which precluded the use of traditional craneage for installation of the anchors. During the tender period a number of options were considered, including the use of helicopters. To overcome the access restrictions an alternative anchor installation method was developed which minimised the environmental impact of the project and reduced the projected project cost.

The installation method employed used a de-rated 360-degree telehandler mounted on a work barge operating on the impounded loch. Anchors were delivered to site pre-coiled and with a 1m section of the bond length grouted. Each anchor was loaded onto a bespoke anchor wheel which could be attached to the telehandler. The work barge then transported the anchor to the borehole location, where it was uncoiled down the borehole using a hydraulic winch on the telehandler mounted anchor wheel. This method was successfully and safely utilised for all of the anchors installed over water.

Figure 4 – Anchor installation using barge mounted telehandler

Anchor Stressing and Capping

Grouting of the anchors was undertaken using a neat Portland cement grout mixed in a high shear colloidal grout mixer. Grout was injected into the borehole through three internal grout lines in the anchor. Once the grout had achieved sufficient compressive strength, cyclic load testing of the installed anchors was undertaken in accordance with BS EN 1537 (1999). All anchors performed within the expected limits both during the suitability and acceptance tests that were undertaken. On completion of the stressing operations removable access covers were installed over the stressing recess to allow the walkway to be re-opened for normal use.

CONCLUSION

The anchoring works have been successfully carried out, using an innovative method of installation giving environmental, cost and programme benefits.

The dam structure now complies with current UK safety standards, and has been issued with a 10(6) certificate.

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