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General Report for the Theme Three – Case Histories in Dams, Embankments and Slopes

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Case Histories in Dams, Embankments and Slopes

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SYNOPSIS Of the approximately forty papers submitted for sessions 3A and 3B on Case Histories in Dams, Embankments and Slopes, eighteen specifically discuss dams, foundations for dams or tailings dams. Table 1 presents a capsule summary of those papers that discuss dams and their foundations. Table 2 is a similar capsule summary of the five papers that discuss performance, design and/or construction of tailings dams or spoil piles. This general report summarizes and comments on the papers listed in Tables 1 and 2, presents several abbreviated case histories that parallel those presented in the papers, comments on several points not covered in the various papers, and ends with a few concluding remarks.

COMMENTS ON INDIVIDUAL PAPERS

The following notes and comments are a brief summary of the main points presented in the various papers submitted to this conference regarding earth and earth-rock dams and their foundations.

The paper by MASSARSCH and FIVES, titled, "The Stability of Shannon Embankment" describes the stability and seepage problems which have occurred to the project since construction was completed in 1929. The Ardnacrusha Hydroelectric Project on the River Shannon near the western coast of Ireland, includes a 12 kilometer long headrace canal and reservoir embankment. During the operating life of the project many stability and seepage problems have occurred which required substantial remedial treatment. In 1980 an engineering study was undertaken to determine the remedial measures necessary to assure the owners of the continued safety of the embankment.

The embankments were dumped directly on original ground after stripping of organic matter. The maximum height of the embankment is 18 meters with an average height of about 8 meters.

Many of the problems that occurred since construction was completed have included springs downstream and slides, slumps and cracks on the upstream and downstream slopes. Past repairs included grouting to repair cracks and to stop the increasing flow of water in springs. These repairs were often not of a permanent nature as continuing problems occurred.

Investigations were carried out to

1. identify potentially unstable embankment sections
2. to evaluate possible stabilizing measures, and
3. to propose appropriate economical measures to assure the long-term stability of the embankment.

Comprehensive field and laboratory investigations were conducted. These investigations included the use of the static cone penetrometer with pore pressure measurements. The penetration resistance of the tip and sleeve and the excess pore pressure were measured using vibrating wire gages mounted in the tip of the penetrometer. Measurements were acoustically transmitted without cables along the penetrometer rods to the recording unit at the ground surface.

A series of stability analyses were performed using various geotechnical parameters to estimate embankment stability under varying conditions of foundation shear strength and pore pressure. The analysis showed that embankment instability is a result of alternating layers of soft clays and sand and the high pore pressure below the embankment.

In all sections where the calculated safety factor for the most realistic conditions was lower than 1.3 the stability was proposed to be increased by at least 30 percent. One or a combination of the following remedial measures was considered:

1. Modification of the existing drainage trench system at the downstream toe of the dam.
2. Stabilizing berms at the downstream face of the embankment.
3. Prevention of flooding at the downstream face of the dam.
4. Lowering of excess pore water pressure by relief wells and horizontal drains.

Table 1

CASE HISTORIES—DAMS

TITLE	PERFORMANCE REPORTED: SATISFACTORY	PROBLEMS: EMBANKMENT	PROBLEMS: FOUNDATION	REMEDIAL TREATMENT: EMBANKMENT	REMEDIAL TREATMENT: FOUNDATION	DESIGN/ANALYSIS
THE STABILITY OF SHANNON EMBANKMENTS MASSARSCH AND FIVES	SINCE 1930	SLIDES, SLUMPS AND CRACKS	SEEPAGE	WEIGHTING BERMS	DRAINAGE TRENCH BACK-FILL	BACK-CALCULATION · SLIDES, FIELD/LAB INVESTIGATIONS
CLAY SHALE FOUNDATION SLIDE OF WACO DAM, TEXAS STROMAN, BEENE AND HULL	SINCE 1965	HIGH PORE PRESSURE AND LOW SHEAR STRENGTH IN FOUNDATION CAUSED SLIDE DURING CONSTRUCTION		LARGE UP-STREAM AND DOWNSTREAM BERMS		BACK-CALCULATION SLIDES, FIELD/LAB INVESTIGATIONS
SEALING LEAKAGE OF EARTH DAM BY CONCRETE DIAPHRAGM SHEN AND JIANG	AFTER REMEDIAL TREATMENT	POOR QUALITY CONTROL	INSUFFICIENT FOUNDATION TREATMENT	CONCRETE DIAPHRAGM WALL THROUGH DAM AND FOUNDATION, 12 DAMS TREATED		ALTERNATIVE CUTOI METHODS STUDIED
THE FAILURE OF A SOIL BLANKET LINING CAUSED BY THE ACTION OF BACTERIA PLANT AND VOSLOO	SINCE 1979	EXCESSIVE LEAKAGE THROUGH RESERVOIR LINING		SEEPAGE COLLECTION WITH RETURN SYSTEM		GEOCHEMICAL ANAL'
THE SANTA HELENA DAM ON COMPRESSIBLE FOUNDATION GARGA, ROCHA AND RAMOS	SINCE 1983	2.4m MAXIMUM SETTLEMENT IN FOUNDATION, EMBANKMENT CRACKING		FLAT DESIGN SLOPES	SAND DRAINS	PORE PRESSURE AND SETTLEMENT ANALY.
THE FOUNDATION OF THE RIGHT BANK IN THE WADI-ZARAT DAM BENYLTAYF, ATALLA AND RABAH	SINCE 1982		CALCAREOUS CRUST SUSCEPTIBLE TO SETTLEMENT PIPING AND DISSOLUTION		DEEP WIDE CORE TRENCH THROUGH CRUST	ANALYSIS OF GROUT AND SLURRY WALL A ERNATIVES, OBSERV. TIONS DURING RESEF VOIR FILLING
PECULIAR BEHAVIOR OF THE MANICOUAGAN 3 DAM'S CORE DASCAL	SINCE 1975	HORIZONTAL ZONES OF WEAKNESS IN CORE WITH HIGH PORE PRESSURES IN DOWNSTREAM PORTION OF CORE		NONE REQUIRED		BACK ANALYSIS OF CONSTRUCTION RECORDS AND OF PERFORMANCE AS MEASURED BY INSTRUMENTS
RECONSIDERATION OF FAILURE INITIATING MECHANISMS FOR TETON DAM LEONARDS AND DAVIDSON		SUBSIDENCE OF PERMEABLE LAYERS, HYDRAULIC FRACTURE	OPEN JOINTS IN UPSTREAM AND DOWNSTREAM CORE TRENCH WALLS			DETAILED BACK-ANALYSIS OF CONSTRUCTION RECORD: AND OF PREVIOUS INVESTIGATIONS
PERFORMANCE OF SOME DAMS IN INDIA RAMAMURTHY	SINCE MID 1950'S	GOOD PERFORMANCE REPORTED		NONE REQUIRED		BACK ANALYSIS OF PORE PRESSURE AND SETTLEMENT AT FIVE DAMS DURING CONSTRUCTION AND PRO JECT OPERATION
BEHAVIOR OF RAMGANGA DAMS LAVANIA	SINCE 1975	GOOD PERFORMANCE REPORTED		NONE REQUIRED		ANALYSIS OF MEASU STRESSES, SETTLEME AND PORE PRESSURE
		LOW EFFECTIVE VERTICAL STRESS IN CORE	SEEPAGE CONTROL DURING CONSTRUCTION		USED DRAINAGE ADITS IN ABUTMENTS TO REDUCE UPLIFT	
EFFICACY OF GROUT CURTAIN AT RAMGANGA DAM GOEL AND SHARMA	SINCE 1975		SINGLE LINE GROUT CURTAIN INEFFECTIVE IN REDUCING DOWNSTREAM PORE PRESSURES	NONE REQUIRED		ANALYSIS OF SINGLE LINE GROUT CURTAI AT MAIN DAM AND U STREAM BLANKET A' SADDLE DAM
EARTH DAMS AT NUCLEAR POWER PLANTS PICHUMANI, GUPTA AND HELLER	SINCE 1977	GOOD PERFORMANCE REPORTED		MODEST REPAIR OF EROSION AROUND RESERVOIR SHORELINE		HIGH STANDARDS AI PLIED TO DESIGN AN CONSTRUCTION OF C FOR EMERGENCY CO COOLING WATER, CAREFUL INSPECTIO AND MAINTENANCE PROCEDURES EMPLO
BEHAVIOR OF TWO BIG ROCKFILL DAMS, AND DESIGN AIMS DE MELLO		GOOD FIELD PERFORMANCE REPORTED, PREDICTED VS. MEASURED STRAINS ARE NOT IN GOOD AGREEMENT		NONE REQUIRED FOR CASES REPORTED		USE DATA FROM EXI. ING DAMS COUPLED \ PLAUSIBLE MODELS ' PREDICT PERFORMAI MODIFY PREDICTION BASED ON OBSERVA' FROM THE START OF STRUCTION THROUG PROJECT OPERATION

Of these methods, stabilizing berms and modifications including filling of the existing open drainage trenches was chosen. Because of varied success in reducing excess pore water pressures in the past this particular technique was not chosen.

The paper by STROMAN, BEENE and HULL titled, "Clay Shale Foundation Slide at Waco Dam, Texas" describes a major slide which occurred during construction of the dam, the analysis of the cause of failure, and the remedial treatment utilized to stabilize the dam and its foundation.

Waco Dam is a rolled earth fill structure about 18,000 ft. long with a maximum height of 140 ft. Throughout the early project investigations and design studies it was assumed that the entire foundation for the embankment except the upper portion of the steep right abutment would be on a single hard dense shale formation. The shale within the river valley was masked by a 10 to 40 foot thick layer of alluvium. The shale was usually penetrated only 10 to 15 ft. by borings spaced on nominal 500 ft. centers. The designers were aware that the project was within the influence of the Balcones Fault Zone but no evidence was detected in any of the borings that faulting occurred in the structure foundation areas.

The original embankment design included 1 vertical to 3 horizontal and 1 vertical to 4.5 horizontal upstream slopes and 1 vertical to 2.5 horizontal and 1 vertical to 3 horizontal downstream slopes. Embankment zoning called for an upstream and central impervious core with a downstream compacted shale zone. An inclined chimney drain and horizontal drainage blanket were incorporated in the downstream section of the dam. The foundation shales were not recognized as controlling the stability of the dam. The potential for high pore pressure development in the shales as a result of embankment loading was not recognized during the original design.

When the embankment height was approximately 85 ft. a series of cracks formed at the top of the fill. Cracking increased during an approximately 10 day period in October, 1961, followed by accelerated movement over the next several days resulting in cumulative movement at the crest of the fill of more than 20 ft. The configuration of the embankment after failure is illustrated on Figure 1. The base of the sliding mass coincides approximately with the middle of the Pepper clay shale stratum.

An extensive investigation was immediately undertaken to determine the cause of the slide and to evaluate foundation and embankment conditions for redesign of the project. The field investigations included drilling large diameter inspection holes, core borings, holes for electrical resistivity logging, and borings

Table 2
CASE HISTORIES—TAILINGS DAMS

TITLE	PERFORMANCE REPORTED: SATISFACTORY	PERFORMANCE REPORTED: PROBLEMS	REMEDIAL TREATMENT	DESIGN/ANALYSIS
FAILURE OF MICACEOUS WASTE TAILINGS DAM BRUMOND	UPON COMPLETION OF RECONSTRUCTION	SLUMPING FAILURE OF 200 FT. SECTION OF DAM CONSTRUCTED USING THE UPSTREAM METHOD	EXCAVATION OF SLIDE DEBRIS; RECONSTRUCTION OF SLOPE TO 2H:1V; QUARRY ROCK USED AT TOE	ANALYSIS OF WASTE PILE PRIOR TO THE SLIDE INDICATED THAT FURTHER LOADING WOULD JEOPARDIZE STABILITY
THE USE OF LIMITED FIELD OBSERVATION IN REMEDIAL DESIGN SULLY AND Mc PHAIL	UPON COMPLETION OF RECONSTRUCTION	SLIDE ON STEEP SLOPE OF TAILINGS PILE, FAILURE CAUSED BY HIGH PHREATIC SURFACE EXISTING ON THE LOWER STEEP SLOPE	NOT DESCRIBED	BACK ANALYSIS OF SLIDE; ANALYSIS OF PREDICTED FUTURE CONDITIONS; EMPHASIS ON OBSERVATIONS TO VERIFY DESIGN
STABILITY OF AN ERRATIC TAILINGS DEPOSIT KOTZIAS STAMATOPUOLIS AND KARAS	SINCE COMPLETION OF WASTE PILE IN 1983	DEEP-SEATED SLIDE IN 1974; NUMEROUS SURFICIAL SLIDES AND CRACKS AT THE TOP OF THE SLOPE	PILE CONSTRUCTED WITH SLOPES YIELDING A FACTOR OF SAFETY OF 1.06	STABILITY ANALYSIS OF PILE AT MID-HEIGHT USED TO PREDICT SLOPES FOR FUTURE RAISING OF THE PILE
EVOLUTION OF DESIGN AND CONSTRUCTION OF LORNEX L-L TAILINGS DAM SCOTT AND LO	SINCE STARTUP	LOW SHEAR STRENGTH MATERIALS IN FOUNDATION OF STARTER DAM; EVOLUTION OF AN ECONOMICAL YET SAFE TECHNIQUE FOR DAM CONSTRUCTION	NO TREATMENT REQUIRED	DESIGN AND ANALYSIS PROCEEDING AS DAM IS CONSTRUCTED; ECONOMIES TO RESULT FROM ANALYSIS OF PERFORMANCE
THE BRENDA MINES' CYCLONED-SAND TAILINGS DAM KLOHN	SINCE LATE 1960'S	SEEPAGE AND PIPING; DETERMINATION OF MAXIMUM HEIGHT	NO TREATMENT REQUIRED	ANALYSIS OF PERFORMANCE THROUGHOUT CONSTRUCTION; DETAILED SEISMIC ANALYSIS

to sample foundation materials and for installation of piezometers and slope indicators. The studies revealed that significant faulting was involved in the embankment foundation and that the faulting had a significant influence on the location and magnitude of the slide. As a result of the faulting the relatively soft Pepper clay shale occurred in the foundation immediately beneath the dam in a reach of approximately 700 ft. The piezometers installed subsequent to the failure showed a high distribution of pore pressure. At the Pepper/Del Rio contact excess pore pressures at the axis of the dam were about 100 percent of the pressure applied by the embankment. The investigations also indicated that the shear strength of the Pepper shale was very low. Direct shear tests performed on pre-cut specimens showed friction angles varying from 7° to 9° with no cohesion.

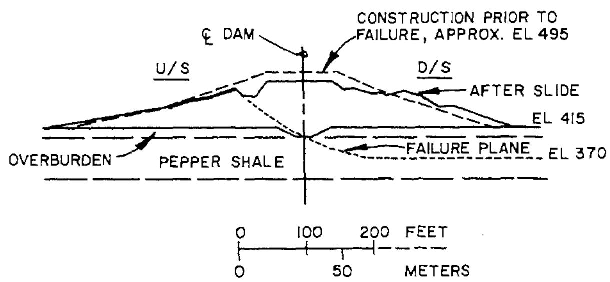


Fig. 1 WACO DAM, TEXAS–EMBANKMENT AFTER FAILURE

Redesign of the embankment considered the high pore pressure beneath the dam and the low strength. Wide weighting berms with slopes of 1 vertical to 20 horizontal were chosen as the main remedial treatment (see Fig. 2). The stability analysis of the section with berms produced a factor of safety of 1.15. It was recognized that this factor of safety would not ordinarily be sufficient but because of the extensive knowledge of the subsurface conditions and because of the use of a field strength derived from analysis of the slide it was considered to be adequate.

The paper by SHEN and JIANG titled, "Sealing Leakage of Earth Dam by Concrete Diaphragm" discusses 12 dams in the People's Republic of China which have been successfully treated by the use of cast-in-place concrete diaphragms located at or near the axis of the dam. The diaphragms were used to seal leakage through concentrated seepage paths through the earthfill or through the alluvial material in the dam foundation. The defects in these dams include seepage channels through concentrations of coarse gravelly material in the fill, tensi cracks in cohesive fills and permeable alluvi in the dam foundation. In many cases the defects were a direct result of poor quali material being used in the dam and inadequa treatment of alluvial foundations. The Chine experience indicates that cast-in-place concre diaphragm walls can be used to repair emban ments and their foundations on a routine basis

At several projects, longitudinal cracking the embankment at the crest and leakage drilling fluid and liquid concrete from t slurry trench took place. These cracks a attributed to hydraulic fracture of the fi because of low lateral stress at the cres Cracks were roughly parallel to the axis of t dam and extended downward to 7 meters or mor The width of the cracks were variable; the mo serious were 25–30 cm. wide. Cracks fir appeared when the slurry was introduced duri trench excavation. Cracks filled with slur then opened further during concrete pouring.

The following modified construction procedure were used to control the cracking problem:

1. Control of the maximum level of the slurı in the trench.

2. Control of the concrete placement rate t one meter per hour.

3. Use of primary, secondary, and tertiar panels, thus increasing the spacing betwee active panels.

4. Stop construction when slurry leakag occurred.

A very interesting case history is presented b Messrs. PLANT and VOSLOO in their paper titled "The Failure of a Soil Blanket Lining Caused b the Action of Bacteria". Treated sewag effluent was used as construction water for th earth embankments and a soil blanket floo lining for a raw water storage reservoir i South Africa.

During the first filling of the reservoir, flow from the blanket drain outlet increased and we areas were apparent around the perimeter of th embankment. Filling was halted when th reservoir was approximately two-thirds full Pore pressure measurements beneath the embank ment provided strong evidence of substantia leakage through the floor lining.

Subsequent analysis of the failure indicate that bacteria within the sewage effluent use

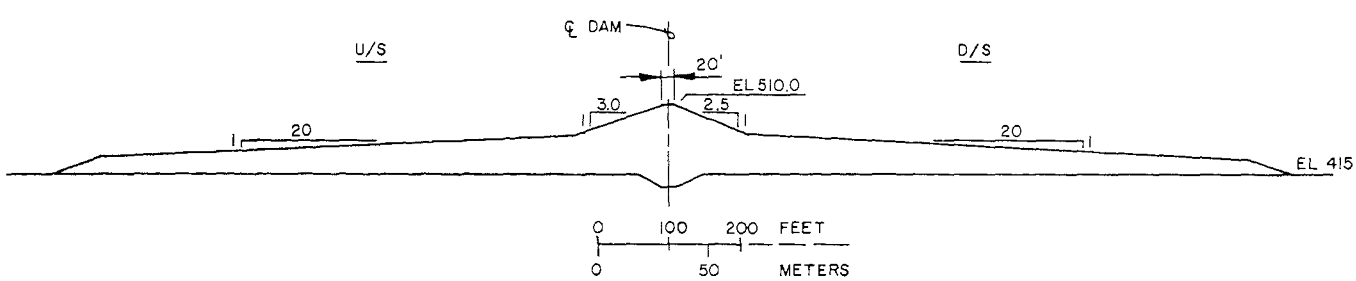


Fig. 2 WACO DAM, TEXAS–REDESIGNED EMBANKMENT SECTION, AS BUILT

during the construction of the lining and embankment was the main cause of the failure. Bacteria within this water converted the ammonia and organic nitrogen to nitrite and subsequently to nitrate under aerobic conditions. Bacteria then converted nitrate to nitrogen gas under anaerobic conditions. The gas thus generated expanded and loosened the soil structure in the floor lining leading to high permeability as demonstrated by the testing of block soil samples. By this action, the soil blanket was made ineffective in reducing seepage. Remedial treatment consisted, simply, of collecting the seepage and pumping it back into the reservoir.

Messrs. GARGA, ROCHA, and RAMOS present the performance of a 28 meter high compacted earthfill dam founded in part over soft to very soft silty clays in their paper titled, "The Santa Helena Dam on Compressible Foundation". For economic reasons a portion of the dam fill was founded directly over an 8 meter thick deposit of soft compressible clay. Large diameter sand drains were installed in the soft clay to accelerate the consolidation and to develop adequate shear strength during construction of the dam. Large settlements were expected and subsequently measured. Typical blow counts in the soft clayey deposit ranged from 1 to less than 1 blow per foot. Instrumentation was used to control the rate of construction and to assess the stability of the dam during its construction. Pneumatic piezometers, open standpipe piezometers and several instrumentation types to monitor settlement were installed. A maximum of about 2.5 meters of foundation settlement occurred during construction as compared to an estimated settlement of somewhat in excess of 4 meters, see Fig. 3.

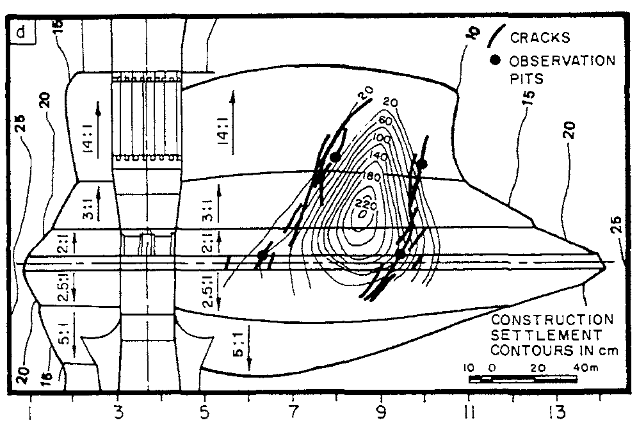


Fig. 3 SANTA HELENA DAM, BRAZIL CONSTRUCTION SETTLEMENT IN FOUNDATION AND OBSERVED CRACKS

Cracking of the fill first occurred when the dam height reached approximately 10 meters. Subsequently, at the completion of the dam, additional cracks were observed. The cracks were a maximum of 3.5 meters deep and were generally of hairline width. Occasionally cracks to a maximum opening of 2 centimeters were observed. Cracks were cleaned by injecting water under low head and subsequently filled with a 5 to 7 percent bentonite mix.

The project has been operating satisfactorily since October 1983 with a reservoir 19 meters above stream bed. Seepage is modest with approximately 4.4 liters per second being measured.

Authors BENLTAYF, ATALLA, and RABAH in their paper, "The Foundation of the Right Bank in Wadi-Zarat Dam" describe the engineering characteristics of a calcareous crust that formed the foundation of the right abutment of the Wadi-Zarat Dam, in Libya. This crust is extremely heterogeneous with potential for collapse on wetting. Locally, it is highly permeable with evidence of dissolution features.

Three proposed methods of treatment were evaluated.

1. A multi-line grout curtain penetrating the calcareous crust to the underlying competent foundation material.

2. A bentonite-cement slurry wall through the crust and anchored in the underlying rock.

3. Excavation of the crust below the center of the dam such that the core and filters are founded on the underlying dolomite.

Excavation of the crust was adopted.

Grout tests indicated that improvement was not significant and that the crust was generally not groutable with ordinary cement grouts. The slurry wall concept was abandoned partly because slurry trench technology is not available in Libya and partly because of the potential for differential settlement to occur under the dam. Figure 4 illustrates the adopted solution showing that more than half of the dam is backfilled in the deep wide core trench. This solution has been commonly used at other dams and represent a positive means to treat a difficult foundation.

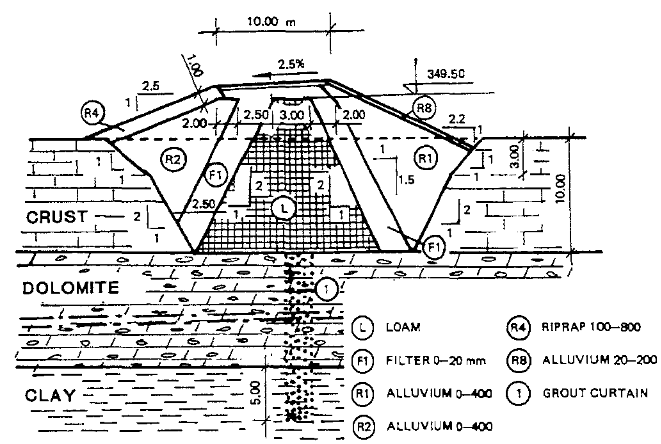


Fig. 4 WADI–ZARAT DAM, LIBYA–CROSS SECTION OF RIGHT BANK AT STATION 1915, AFTER REMOVAL OF CREST

The paper titled, "Peculiar Behavior of the Manicouagan 3 Dam's Core" by DASCAL presents a description of horizontal zones of weakness which led to high pore pressure development in the downstream part of the relatively wide core of the dam. The core of the dam consists of well graded till with approximately 50 percent silt and clay size particles. In general the material passing the No. 40 sieve is non-plastic; real clay minerals in the well graded till are limited. The material in the lower portion of the core was placed wet of optimum. Subsequently the core material was dried in a rotary kiln reducing its moisture content substantially. The average degree of compaction reached approximately 98 percent (Standard Proctor) at an average water content of approximately 1 percent over the optimum. The filter transition zones on both sides of the core consist of well graded sands and gravels compacted to an average relative density of 88 percent. In the lower somewhat wetter part of the core, the pore pressure ratio, r_u (r_u = $u/\gamma h$ where u = pore pressure, γh = total weight of overlying fill) reached 0.40 - 0.55. At higher elevations, an $r_u < 0.20$ was generally obtained.

Upon completion of reservoir filling the piezometers in the downstream portion of the core recorded high pore pressures i.e. only a few feet below the reservoir level. Analysis of this condition included evaluation of the piezometric, stress and settlement data. In addition, a series of holes were drilled through the core to determine the nature and characteristics of the core material downstream from the centerline. Zones of soft and permeable materials were found at depths below about 100 ft. These apparently continuous weak layers consisted of loose muddy material that transmit the reservoir pressure with little loss of head. The permeability of these layers exceeded by a thousand times the permeability of adjacent more dense material. The author suggests that these soft zones started during construction as horizontal cracks caused by arching of the core between the much less compressible filter transition zones and shells. Significant differences in compressibility of these two materials results from the relatively high water content of the compacted till core and the high relative density of the compacted filter transition zones. Arching is further confirmed by the low vertical effective stresses of $0.2\,\gamma h$ measured over the top of the inspection gallery at the base of the dam.

The presence of these soft and permeable zones does not represent any danger to the stability of the dam. The wide filter transition zones and a downstream shell consisting of sands and gravels precludes piping of the core material. No phreatic water surface is present in the downstream shell and therefore it can be assumed that the discharge through these zones of weakness is relatively small.

LEONARDS and DAVIDSON in their paper titled, "Reconsideration of Failure Initiating Mechanisms for Teton Dam" review the failure mechanisms previously suggested for Teton Dam and suggest a failure initiating mechanism which they believe satisfies all of the available data. The authors summarize and review the findings of the Independent Panel and of the Department of the Interior review group. They conclude that several of the findings of the Independent Panel and of the Interior review group are not fully supported by all of th data.

The authors studied the construction recorc with respect to compaction and moisture conter of layers contained within the dam in the are where failure occurred. Upon close examinatic of this data and subsequent additiona laboratory investigation the authors concluc that a high permeability layer could have bee built completely across the right key trenc fill, this material having been compacted wel on the dry side of optimum. This more permeabl material could have collapsed or subsided c wetting. Tests on core material taken from we seams on the left bank indicate at 2.6% reduc tion in volume when compacted to 93% of Proctc maximum at 3.5% dry of the optimum moistur content. This amount of collapse was considere excessive. The authors conclude that

> "subsidence or collapse of a permeable dr side compacted layer spanning the width c the key trench of the right abutmen permitted hydraulic fracturing (o separation) to occur in the key trench fil thereby allowing flow from open joints i the upstream wall (with access to th reservoir) to open joints in the downstrea wall".

They go on to say that,

> "only the combination of a permeable laye collapsing on saturation and open joints i the walls of the key trench with access t the reservoir, could provide the necessar discontinuity and sufficient reduction i stress on horizontal planes to induc hydraulic fracturing and result in so rapi a failure of the dam".

The authors basically agree with Seed an Duncan's list of lessons to be learned from th Teton Dam failure

> "especially with the principal of 'multipl lines of defense' advocated by Carl Terzagh and Arthur Casagrande."

In his paper, "Performance of Some Embankmen Dams in India" RAMAMURTHY reviews the perfor mance of five earth and earth-rock dams. Th dams are located in varying geologic environ ments and have both thin and thick core zones In all of the cases presented, performance ha been quite satisfactory with deformations an pore pressures showing more or less predictabl results.

At Beas Dam, a 132 meter high embankment, th maximum cumulative embankment settlement wa about 2.5 percent. At Ramganga, a 128 mete high embankment dam, the maximum vertica settlement was about 1.8 percent. The autho reports a tendency for displacements to b directed towards the center of the valley whe the valley side walls are steep whereas thi type of deformation did not occur for dams i broad valleys.

LAVANIA in his paper titled, "Behavior o Ramganga Dams" reports on the performance of tw earth and boulder fill dams in India. Both dam have cores consisting of crushed clay shal encased in compacted crushed sandstone fill. No major problems occurred either during construction or during project operation. Stress meters installed within the core of the main dam show effective stresses less than 1/2 of the overburden effective stress thereby indicating arching caused by the interaction between the clayey core and the sand shells of the dam. Study of the development of pore water pressures during the last nine reservoir fillings indicates that saturation of the clay core adjacent to the upsteam sand fill has started, but that the phreatic line has not yet developed through the core.

GOEL and SHARMA in their paper titled, "Efficacy of Grout Curtain at Ramganga Dam" analyzed the foundation piezometer records at the main dam and saddle dam. The data indicates that the single row grout curtain at the main dam constructed in friable sandstones and clay shales is ineffective with respect to hydrostatic pressure reduction in the foundation downstream from the grout curtain. No piezometers are available to evaluate the effectiveness of the curtain in areas where five rows of grouting was needed because of substantial fracturing of the foundation rocks. Under similar foundation conditions at the saddle dam the upstream impervious blanket has proved to be more effective in foundation pore pressure reduction.

These findings at Ramganga are not dissimilar from the experience obtained at other dam sites where foundation rocks do not contain major defects.

PICHUMANI, GUPTA and HELLER in their paper titled, "Earth Dams and Nuclear Power Plants" describe the general practice of the U.S. Nuclear Regulatory Commission in its review of the design and analysis of dams associated with nuclear power plants. The authors describe an example of an earth dam including some of the unusual design features incorporated in the structure. The authors state

> "where well established procedures have been properly implemented during design, analysis and construction and when supported by field monitoring and periodic inspections, the dams have not only performed adequately, but have enhanced public confidence in their integrity and in the overall safety of the associated power plants".

Performance monitoring of these dams is considered essential to insure that they remain functional and impound the required quantity of cooling water for safe shutdown of the nuclear plant in the event of an emergency. In many cases seepage tests are conducted after the initial filling of the dam to estimate the rate and quantity of water loss due to seepage from the reservoir. Recently, increased attention has been focused on probabilistic risk assessment techniques to characterize uncertainties and to establish probability models for evaluating soil and rock parameters associated with geotechnical and dam engineering. The authors provide a good summary of sound engineering practice for dam engineering in general. Their outline of points to be considered should be used by all engineers in the dam design profession.

Dr. de MELLO in his general lecture on the "Behavior of Two Big Rockfill Dams, and Design Aims" presents the case histories of two major embankment dams that have behaved quite well with respect to all major points of concern. I cannot attempt in this summary of technical papers to fully describe or even briefly describe the many points made by Dr. de Mello. Much of his paper is devoted to the compressibility and deformation of embankment dams as observed in the field versus predictions of compressibility based on laboratory test data. He points out that predictions based on laboratory data often overestimate field compressibility by factors of three of more. He states that sophisticated laboratory tests on undisturbed specimens from block samples taken from the dam offer some improvement to the prediction of prototype behavior. He believes, however, that the recompression strain curves from routine odometer tests on undisturbed block samples come adequately close to representing field behavior and suggests that it is preferable to employ these more routine tests for a more rapid and economical determination of field behavior.

In response to longitudinal cracks that have been observed at the crest of dams, Dr. de Mello suggests minimizing the number of zones in the top 20 meters of a high dam and moving toward an embankment section in the crest which is more homogeneous. Figure 5 illustrates this concept.

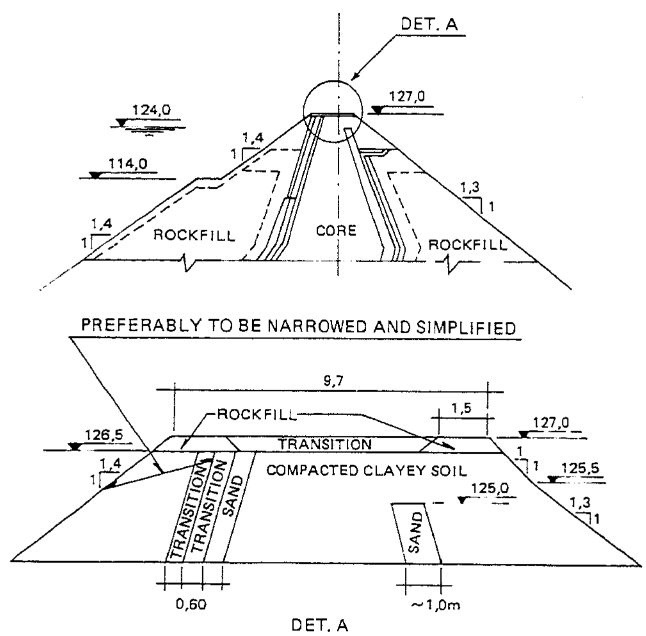


Fig. 5 SUGGESTED DESIGN REVISION NEAR CREST

The paper titled, "Failure of Micaceous Waste Tailings Dam" by BRUMUND, describes the failure and reconstruction of an 80 foot high, eight acre, tailings impoundment in North Carolina. The dam was constructed using the upstream method of construction. The outer layer of coarse to fine sand tailings was hauled to the dam and placed and spread with a Caterpillar D8 dozer; the only compaction effort was that obtained by the bulldozer. The fine fraction tailings consisted of low density, layered fine sands and sandy silts with frequent interbedding of micaceous silty clays.

Stability analysis prior to the failure indicated a factor of safety slightly above 1.0. It was concluded that the dam was only marginally stable and should not be raised further. During the following seven or eight months the dam was raised several times to an elevation eight feet higher than at the time the analyses were made. Following rains and erosion on the outer face of the tailings dam, a 200-foot long section of the dam failed resulting in about 50,000 cubic yards of tailings sliding and slumping into the adjacent river.

Repairs consisted of cleanup of the slide debris and reconstructing the failed section of the dam. Slopes were flattened to two horizontal to one vertical which required some placement of fill in the river bed.

Figure 6 indicates the position of the phreatic surface within the tailings dam and the values of standard penetration tests. SPT values within the slimes range from one to five blows per foot indicating very soft low strength and low density materials.

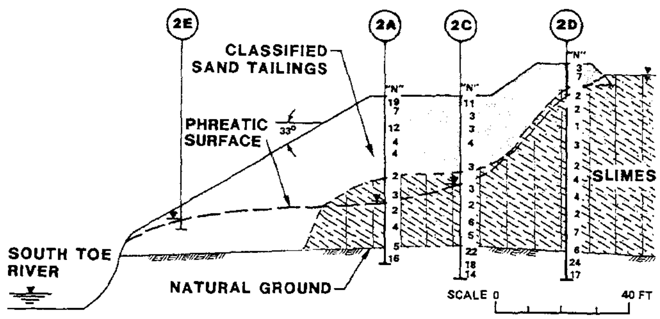


Fig. 6 MICACEOUS WASTE TAILINGS DAM, NORTH CAROLINA SECTION THROUGH DAM NEAR FAILURE ZONE

Authors SULLY and MCPHAIL in their paper titled, "The Use of Limited Field Observation in Remedial Design" describe the investigations carried out to determine the present and future stability of a gold tailings dam in South Africa. The investigations were performed after a slope failure occurred in the outer wall of the tailings dam. The failure itself was small and relatively easily repaired. Its importance lay in the implications it held with respect to the present and future stability of the dam. The failure occurred in an area with a steep average slope, i.e. 52 to 55 degrees, where continual deposition of slimes occurred during the previous nine days and where high groundwater conditions in the outer slope of the pile were observed. Several days of heavy rainfall preceeded the failure. The authors back-analyzed the failure using finite element methods to estimate the phreatic surface prior to the time of failure. They then studied the effects of increasing the tailings dam height by 20 meters with superimposed seepage conditions within the pile assuming the edge of the water surface within 110 meters of the outer crest the transient condition with edge of pool witl 20 meters of the outer slope.

Among their conclusions the authors state tl continued monitoring must be carried out verify the assumptions used in the analys especially with respect to predictions of futı conditions. This is essential. It is extreme difficult and sometimes impossible to predi future conditions accurately, especially changes in the mill occur or if production scheduled to be increased.

In the paper titled, "Stability of an Errat Tailings Deposit", KOTZIAS, STAMATOPOULOS a KARAS describe the investigations performed analyze the large spoil pile from the Kard lignite fields in Greece. Investigations we carried out when the pile was about half i ultimate height of 73 meters. Convention geotechnical studies were conducted includi drilling, sampling and testing of material Measurements of the water table within the pi and standard penetration testing were perform in all of the holes. Using this data, ba analysis of a deep slide that had occurr previously was conducted and studies of t surficial slides that frequently occurred we made.
The analyses and investigations allowed accurate selection of strength parameters f use in evaluating the safety of the pile at i ultimate height. Various outer slope configur tions were used to study the stability of t pile at its ultimate height. The pile slo geometry finally selected yielded factors safety of 1.06. The pile was successful constructed using procedures and outer slo geometry as recommended by the authors.

Authors SCOTT and LO in their paper title "Evolution of Design and Construction of Lorn L-L Tailings Dam" describe the design a construction phases of a major tailings dam British Columbia. Design and construction this tailings dam has been adapted to meet t conditions encountered and to provide a safe y economical structure to retain the mi tailings. The tailings dam was initiated with starter dam consisting of a glacial till co and outer slopes of random fill placed on foundation of soft swamp deposits underlain glacial tills. Subsequent raises of tl tailings dam are being constructed using t centerline technique and double cycloning of t sands to obtain relatively clean sands f construction of the outer slope, see Figure The swamp deposits are being excavated fr beneath the downstream sand shell of the daı The downstream shell of the dam involve placement of the sands by hydraulic means cells measuring 120 meters long and 60 fe wide. The sands are spread and compacted wi dozers. Construction and distribution tecl niques are continually being monitored t evaluate performance of the structure and experiment with less costly techniques fc construction.

Options under active consideration include:

1. Incorporating a substantial zone c hydraulically placed uncompacted tailinc sands in the downstream zone above tł

saturation line.

2. Elimination of the glacial till core and using instead a wide tailings beach as the seepage barrier; and

3. Developing more cost effective methods in placing the sand fills.

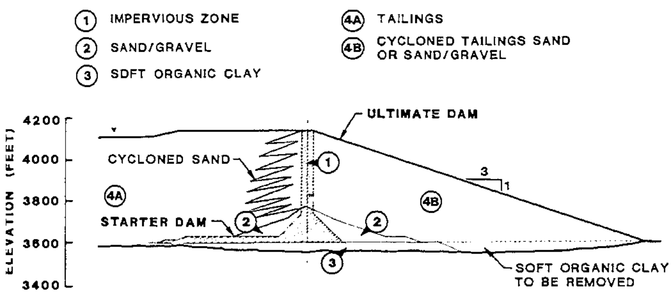


Fig. 7 TYPICAL SECTION THROUGH ULTIMATE LORNEX L—L TAILINGS DAM

The centerline construction technique as employed at Lornex and other major tailings develops a fundamentally safer structure than one constructed using the upstream method. In the centerline technique, classified sands form the entire downstream shell of the dam; drainage, compaction, and a core, as at Lornex, can be incorporated if desirable. With the upstream method, an outer skin of coarser tailings is founded on finer grained soft weak layers. The inter-fingering of the fine and coarse materials at low density can result in highly suspect predictions of present and future stability.

In his general lecture titled, "The Brenda Mines Cyclone-Sand Tailings Dam", EARLE KLOHN presents a detailed description of the design construction and performance of a large tailings dam in British Columbia. The author begins with an overview of tailings dam design and construction in common use today. He points out that tailings dam design differs from conventional dam design in three important ways.

1. The material stored behind the dam is soft, loose, relatively impervious tailings rather than water. These semi-fluid materials under severe seismic shock are likely to liquify and become a fluid with high unit weight exerting additional thrust on the dam.

2. A large part of the dam is usually constructed with the coarser sand fraction of the tailings.

3. Most of the dam construction is carried out by mining personnel as a part of the overall tailings disposal operation with the dam being continuously raised as required to stay ahead of the rising tailings pond.

The author makes the point that because tailings dam usually are constructed slowly over a period of many years, the designer is able to select a design and then check its performance making modifications as required throughout the long construction period. He notes that this is a critically important aspect of tailings dam design. Unfortunately the designer is not always able to follow the performance of a tailings dam during construction simply because the owner does not retain the services of the designer throughout the construction period.

In his overview Mr. Klohn summarizes the basic tailings dam designs currently in use including the upstream tailings dam, the downstream tailings dam and the conventional water storage dam. He discusses the disadvantages of construction using the upstream method and lists the advantages of the downstream centerline technique. These advantages include

1. construction over prepared foundation.

2. placement and compaction control can be exercised as required.

3. underdrainage facilities can be installed.

4. the dam can be raised above its originally planned design height with a minimum of problems and design modifications.

The Brenda Mines tailings dam is located in southcentral British Columbia in an area where 60% of the precipitation falls as snow during the period November to April. Seismicity studies indicate that the maximum credible earthquake could be a Richter magnitude 6.5 event with an epicenter conceivably as close as 11 miles from the dam site. The dam was begun in the late 1960's and was designed to be constructed to a maximum height of approximately 400 feet. Recent studies including state-of-the-art seismic analyses indicate that the dam can be raised to an ultimate height of 530 feet.

Mr. Klohn presents a very thorough description of the original design and construction history of the dam and some of the problems which occurred during the construction including small failures caused by seepage and piping. Detailed data on the density, strength and other properties of the tailings dam are presented along with the analysis of the seismic stability of the structure. His paper is required reading for all tailings dam design engineers.

SOME ADDITIONAL CASE HISTORIES

Rio Lindo Dam, Honduras

The 20 m high Rio Lindo Dam in Honduras is founded on a soft, highly compressible foundation of colluvium and volcanic ash layers. The ash consists of a loose, open structure of silt-size particles with a dry unit weight commonly less than 60 pounds/ft^3. Free water could easily be squeezed by hand from the sample. Large settlements during construction were anticipated.

The design of the dam, see Fig. 8, included weighting berms upstream and downstream at 8 horizontal to 1 vertical and a compacted silty clay facing placed after completion of the main body of the dam.

Settlements and pore pressures were measured in the foundation during construction. Maximum settlement exceeded 2.0 meters prior to placement of the clay facing and slope protection. Subsequent settlements during construction of the clay facing were modest; no cracking was observed because of the granular cohesionless body of the dam and the placement of the clay facing after nearly all of the settlement had taken place.

Excess pore pressure within the foundation was negligible. Although it had been anticipated that construction placement rates might require control, the rapid dissipation of pore pressure with load precluded any necessity for construction control.

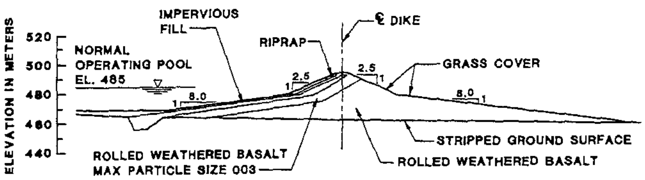


Fig. 8 RIO LINDO HYDROELECTRIC PROJECT, HONDURAS TYPICAL DIKE SECTION

Mangla Dam, Pakistan

Low bedding shear strength of clay shales markedly affected the design of structures at the Mangla Project in Pakistan. During the first months of construction, it became evident that bedding shears were present in rocks with regional dips of only 10° or 12°. This prompted an extensive program of sampling and testing, a major change in design criteria and substantial changes to cut and fill slope design. Mangla, completed in 1967, is operating successfully without difficulty.

The Mangla project is sited in low hills composed of beds of the Siwalik system. The Siwaliks are fresh-water sediments deposited by streams rejuvenated by a major uplift of the Himalayas. The area is seismically unstable and destructive earthquakes occur from time to time.

The Siwaliks consist of intercalated beds of sandstone, siltstone and clay. Some of the beds are over 100 ft. in thickness and can be traced individually over long distances but, because the deposits are of fresh-water origin, most of the thinner beds are lenticular and pinch out rapidly.

The sandstones are fine to medium-grained and 90% of the beds are so poorly cemented that they may be crumbled between the fingers. The remaining 10% form hard lenticlular masses or ribs of better cemented sandstone.

The siltstones and clays are quite hard, but a bed containing an appreciable clay fraction shrinks rapidly and develops cracks when exposed to air. These beds slake rapidly with alternate wetting and drying. The clays are strongly stratified, heavily overconsolidated, and are intersected by a number of discontinuities. Bedding plane shears have occurred as a result of folding. These shears, which vary in thickness from a fraction of an inch up to about 3 ft, have been found in approximately tv thirds of the beds and some beds contain mo than one zone. The shear zones appear to continuous throughout the full extent of a le or bed. The strength of the clay within a sh zone has been reduced to the residual conditi

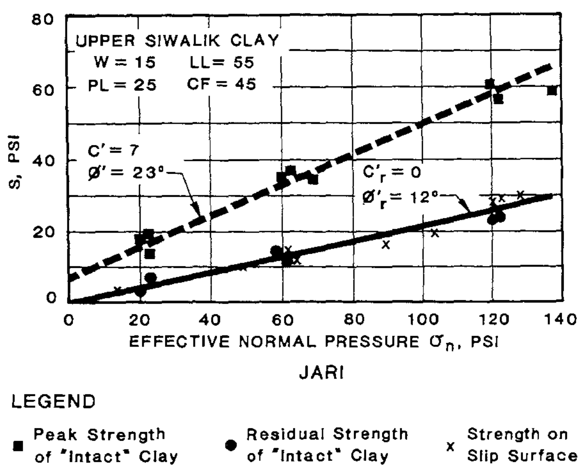
When the Contract design of the project being prepared, effective shear streng parameters for clay and siltstones for des purposes were derived from testing samples fı boreholes. At about this time Profess Skempton was developing his ideas on slc stability and residual strength. An attempt made to measure the residual strength paramet of clay bedrock samples by triaxial compress tests carried beyond the point of maxin resistance then commonly taken as the termin ing point of such tests. The design paramet derived from these tests, which were used the Contract design, are listed in Table 3.

Table 3
MANGLA PROJECT, PAKISTAN
CONTRACT DESIGN BEDROCK PARAMETERS

Position	Clay bedrock		Sandstone bed
	c' lb/sq. in.	ϕ'	c' lb/sq. in.
Mangla Dam	3·5	32°	
Intake embankment	0	30°	
Sukian Dam	0	31°	0
Jari Dam and rimworks	0	28°	
Main spillway	3·5	32°	

SOURCE: Mangla, Binnie and Partners

Subsequent development of testing techniqu showed that the strain to which the triaxi specimens were taken (about 8%) was insufficie to reduce the strength of intact clay to t residual. In 1964, the shear boxes at Mang were adapted to allow the reversal shear b technique. With this form of testing, it w possible to closely approach the true residu condition. Typical shear box test results f the Jari clays are shown in Fig. 9.



SOURCE: Mangla, Binnie and Partners

Fig. 9 MANGLA PROJECT, PAKISTAN—TYPICAL TEST RESULTS OF SLIP SURFACE IN SHEAR ZONE D

The effect of these discontinuities on design may be summarized as follows:

Shear Zones

The presence of shear zones resulted in the assumption for design purposes that all clay beds contain a shear zone; that shear zones are continuous; and that the clay within a shear zone has been reduced to the residual condition. Thus the residual factor R as defined by Skempton was taken to be 100% for all potential failure surfaces parallel to bedding planes in clay.

Random Fissures and Thrust Shear Joints

The presence of joints and fissures in the bedrock was taken into account in assessing the shear strength parameters of clay bedrock in any direction other than parallel to bedding. It was assumed that, of that part of a potential slip surface passing though clay bedrock, half would be in intact clay and half would follow joints or fissures. Thus the residual factor was taken to be 50%. This represents an estimate of the present condition which, it was assumed, would remain unchanged in the long term in foundations which were substantially loaded by the construction as, for example, under the three dams. Where bedrock slopes were not so loaded, as for example at the intake embankment, it was assumed that with time the intact clay would deteriorate to the residual condition. Thus the residual factor for across bedding shear strengths was taken to be 100% for these conditions.

To reduce the number of shear tests required, an attempt was made to establish a correlation between the residual angle of shearing resistance ø' and a readily determined index property of the clay. A correlation was obtained with the clay fraction (percentage by weight of particles less than 2 microns in size). The correlations curves for Mangla and Jari clay bedrock were developed from tests on both intact and naturally sheared clay from block samples. The curves are shown in Fig. 10.

Table 4 lists the final clay bedrock parameters used in design and also shows the values of the residual factor R used in the various locations. Stability analyses were performed by hand using the Morgenstern-Price method on trial wedges at many cross-sections through the several main project features.

A number of modifications to the design were adopted as a result of the substantial difference between shear strength values used to develop the contract documents and those finally selected. The modifications varied from structure to structure but, in general, consisted of one or a combination of the following techniques:

- weighting berms
- additional drainage to reduce uplift
- slope flattening
- over-excavation and backfill

Bath County Pumped-Storage Project

The Bath County Project, in Virginia, when completed, will have a total installed capacity of 2,100 MW making it the world's largest pumped-storage project. The lower reservoir will be formed by constructing an embankment across Back Creek. The upper reservoir will be formed by a 460-foot high embankment dam containing about 17 million cubic yards of fill. The embankment includes a central core protected by filters and supported by compacted earth and rockfill. After a two year delay, full scale construction resumed in May of 1982. The revised schedule will place the first two pumping-generating units on line in the spring of 1985.

The dominant land forms in the project area are subparallel northeast trending ridges and valleys typical of the Appalachian Valley and Ridge physiographic province. The ridges are underlain or capped by comparatively resistant sedimentary rocks, while the valleys have evolved along weaker, less resistant rock units.

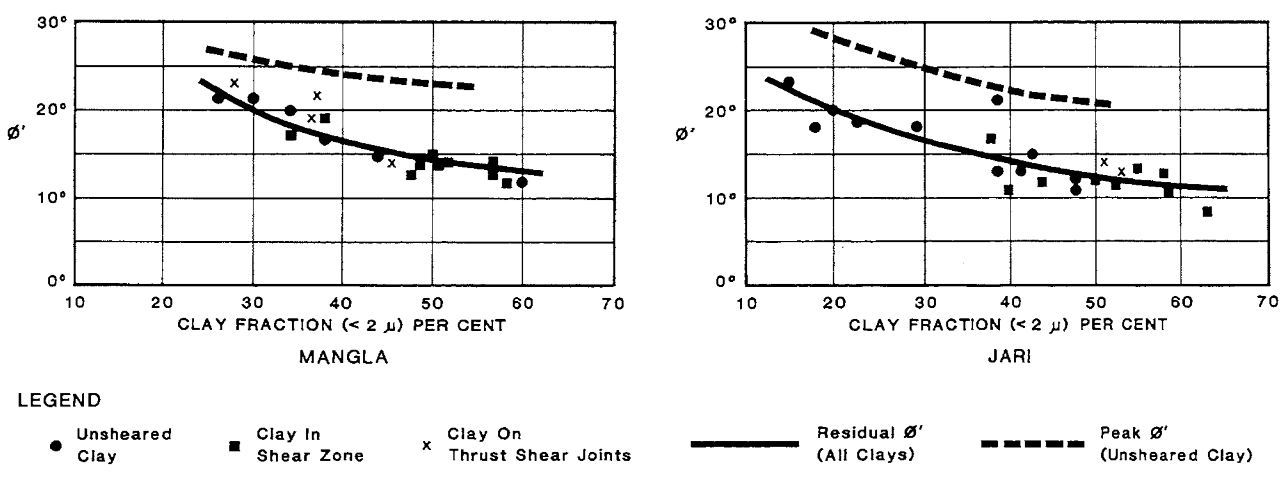


Fig. 10 MANGLA PROJECT, PAKISTAN — CORRELATION BETWEEN RESIDUAL ANGLE OF FRICTION AND CLAY FRICTION FOR CLAY BEDROCK AT MANGLA AND JARI

Table 4
MANGLA PROJECT, PAKISTAN
FINAL DESIGN BEDROCK PARAMETERS

Position	Clay bedrock						Bentonite			Sandstone bedrock		
	Along bedding			Across bedding								
	c' *lb/sq. in.*	*φ'*	*R* %	*c'* *lb/sq. in.*	*φ'*	*R* %	*c'* *lb/sq. in.*	*φ'*	*R* %	*c'* *lb/sq. in*	*φ'*	*R* %
Mangla Dam	0	18°	100	6	26°	50	—	—	—			
Intake embankment	0	18°	100	0	24°	100	—	—	—			
Sukian Dam (western half)	0	18°	100	6	25°	50	0	12°	100			
Sukian Dam (eastern half)	0	16°	100	6	25°	50	0	12°	100	0	38°	100
Jari Dam	0	13°	100	5	20°	50	—	—	—			
Jari rimworks	0	13°	100	0	16°	100	—	—	—			
Main spillway (headworks and upper chute)	0	18°	100	0	24°	100	—	—	—			
Main spillway (stilling basins)	0	18°	100	3	26°	75	—	—	—	3 (above water-table) 0 (below water-table)	38·5° 38·5°	75 75

SOURCE: Mangla, Binnie and Partners

The lower reservoir, lower dam, powerhouse, spillway and the lower part of the power tunnels are founded on or situated in the Brallier Formation. The upper reservoir, upper dam, intakes, surge tanks, and upper portion of the power tunnels involve rocks of the Chemung Formation. The rocks are compaction sediments that have been indurated by overburden loads and tectonic forces. For the most part, shales and claystones in the rock sequence slake and tend to disintegrate upon freeze-thaw and wetting-drying cycles.

Direct shear tests on clays taken from exposures at the base of cliffs indicated a residual shear strength of 17°-23°. Although all project features were designed for low bedding shear strength, a number of problems occurred during construction which were directly related to bedding plane joints and/or shears.

Upper Dam Foundation

Since the earliest project investigations, attention had been drawn to the geologic structure of the upper dam foundation and its effect on stability. Interbedded Chemung sandstone and siltstone/shale layers dip at 10-15 degrees out of the left abutment (Fig. 11 Because of a regional joint set that strik almost parallel to the valley, the rock surfa exposed by the core trench is steplike.

Special treatment details for the core found tion of the upper dam were developed aft exposure of the foundation during constructio Excavation of the core trench resulted in ragged, step-like surface caused by mark bedrock stratification and jointing normal bedding. The varying thickness and competen of the different bedrock units resulted in ste ranging in height from a few inches to over feet.

On the left abutment, foundation shaping w required to remove unacceptable overhang Minor overhangs were tolerated, if less than inches, if the overhang was not more than 10-feet long and if the bed thickness was less th 6 feet. More competent, thicker sandsto ledges, with more pronounced overhangs we shaped by presplitting. However, the presen of locally occurring clay seams along beddi planes, or weak shales at the base of so higher ledges, necessitated stabilization pri to presplitting by rock-bolting to stable ro

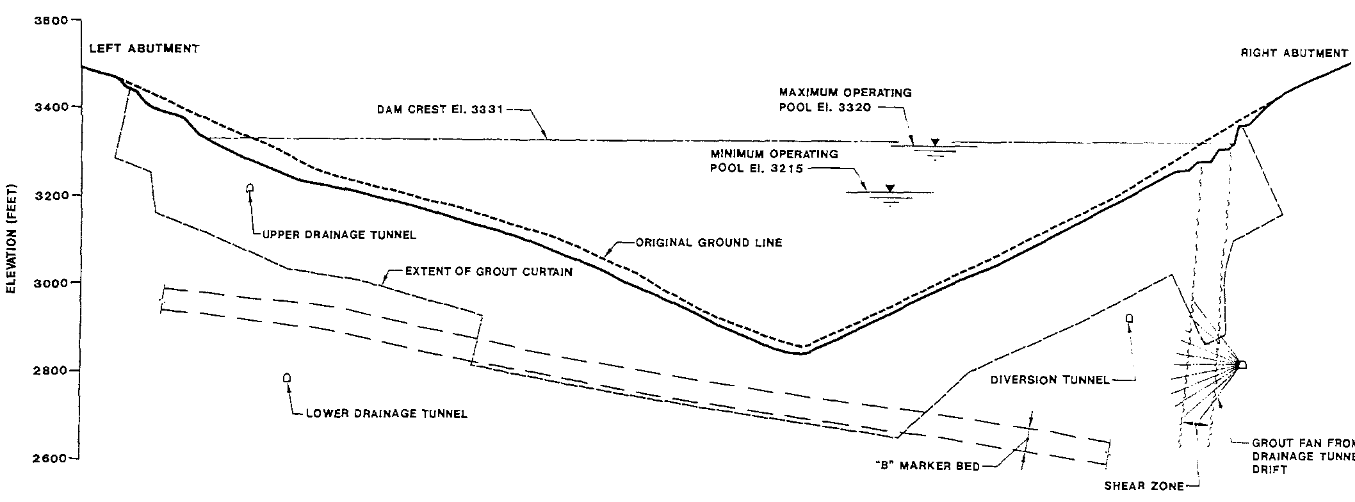


Fig. 11 BATH COUNTY PUMPED STORAGE PROJECT, VIRGINIA UPPER DAM CROSS VALLEY PROFILE

beneath (Fig. 12).

Sharp topographic re-entrants on the left abutment created an adversely sloping foundation surface immediately downstream of the core. This condition exposed foundation bedding joints along which fines could conceivably move. A two-stage filter system was placed over foundation surfaces to prevent potential movement of foundation fines into the downstream shell (Fig. 13).

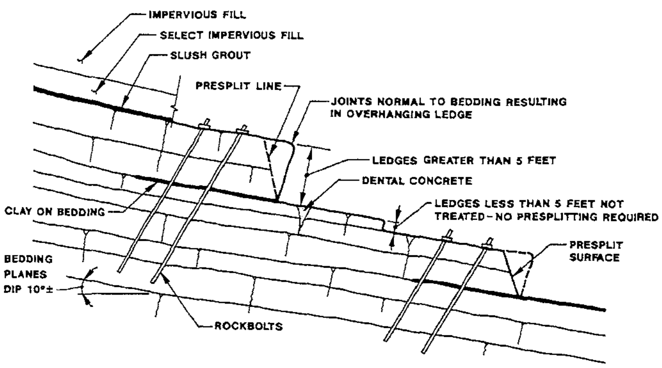


Fig. 12 BATH COUNTY PUMPED STORAGE PROJECT, VIRGINIA— UPPER DAM LEFT ABUTMENT CORE FOUNDATION

<u>Use of Drainage Adits</u>

Abutment drainage by means of man-sized adits and closely spaced drain holes is becoming more popular with many designers of major dams. Drainage by adits and drain holes provide stability to the abutments, controls seepage passing through the foundation and abutments and intercepts seepage before it enters the downstream shell of the dam. Adits also serve other extremely useful purposes in addition to drainage:

1. Adits provide an opportunity to examine the in-situ rock conditions in ways that can only be approximated by drilling techniques, especially in low quality rock where core recovery is poor. Adits have been used on many large projects both during detail design and during the initial phases of construction to further explore the abutments.

2. Adits provide access for drilling and grouting during project construction and during project operation should the need arise for remedial treatment. It is becoming more common to provide both drainage and grouting adits.

3. Adits provide access for instrumentation to monitor abutment behavior. It is relatively simple to install, operate and maintain piezometer systems and other instrumentation from inside the adits.

Grouting adits are commonly lined with concrete in both good or bad rock conditions. Drainage adits may be lined or unlined depending on the competence of the rock. Adits which are to be used as permanent project features should have adequate lighting, ventilation, a stable floor and positive drainage by gravity.

Table 5 summarizes Harza's recent experience in using adits at major dam sites.

Figures 14 and 15 illustrate the proposed adit systems for the Maqarin project in Jordan. A system of both grouting and drainage adits are planned in both abutments. Substantially more work will be required in the left abutment because of the permeable altered limestone found in that abutment. During the first 18 months of project construction, the drainage and grouting adits will be excavated, mapped and studied in detail. Modifications to the grouting, drainage and cutoff details may be needed based on the nature of the abutments as observed in the adits.

<u>Jackson Lake Dam, Wyoming</u>

Many old dams located in seismically active areas have recently been studied to evaluate their resistance to major earthquake. The United States Bureau of Reclamation has performed such an analysis at its Jackson Lake Dam and has found that the loose cohesionless

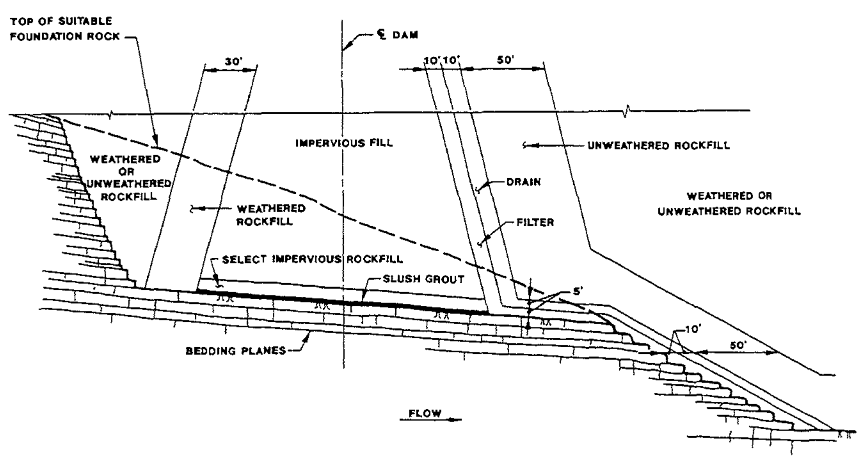


Fig. 13 BATH COUNTY PUMPED STORAGE PROJECT, VIRGINIA—UPPER DAM LEFT ABUTMENT TREATMENT AT TOPOGRAPHIC REENTRANT

Table 5
HARZA DAMS WITH ABUTMENT ADITS

NAME OF DAM	LOCATION	MAXIMUM HEIGHT OF DAM, m	TYPE OF DAM	TOTAL LENGTH OF ADITS, m	PURPOSE OF ADITS			
					EXPLORATION	DRAINAGE	GROUTING	INSTRUMENTATI
MOSSYROCK	USA	185	CONCRETE ARCH	1000	●	●		●
REZA SHAH KABIR	IRAN	200	CONCRETE ARCH	5300	●	●	●	●
LA HONDA	VENEZUELA	120	EARTHFILL	3700	●	●	●	●
GURI	VENEZUELA	100	EARTHFILL & ROCKFILL	400	●	●		
SAN LORENZO	EL SALVADOR	40	EARTH AND ROCK	500	●	●		●
CERRON GRANDE	EL SALVADOR	80	EARTH AND ROCK	850	●	●		●
BAO	DOMINICAN REPUBLIC	110	GRAVEL FILL	1600		●		
BATH COUNTY UPPER DAM & RESERVOIR RIM	USA	145	ROCKFILL	7500	●	●		●
MAQARIN*	JORDAN	140	ROCKFILL	6000	●	●	●	●
WATANA**	USA, ALASKA	270	EARTH, GRAVEL & ROCKFILL	2000	●	●	●	●
TWO FORKS**	USA, COLORADO	200	CONCRETE ARCH	750	●	●		●

*DESIGN COMPLETE, CONSTRUCTION PENDING
**DESIGN IN PROGRESS

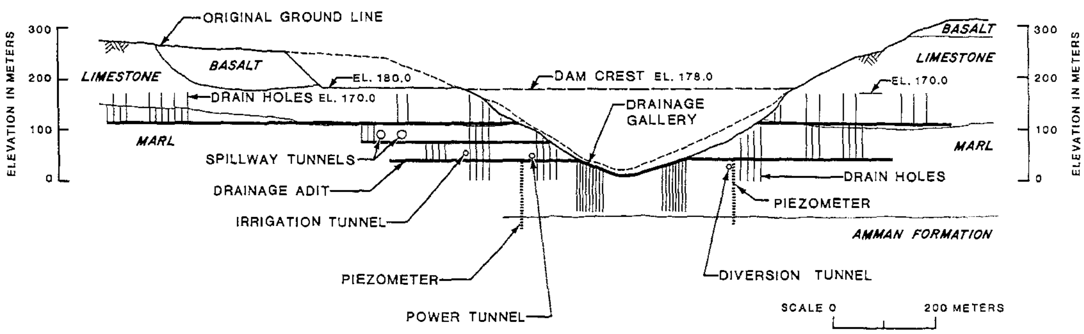


Fig. 14 MAQARIN DAM, JORDAN–PROFILE OF DRAINAGE ADITS

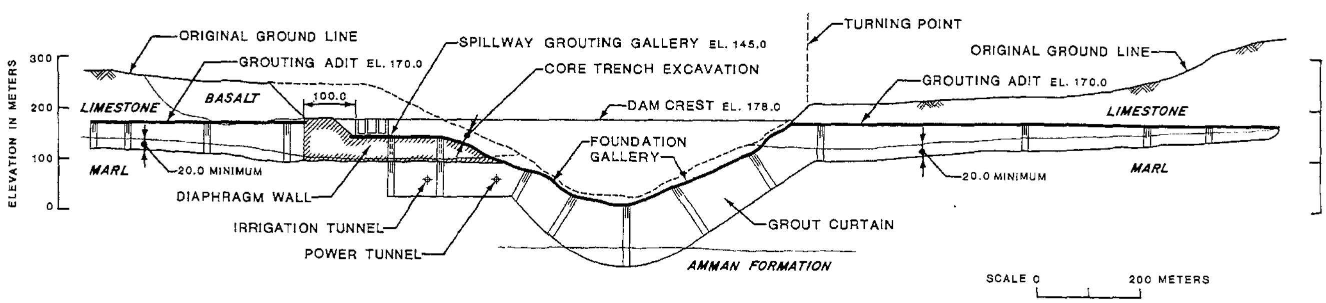


Fig. 15 MAQARIN DAM, JORDAN–PROFILE OF GROUTING GALLERIES

material in the dam and in the foundation is susceptible to liquefaction as a result of seismic shaking. In-situ field tests by the Bureau have indicated that the foundation can be seismically improved by installing compaction piles on six foot centers.

A brief study was recently conducted to evaluate various concepts to seismically strengthen the Jackson Lake Dam. These included several innovative ideas, some of which have little or no precedent. One concept includes foundation stabilization using compaction piles and excavation and reconstruction of the embankment dam, as illustrated in Fig. 16. Compaction piles are used only under the central portion of the dam to minimize construction cost. The concept allows slumping and cracking of the outer slopes of the dam during the design earthquake but maintains freeboard and the integrity of the central portion of the structure. The final design and details of the scheme have not yet been developed.

<u>La Honda Dam, Venezuela</u>

The 140 m high La Honda dam consists of a central core of sandy silty clays flanked by densely compacted sands with about 10% silty fines, Fig. 17. The dam was topped out late in 1983.

During construction, the performance of the dam was monitored by piezometers, slope indicators,

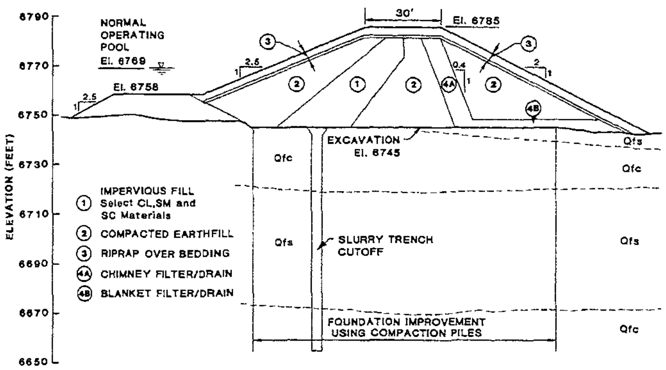


Fig. 16 JACKSON LAKE DAM, WYOMING—TYPICAL SECTION OF EARTHFILL DAM WITH FOUNDATION IMPROVEMENTS

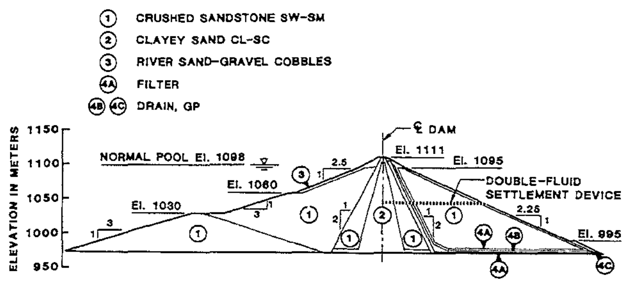


Fig. 17 URIBANTE—DORADAS PROJECT, VENEZUELA MAXIMUM HEIGHT CROSS SECTION OF LA HONDA DAM

and the double-fluid settlement device. This latter instrument has been used successfully at Tarbela and at Guri in Venezuela. It provides a continuous record of displacement on a horizontal plane within the dam under varying embankment loads above the instrument. Readings taken when the dam had nearly reached crest elevation are shown on Fig. 18. Note that the profile indicates a displacement within the sand shell of about 0.5 m and a displacement of 1.5 m within the clayey core. A differential displacement of about 1.0 m has occurred over a distance of about 8 m indicating that shear strains in excess of 10% have occurred. There is, however, no abrupt displacement at the interface between core and shell.

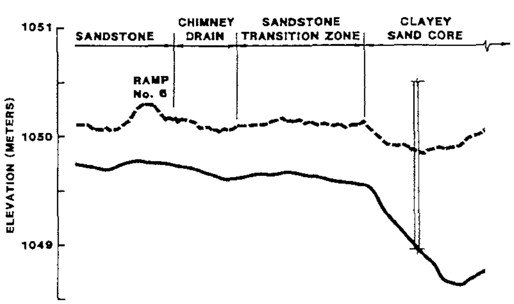


Fig. 18 URIBANTE—DORADAS PROJECT, VENEZUELA SETTLEMENT PROFILE OF LA HONDA DAM

CONCLUDING REMARKS

The various technical papers submitted for this session highlight the necessity of proper design, analysis and construction. Analysis and evaluation of performance extends throughout the entire life of a dam from the initiation of planning studies through construction, first filling, project operation and maintenance. Tailings dams must be watched most carefully because of the means by which they are built and because construction and operation proceed concurrently throughout their useful life.

Several points can be made in summary:

1. Incorporate adequate defensive measures in design. Proper foundation treatment, correct use of materials and the use of filters, transition zones, and drains are keys to successful dam design.

2. Peer review of design, analysis, and construction. Continual review of decisions and judgments during planning, design, construction and reservoir filling is useful and occasionally essential to the success of the project. Peer review forces the designer to periodically collect his thoughts, to question his own design and analysis and to defend his assessment of problems.

3. Design participation during construction, reservoir filling and early project operation. The design engineer must participate during construction to determine if the design intent is being fulfilled and to observe the performance of the embankment and foundations during construction and reservoir filling. Construction personnel cannot fully interpret design intent and, therefore, important design decisions cannot be left solely to the field construction staff.

4. Performance monitoring. Evaluation of performance during and after construction and during reservoir filling is required for successful project operation. The evaluation may be elaborate or not depending on the type of the structure and its foundation conditions. Performance monitoring is especially important to the successful construction and operation of tailings dams.

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