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Stability of Shannon Embankments

K. R. Massarsch
Royal Institute of Technology (KTH), and Consultant to VBB/SWECO, Stockholm, Sweden

M. O. Fives
Electricity Supply Board (E.S.B.), Dublin, Ireland

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K. R. Massarsch

Adjunct Professor, Royal Institute of Technology (KTH), and Consultant to VBB/SWECO, Stockholm, Sweden

M. O. Fives

Assistant Chief Civil Engineer, Electricity Supply Board (E.S.B.), Dublin, Ireland

SYNOPSIS During the operating life of the Shannon Embankments, stability and seepage problems have occurred which caused concern regarding long term safety. The geology was studied, a comprehensive scheme of field and laboratory investigation was undertaken and the stability was analyzed and back calculated under failure conditions. The relative significance of various parameters was studied and provided the basis for the design of remedial measures. This paper deals in detail with the study of the stability of one embankment section (Fort Henry) and the recommendations on remedial measures.

INTRODUCTION

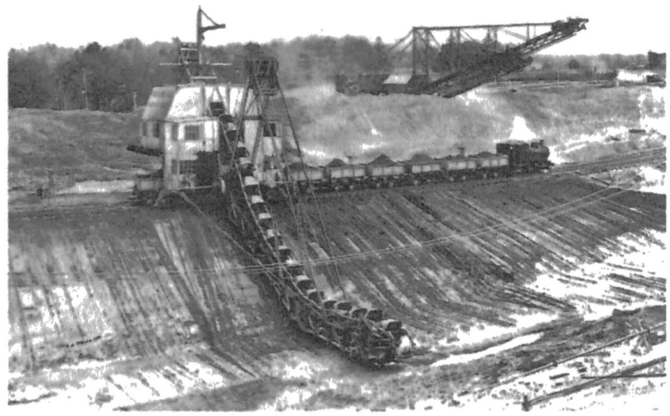
The Ardnacrusha hydroelectric project on the river Shannon near the western coast of Ireland was constructed by Siemens in the period 1926 to 1929. This project included a 12 km long headrace canal and the reservoir embankments called respectively the Fort Henry and the Ardclooney embankments. During the operating life of the project stability and seepage problems have occurred and gave rise to concern regarding the long-term stability and safety of the installation. In 1980-81 a major engineering study was undertaken to determine the remedial measures necessary to assure the owners of the safety of the embankments. This paper deals specifically with the study of the 3.5 km long Fort Henry embankment.

CONSTRUCTION

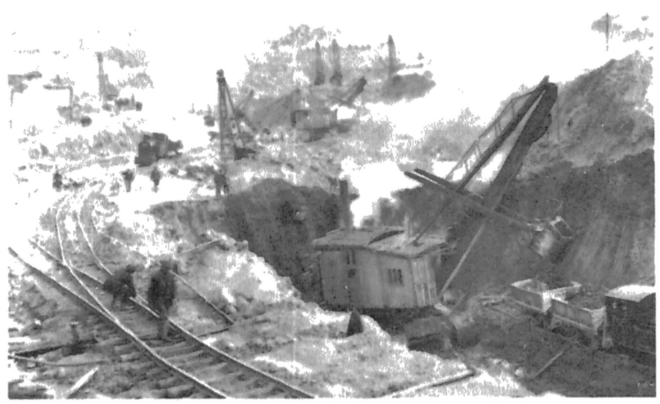
Construction was started in 1926 and took approximately four years. Construction involved the removal of 5 million cubic metres of earth and 0.3 million cubic metres of rock. The headrace canal was constructed in cut or embankment for 11.5 km, 2 km of which in rock, Fig. 1 a. Boulder clay for construction was drawn from several borrow pits. On porous subsoils, the canal bed and sides up to original ground level were given a 60 cm puddle lining (clay) Fig. 1b. The embankment was tipped directly on original ground after stripping of humus, roots and organic matter. The maximum height of the embankment is 18 metres with an average height of 8 metres.

Construction of the about 7 m high reservoir embankments began early in 1927. The design evolved as construction work proceeded. At the northern 1.4 km long embankment (Ardclooney) a trial core trench failed due to inflow of sand. Timber sheet piling was provided along a short section of the embankment only.

In the construction of the Fort Henry Embankment an about 800 metres long core trench was excavated. Also sheet piling was provided over the greater part of the length. A flooding of the reservoir in 1928 delayed construction because of incipient sliding of the then 2.5 m high embankment.



a) Multiple bucket excavating headrace canal



b) Steam shovel excavator

Fig 1. Construction of Shannon Embankments

PERFORMANCE

The Ardnacrusha Project including its embankments has performed extremely satisfactorily over the past fifty years considering the state of the science of soil mechanics at the time of its design. However, some problems have occurred such as slides at the upstream and downstream slopes, slumps and cracks. In Table I, a chronological documentation of stability problems is presented. This list provides valuable background for assessing the cause of stability problems at Shannon Embankments.

TABLE I. Chronological documentation of stability problems

Year	1929-40	41-50	51-60	61-70	71-80
Headrace canal	11	5	4	4	2
Ardclooney	2	4	1	1	0
Fort Henry	5	5	0	1	2
Total	18	14	5	6	4

The number of stability problems has decreased significantly after the first 20 years following filling of the reservoir. Since 1961, a total of ten different cases of stability problems have been observed, ranging from slips or slumps in the vicinity of culverts or areas with known springs, cracks in the embankment or softening of the ground at the toe of the embankment.

Grouting work has been commonly used at Shannon Embankments to repair slumps, slides, cracks or to stop the increasing flow of water in springs. In Table II, the number of grouting projects carried out during the period of 1929-1980 is presented. This information, however, can only provide an approximate indication of the chronological distribution of stability and seepage problems.

TABLE II. Chronological documentation of grouting projects

Year	1929-40	41-50	51-60	61-70	71-80
Headrace canal	11	3	4	0	1
Ardclooney	0	2	2	3	0
Fort Henry	0	1	0	0	1
Total	11	6	6	3	2

The number of grouting projects has decreased significantly after the first decade of operation. It could be observed that in several cases new springs emerged several years after the grouting of springs in the vicinity. Along the headrace canal stability and seepage problems seem often to be concentrated to rather well-defined areas. In some cases, however, problems have occurred after many years of operation and in areas without a previous history of problems.

STABILITY OF FORT HENRY EMBANKMENT

The Slide at Fort Henry in 1948

On the night of 4th January, 1948, a slide occurred, affecting the downstram face over a length of 50 m. Heavy rain had been falling fo some days previously and over 29 mm in the pre ceeding 24 hours. The reservoir level before t slide was at 33.16 m and the level of a small river in the vicinity of the embankment toe wa at 29.1 m. No indication of incipient slope failure, such as cracks or slumps, was observe before the slide. After the slide the top of the embankment (34.80) was lowered to 34.1 m. The whole surface of the slide area was found extremely soft.

After the slide, the reservoir was slowly lowered to 32.6 m. The immediate repair work consisted of temporary measures such as sealing of cracks with dry clay and flattening of stee slopes. For the permanent repair, the slide area was filled with dry bank material to form a flat slope of 5:1 up to 32.0 m. Stone-filled drainage trenches were provided in this area. Above that level the bank was refilled to the original slope of 2.5:1.

Before construction of the embankment in 1926, the surface soil was stripped from the foundation and a longitudinal trench was cut to inte cept long drains and to form a core trench. Timber sheet piles were driven to a depth of 5 m. The material used for construction of the embankment was sandy silt. A soil boring was reported in the vicinity of the slide area, suggesting the following layer sequence: surfa soil (0.5 m), yellow sand (1.5 m), blue gravel (0.5 m), running sand (1.0 m), clay and sand (3.0 m), yellow clay (0.50 m) on coarse gravel

Based on a total stress analysis, assuming undrained shear strength for the embankment and foundation material, a stone berm was designed to increase the factor of safety to approximately 1.45 on a length of 1 200 m. A detailed description of the repair work has been given by Harty (1953).

The failure of Fort Henry provided a unique opportunity to check analysis and investigatio methods to be employed in 1980 for the reevalu tion of Shannon embankments.

Fig. 2 Slide of downstream slope in 1948

SCOPE OF INVESTIGATIONS

General

The soil deposits can be complex and in the reservoir area the basic sequence consist of an initial fill of brown and blue stiff laminated clay, 5-8 cm thick in the boulder clay lined valley. This in turn is cut into by a valley fill of fine sand/sandy gravel. This coarser layer is generally 2.5 m thick but may locally exceed 5-10 m and reach 20 m where it reaches bedrock. Abandonned minor valleys and depressions on top of the sand layer were filled with a complex sequence of soft laminated clays, peaty clays and white lake marls, while the major abandonned valleys are filled with peat 0-6 m thick. Rapid lateral changes in grain size, and consequently of permeability are a typical feature.

The regional ground water system follows generally the flow pattern of the surface water. In the reservoir area, the ground water flows from the surrounding elevated terrain towards the reservoir.

The complex geological conditions and the size of the area to be studied required a detailed planning of the geotechnical field and laboratory investigations. The objective of the investigations was

- to identify potentially unstable embankment sections
- to evaluate possible stabilizing measures
- to propose appropriate measures which at an economically acceptable level assure the long-term stability of Shannon embankments.

The investigations were carried out according to the following programme. At first, a parameter study of typical embankment sections was carried, in order to determine those factors which have contributed to the stability problems in the past. The geotechnical and geohydrological conditions were then established by comprehensive geotechnical and geohydrological field and laboratory investigations. The validity of the stability analysis was then checked at the section of Fort Henry which had failed in 1948. The stability of critical embankment sections was then calculated using different critical shear surfaces. A case with likely input parameters for soil properties and pore water pressure was compared with a case using conservative assumptions. Pore pressures as well as strength parameters were varied.

Based on geological and geohydrological investigations, potentially critical areas were identified. A parameter study as well as past experience indicated that layers or seams of silt and sand in the clay deposit below the embankment are of great importance (Massarsch, 1979). Also the shear strength of the embankment material can to some extent affect the safety factor.

Geotechnical field investigations

Owing to the complex geotechnical conditions, specialized field investigation methods had to be employed in addition to conventional testing techniques. One important aspect was to determine in detail the variation of soil layers and geotechnical properties along potentially critical embankment sections.

In 1980, a comprehensive scheme of field and laboratory investigations was conducted. At the downstream face of the embankment as well as the toe, static cone penetration tests and pore pressure soundings were performed. The penetration resistance (tip and sleeve resistance) and excess pore pressure were measured using vibrating wire gauges mounted in the tip of the penetrometer. The unique feature of the penetrometer (type Geotech) was that the measuring signal was acoustically transmitted without cables along the penetrometer rods to the recording unit at the ground surface.

Careful interpretation of the pore pressure and static cone soundings enabled the varying thickness of sand and clay layers to be accurately established. Permanent piezometers were installed at critical embankment sections to determine the excess pore water pressure in permeable layers, embedded in the cohesive soil deposits. In addition, disturbed and undisturbed samples were obtained from the embankment and foundation material. Disturbed auger samples were taken primarily for soil classification purposes. In clay layers, undisturbed samples were taken using the Swedish Standard Piston Sampler, for laboratory shear and consolidation tests.

The reliability of field investigations could be checked at several sections, e.g. in the vicinity of the slide area at Fort Henry embankment where extensive field investigations had been carried out in the past.

The soil conditions recorded are more complex than reported in the original investigations. Below a 0.4 m thick surface layer of organic soil follow layers of silt, clay, sand and marl of varying thickness down to 3.5 m depth. Between 2.5 and 6.0 m, a sand layer was found, above a deposit of varved, silty clay, which extend to about 11 m below ground surface. The static cone penetration tests indicated that the sand layers near the ground surface down to 3.5 m depth were loose to very loose (<2.5 MPa). The sand layer at depth between 3.5 and 6.0 m was found to be medium dense (q_c = 5 to 10 MPa).

Laboratory investigations

The purpose of the laboratory investigations was to determine the strength properties of soil layers which are of significance for the embankment stability. The effective shear strength parameters were determined on reconsolidated samples under drained loading conditions. In addition to standard triaxial tests, also drained direct static and cyclic shear tests were performed. Stress-controlled as well as strain-controlled tests were carried out and showed good agreement.

In overconsolidated clays the effective cohesion can gradually decrease with time. Therefore, some of the clay samples were consolidated and then sheared to failure under drained conditions. Then the samples were subjected to five undrained load cycles, with the aim to study the possible long-term decrease of the effective cohesion below the embankment. Then the samples were again sheared to failure under drained loading conditions. The shear stress was then plotted against the effective normal stress. In the stability analysis the angle of internal friction and the cohesion intercept were chosen according to the actual stress conditions which could be expected to exist at each of the investigated embankment sections.

The shear tests on embankment material were performed on disturbed samples, recompacted to the same density as determined by field tests. Typical values of the effective angle of friction were approximately $\phi = 20^{o}$ degrees. Two drained direct shear tests on marl yielded an effective angle of friction $\phi' = 20$ to 22 degrees, and a cohesion intercept of $c' = 5$ to 9 kPa. The effective angle of friction in the sand layer near the ground surface was estimated to $\phi = 30^{o}$ and that of the organic soil just below the ground surface to $\phi = 20$ to 22 degrees.

STABILITY OF THE DOWNSTREAM SLOPE

Stability Analysis

The stability analysis was performed in terms of effective stresses, employing a computer program (SSTAB) based on the concept proposed by Morgenstern-Price.

A typical embankment section at Fort Henry is shown in Fig. 3.

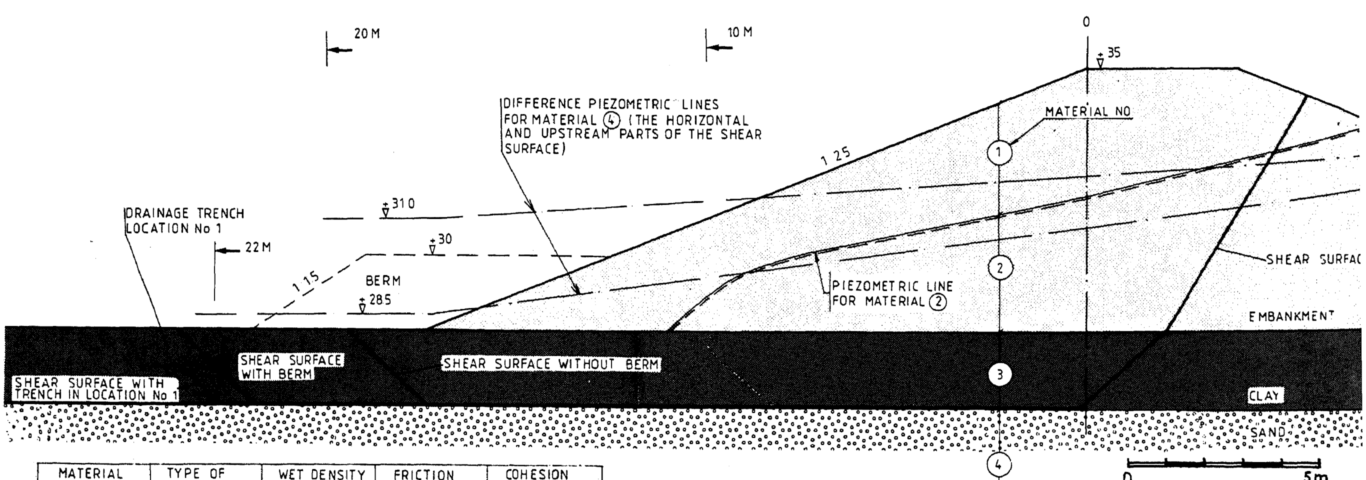
Result of Parameter Study

The results of the stability analysis are summarized in Fig. 4 . The relative significance of various geotechnical parameters on embankment stability are shown.

The stability analysis showed that embankment instability is a result of alternating layers of clay and sand with high pore water pressure below the embankment. Excess pore water pressures are caused by the water pressure in the reservoir and can significantly reduce the embankment stability. The stability analysis als suggests that embankment sections, where drain age trenches have been constructed in the past have a detrimental effect on the downstream stability. The parameter study also provides valuable information on the effect of variations in geotechnical parameters on the analys result.

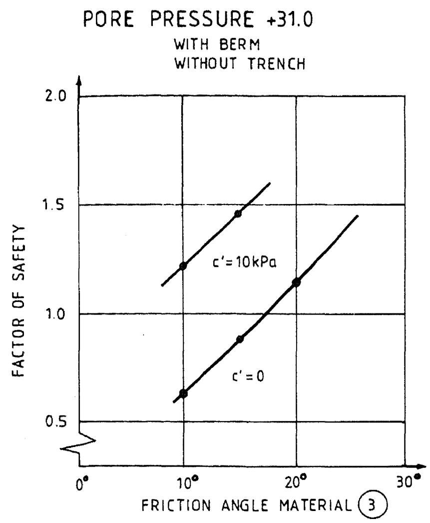
The stability of the downstream slope depends to a large extent on the shear strength of the clay layers below the embankment. Below the central part of the embankment the clay will be normally consolidated, due to the increase the effective stress as a result of the embank ment construction. At the down-stream toe, how ever, as a result of high excess pore water pressures in the pervious layers, the effectiv stress is reduced and the clay becomes overcon solidated. This reduction of effective stress will reduce the shear strength of the clay as the water content gradually increases. There i a further reduction of the shear strength with time from the decrease of the effective cohesi c'. This is partially compensated by an increa in the effective angle of internal friction ϕ by the cyclic loading, resulting from the chan of the reservoir level.

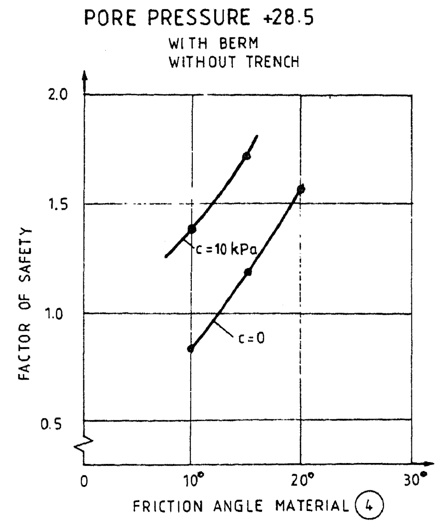
The reduction of the shear strength due to the decrease of the effective vertical stress (increase in pore water pressure) in the soil wil occur first close to the pervious sand or silt layers and seams and gradually spread through the whole layer.



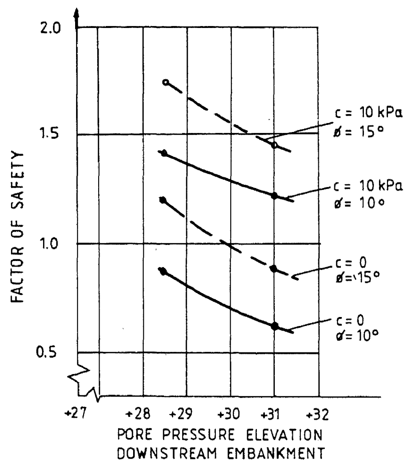
MATERIAL No	TYPE OF MATERIAL	WET DENSITY t/m^3	FRICTION ANGLE ϕ'	COHESION (kPa) c'
1	EMBANKM. MAT. ABOVE GROUND WATER	1.9	20° 30° 40°	0
2	EMBANKM. MAT. BELOW GROUND WATER	2.1	20° 30° 40°	0
3	CLAY LAYER	1.8	10° 15° 20°	0, 10
4	SAND LAYER	2.1	—	—

Fig. 3 Typical embankment section at Fort Henr





INFLUENCE OF PORE PRESSURE
AND SHEAR STRENGTH IN MATERIAL No 3



INFLUENCE OF SHEAR STRENGTH IN THE EMBANKMENT MATERIAL

FRICTION ANGLE	FACTOR OF SAFETY	CHANGES %
30°	1.187	0
20°	1.115	-6
40°	1.260	+6

INFLUENCE OF THE BERM AND TRENCHES
(MATERIAL No 1 AND 2 c'=0, ø'= 15° PORE WATER PRESSURE +28.5)

SLOPE GEOMETRY	FACTOR OF SAFETY	CHANGES %
WITH BERM	1.187	0
WITHOUT BERM	1.080	-10
WITH BERM AND TRENCH IN LOC. No1	1.100	-8
WITH BERM AND TRENCH IN LOC. No2	1.19	0

Fig. 4 Results of parameter study

The pore water pressure below the embankment depends on the occurence of permeable layers which are in contact with the reservoir. The pore water pressure can thus vary significantly with depth.

The magnitude and distribution of the pore water pressure is also affected by the existance of cut-off trenches and sheet pile walls. They will reduce the pore water pressure at the downstream side while the pore water pressure at the upstream side will be increased. Cut-off trences also decrease the effect of the fluctuation of reservoir level.

A gradual increase of pore water pressure may also be caused by oxidation and flocculation of iron components in the ground water. This oxidation effect could be observed in some wells.

As the pore water pressure in the pervious layers gradually increases, there occurs a corresponding reduction of the passive earth pressure and thus of the embankment stability.

The reduction in effective stress in alternating layers of clay and sand below the embankment may be greater than the stabilizing effect of the water pressure, acting at the downstream face, Fig. 5.

The effect of flooding on embankment stability depends mainly on the location of the contact between the clay and permeable layers in the flooded area, and on the geometry of the embankment. Thus, under unfavourable conditions, flooding of the downstream face can initiate embankment failure.

The significance of the shear strength of the embankment material for the stability was in the present case relatively small (about 15%) with respect to base failure, where the critical failure surface follows the soft clay layers below the embankment.

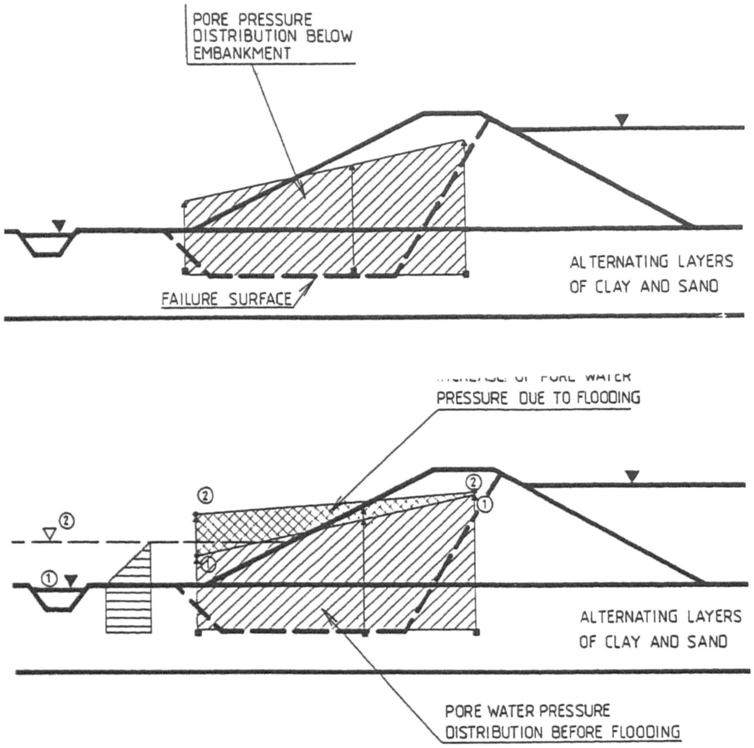


Fig. 5 Schematic sketch of pore water increase below embankment as a result of flooding downstream.

REMEDIAL MEASURES

Two criteria for calculating the factor of safety were chosen. The most conservative assumption of input parameters for the stability analysis was supposed to give a lower boundary for the factor of safety. A realistic assessment of the actual stability conditions was considered to be obtained based on the most likely values for soil properties and pore water pressures. A comparison of the factor of safety for the two cases provided an indication of the sensitivity of the analysis on the chosen input parameters. In all sections, where the calculated safety factor for likely conditions was lower than 1.3, the stability was proposed to be increased by at least 30 percent. At Fort Henry, one of the following measures, or a combination of them was possible

- modification of existing drainage trench system at the toe of the embankment, by providing the trenches with drain pipes and filling with filter material
- stabilizing berms at the downstream face of the embankment
- prevention of flooding at the downstream face of the embankment
- lowering of excess pore water pressure below and at the toe of the embankment by relief wells and horizontal drains.

The final scope of remedial measures included stabilizing berms and filling of the existing open drainage trenches. Lowering of excess por water pressure had been used in the past at Fort Henry with varying success. The long-term performance of deep filter wells is affected b many parameters which may be difficult to control.

The recommendations for remedial works have been fully implemented by the owners with the result that there is full confidence in the integrity and safety of the embankments. A further effect is that surveillance and maintenance work is reduced and the reservoir can be more flexibly operated resulting in financ benefits, Fig. 6.

Fig. 6 Stabilizing berm and drainage scheme at Shannon embankment.

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