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[a]Discussion period closed for this paper. Any other discussion received during this discussion period will be published in subsequent Journals.

Appendix II.—Notation

The following symbols have been used in this paper:

a = acceleration;
e = principal incremental strain;
F_C = nondimensional force coefficient;
F_s = static force;
F_t = total force;
g = acceleration of gravity;
H = depth;
m_{eq} = equivalent mass;
t = time;
U = stress (see Fig. 1);
u = displacement;
v = displacement;
$\dot{v}$ = velocity;
w = width;
X = body force;
x = horizontal coordinate;
Z = body force;
z = vertical coordinate;
α = $(\pi/4) - (\nu/2)$;
γ = specific gravity of soil, and shear strain;
θ = angle between major principal axis and x-axis;
θ_w = incremental wall angle of rotation;
μ = $(\pi/4) - (\phi/2)$;
ν = angle of dilation;
σ = normal stress;
τ = shear stress; and
ϕ = angle of internal friction.

JOURNAL OF THE GEOTECHNICAL ENGINEERING DIVISION

Aguada Blanca Rockfill Dam with Metal Facing

By Piero Sembenelli,[1] M. ASCE and Marco Fagiolo[2]

Introduction

The Aguada Blanca Dam is located on the Chili River, a stream flowing on the pacific side of the Andes, near the town of Arequipa in southern Peru. The Ministry of Agriculture of Peru is the owner. The Aguada Blanca was built to expand the storage system for the Pampas de la Joya irrigation, which includes two more reservoirs—the Frayle and Pañe.

The dam, shown in Fig. 1, is a rockfill type with a 3/16-in. (5-mm) thick metal facing, a height of 148 ft (45 m), a volume of 130,000 cu yd (100,000 m^3), a crest elevation of 12,044 ft (3,671 m), and it forms a reservoir with a 35,000-acre-ft (43×10^6-m^3) capacity. The catchment area controlled by Aguada Blanca is over 1,065 sq miles (2,900 km^2).

The annual mean runoff is 215,000 acre-ft (255×10^6 m^3), the peak estimated 1,000-yr flood is 17,860 cfs (500 m^3/s) while the maximum recorded flood, in 20 yr of river gaging, is 9,280 cfs (260 m^3/s).

Ancillary works, shown in Fig. 2, have been designed, based on model studies, as an integrated system whose main element is a tunnel 18.4 ft (5.60 m) in diameter. This tunnel, after serving as temporary diversion with a discharge capacity of 6,430 cfs (180 m^3/s), was gated to work as bottom outlet. It receives, from the left side, 29.5 ft (9 m) downstream from the gate, a 6.50-ft (2-m) regulation conduit and 98.5 ft (30 m) further downstream, the 20-ft (6.10-m) shaft of a morning glory spillway.

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A secondary regulation tunnel, also 6.50 ft (2 m) in diameter, parallels the main one and can convey the regulated discharge (which must be guaranteed at all times) in case maintenance of the main system is necessary. All tunnels are concrete lined.

The bottom outlet top capacity is 5,360 cfs (150 m^3/s) through a 6.5-ft × 9.8-ft (2-m × 3-m) roller gate. The regulation conduit top capacity is 1,960 cfs (55 m^3/s) through a 4.9-ft × 6.5-ft (1.5-m × 2-m) sliding valve equipped for automatic flow regulation.

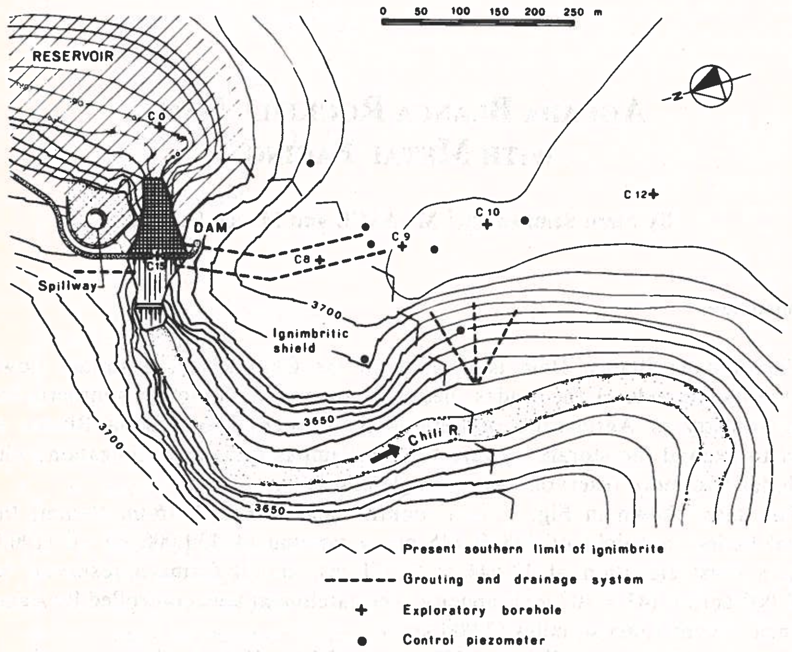

FIG. 1.—Aguada Blanca Dam—Area Plan

A morning glory spillway, with a 65-ft (20-m) weir diam, can pass floods of 17,860 cfs (500 m^3/s) with a head of 7.2 ft (2.5 m). The minimum freeboard is 7.2 ft (2.5 m) plus wave parapet.

The reservoir level will raise to spillway weir between January and March and will empty completely during the following 8 months with anticipated drawdown rates up to 3.3 ft (1 m) per day.

Dam construction began at the beginning of 1968, and the reservoir was first filled at the end of 1970.

Climate

The dam site lies on a high plateau at the border of the pacific coastal desert. The climate is consequently very dry and very cold. The average yearly rainfall is less than 4 in. (100 mm) and atmospheric relative humidity varies between 30% and 60%.

Since it is at a high altitude and also near the equator, the temperature changes are extreme. Temperatures recorded daily over a span of 4 yr, ranged from 7° F to 86° F (−22° C to +31° C). Due to the particular location of the site, the shift from low to high figures is not gradual, producing over a 1/2-yr span,

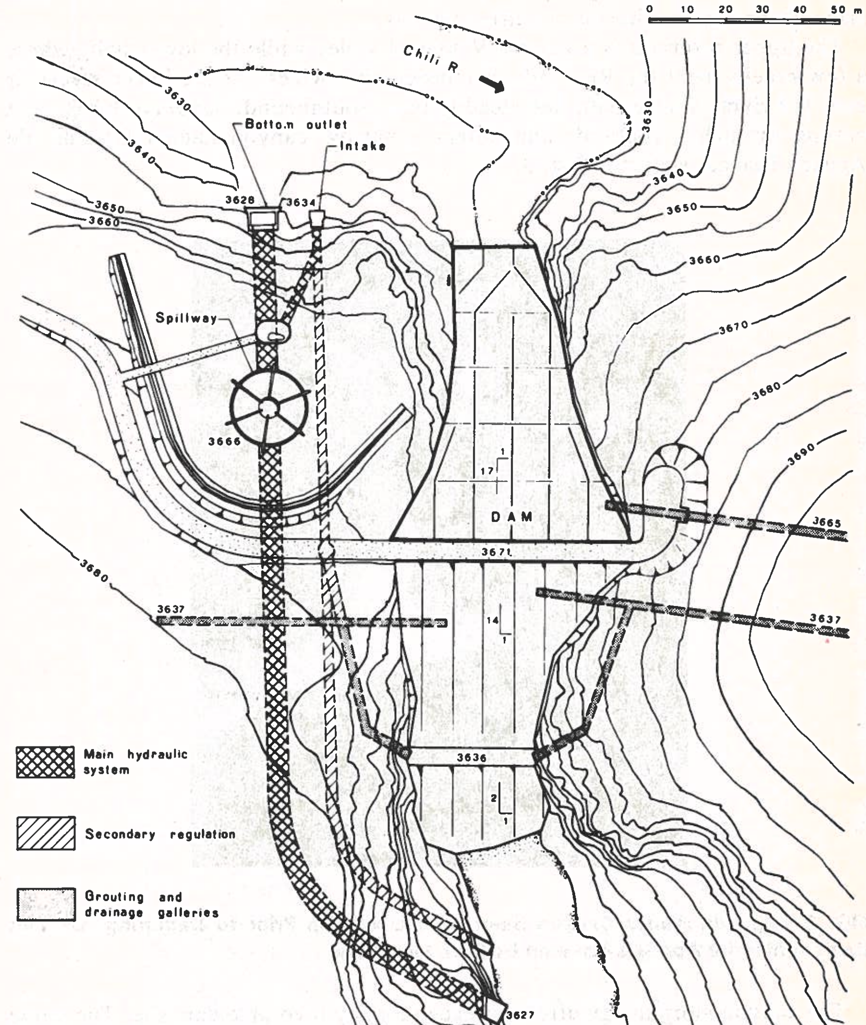

FIG. 2.—Aguada Blanca Dam—Plan of Rockfill and Ancillary Works

but rather a repeated one, sharp and cyclical, taking place in only a few hours.

It has been recorded that day-night temperature drops are often in excess of 75° F (40° C) and as few as 39 days per year without temperatures below freezing point.

Winds are strong and frequent.

Geology

The general geomorphology of the area is typical for the andean "Puna," a high plateau with average elevations around 13,000 ft (4,000 m). Isolated volcanoes and volcanic groups rise above the plateau while rivers of limited size cut through its shallow, ramified valleys.

The upper reservoir is a narrow V shaped valley while the lower half, where a few creeks meet the Rio Chili, is considerably wider. At the lower reservoir end, the river, whose path has steadily been southbound, suddenly turns west cutting through a rock rift and enters a narrow canyon named Boquilla de Aguada Blanca, shown in Fig. 3.

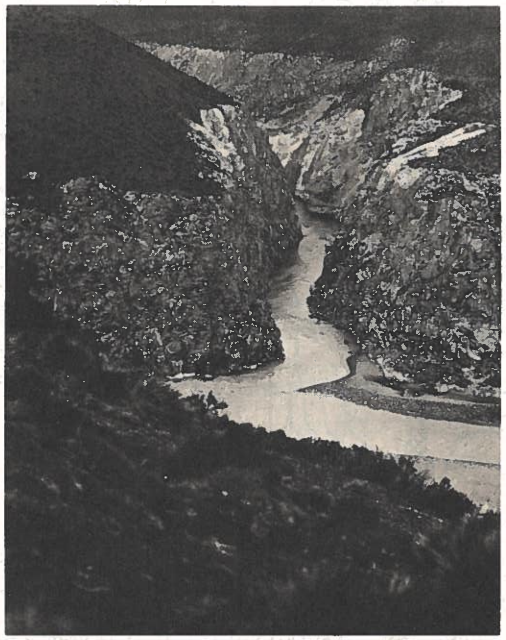

FIG. 3.—Aguada Blanca Canyon Seen from Upstream Prior to Damming; On Left Bank, Ignimbite Appears Covered by Dark Piroclasts

The canyon morphology offers an exceptionally favorable dam site. The gorge is only 650 ft (200 m) long and 65 ft (20 m) wide at riverbed. Its steep rocky walls keep the canyon width below 230 ft (70 m) at the crest elevation. While the right wall is only 160 ft (50 m) high and then almost flattens off, the left wall is somewhat steeper and rises over 245 ft (75 m), well above crest elevation.

The geology of the area, shown in Fig. 4, is entirely related to local volcanism and is the result of alternating and antagonic volcanic eruptions, lacustrine sedimentations, and river erosions.

The primary tuff can be taken as the starting term, originated from nearby volcanoes, it was eroded and smoothened in a peneplain. Subsequently, a series

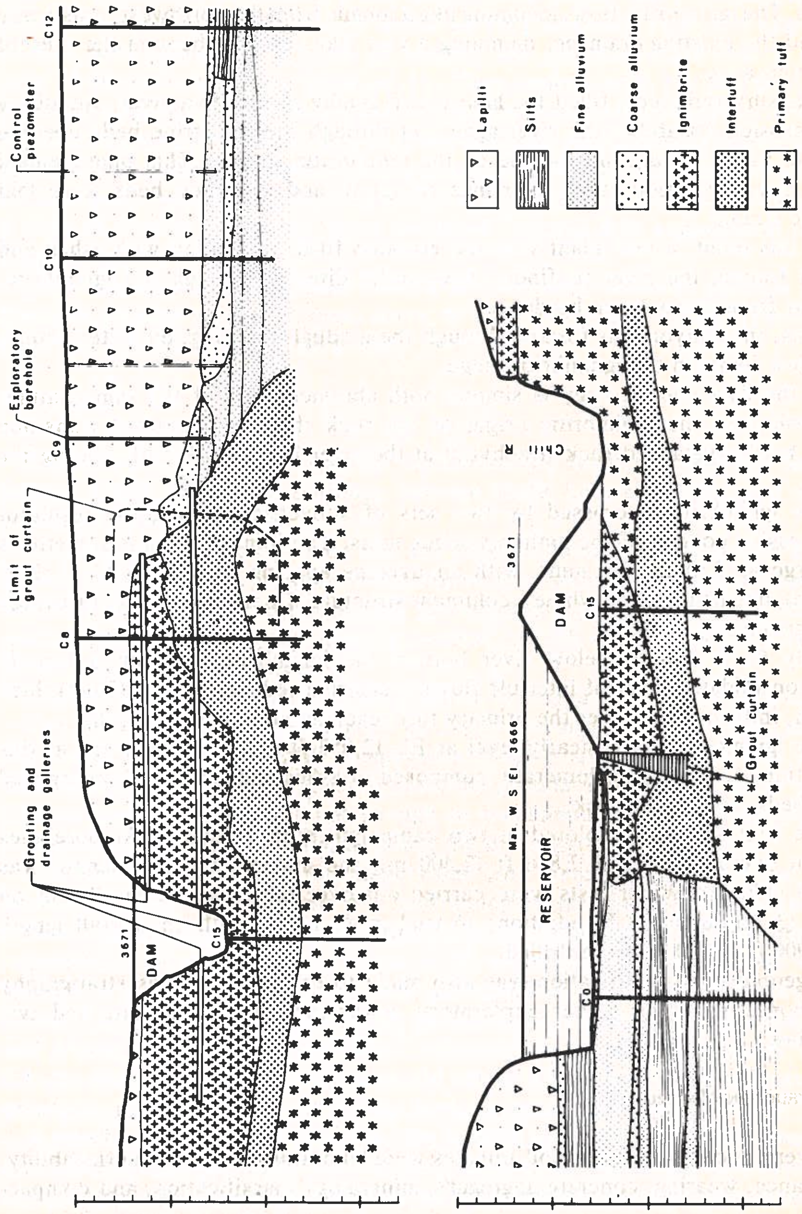

FIG. 4.—Aguada Blanca Dam—Geological Sections

of fluviola custrine depositions, markedly zoned and rapidly tapering off, covered this floor. They remain now as an intertuff formation.

The paramount geological event is the sudden super-position of an ignimbritic shield. The ignimbritic flow, incoming like a tongue from the northwest, drastically altered the existing drainage, damming a vast lake reaching beyond the present reservoir area.

Fine silty sediments filled the lake which finally shrunk to a river. Acquiring new erosion capability the river again cut through the lacustrine beds opening a wide valley all around the toe of the ignimbritic tongue. This phase ended in a new deposition stage interfingered sandy and gravelley beds were laid by the stream.

At this point, a new giant volcanic eruption filled the valley with ashes and lapilli forcing the river to find a new outlet directly through the ignimbrite, not far from its southern border.

Thus, the canyon was formed through the gradual widening, by water action, of a few joints in the ignimbritic ridge.

At the dam site, geology is simple; both abutments are of the same nature. According to the ignimbritic origin of the rock there is a gradual transition from hard crystalized rock (trachytic) at the river bed to soft tuff nearing the crest.

The ignimbrite is crossed by two sets of subvertical joints, the result of progressive cooling of the molten viscous mass. The joint opening is sometimes as large as 3/4 in. (20 mm), with an average spacing of 6 ft to 15 ft (2 m to 5 m), imparting to the whole a columnar structure. Locally conchoidal surfaces become clearly visible.

Only 60 ft (20 m) below river bottom the ignimbrite rests on a heavily overconsolidated layer of intertuff fluviola-custrine sediments 60 ft (20 m) thick which, in turn, bears over the primary tuff reaching an unknown depth.

The ignimbrite top is nearly level at El. 12,057 ft (3,675 m). Starting at this elevation, a volcanic agglomerate, composed of ashes and lapilli with interposed silty beds, covers the rock.

The dam site was explored in two campaigns with a total of 31 boreholes for an overall length of 7,870 ft (2,400 m), the maximum boring length was 425 ft (130 m). Water tests were carried out over the entire boring length on 10 ft (3 m) sections. In addition, 15 trial grout holes, with an overall length of 5,900 ft (1,800 m) were drilled.

A geophysical investigation was also made to extend boreholes stratigraphy into bordering areas. Other exploratory works included three adits and two trenches.

Construction Materials

Several rock, gravel, and soil samples were laboratory tested for permeability, resistance, wearing, concrete aggregate, mineralogy classification, and compaction.

Local construction materials are limited, from a practical standpoint, to andesite, from a nearby lava outcrop and to river alluvium. [Andesite, proposed as a rock source, exists 2.5 miles (4 km) away from the dam site; it has a bulk unit weight of 164 pcf (2.63 t/m^3), a porosity of 1.5%, an abrasion resistance (Los Angeles test) of 19%, and a sound vitrified appearance. Ignimbrite from the dam site area, the only alternative rock, is a soft rock, compressible when broken up by quarrying. The bulk unit weight can be as low as 120 pcf (1.9 t/m^3).] Ignimbrite is, in fact, a low quality rock which can be obtained near riverbeds where opening a quarry would be very difficult. Fines are dispersed in thin beds throughout a mass of coarser low plasticity silts with a liquid limit of 40 and a plasticity index of 13. They contain only 2% to 5% of clay fraction (below 0.002 mm), show poor compaction characteristics with an optimum dry unit weight of 83 pcf (1.33 Mg/m^3), and marked compressibility (compression index 0.2 for material compacted at optimum conditions, in the straight portion of oedometer curve). In addition, silts are dispersed in thin beds throughout a mass of coarser sediments, often cemented, making their exploitation totally unattractive.

River alluvium is well graded from 8 in. (200 mm) down to medium sand. Fraction passing a 3-in. (75-mm) sieve is 80%, passing a 1/2-in. (12-mm) is 45%, no. 10 is 25% and no. 40 is 5%.

Dam Design

The chosen scheme and type of dam were the result of weighing several factors whose relative importance became quite known by the end of all preliminary investigations.

Geology was practically ruled out any rigid solution, like a concrete arc or gravity, because of the possibility of deformations within interuff and lacustrine materials underlaying and buttressing the ignimbrite bloc. In addition, the low mechanical properties of the upper ignimbrite, in both abutments, was a serious factor against any such structure.

Soon it became clear that a solution had to be found within a fill type dam.

The lack of suitable fines quickly excluded the adoption of a cross section with large volume of earthfill components. Such a solution would also suffer from additional problems originating from the peculiar morphology of the gorge. Canyon length, in fact, was not enough to accommodate, without making use of elaborate solutions, both the dam and the diversion-discharge works.

Any zoned solution with thin impervious core were prevented for the same reasons of fines suitability and also morphological difficulties. Construction of a zoned embankment in such a narrow space, complicated by the practical impossibility of cutting access roads to fill elevation, were considered strong enough to discard this solution. Any additional difficulty was considered to obtain a proper seal between a highly broken canyon wall, at sites overhanging, and a core whose settlements were estimated to be on the order of 10 in. (25 cm) during construction and 4 in. (10 cm) in the earliest years of operation, assuming no appreciable contribution from the foundation.

A possible solution considered was the rockfill type, with upstream impervious membranes. This particular dam, in fact, offered the advantage of a minimum base length, permitted the adoption of the simplest and shortest diversion-discharge system, and had the advantage of canyon geomorphological features. It also reduced problems related to embankment construction because the bulk of the dam could be made out of one material only which could be dumped from crest level down to fill grade. The watertight element could be built under

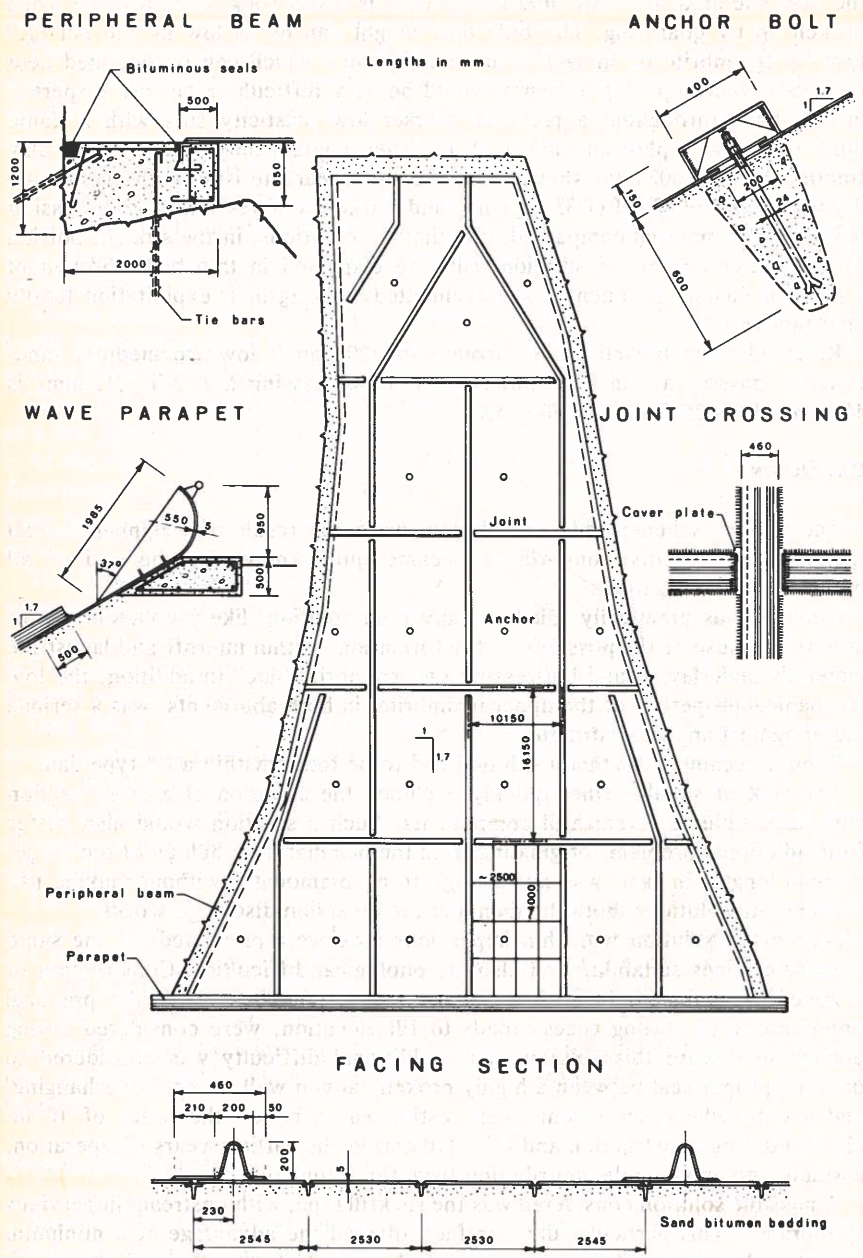

FIG. 5.—Aguada Blanca Dam—Plan of Metal Facing Showing Joint Pattern and Design Details

a relatively independent schedule easing both planning and working at abutment contact.

It was felt that two delicate problems had to be adequately studied and solved: the tight fit between membrane and abutment rock all along its periphery, and

TABLE 1.—Chemical Composition Specified for Aguada Blanca Facing Steel

Chemical (1)	Values (2)
Fe	99.75% min
Cu	0.15 max
P	0.01 max
S	0.03 max
P + S + C + Mn + Si	0.10 max

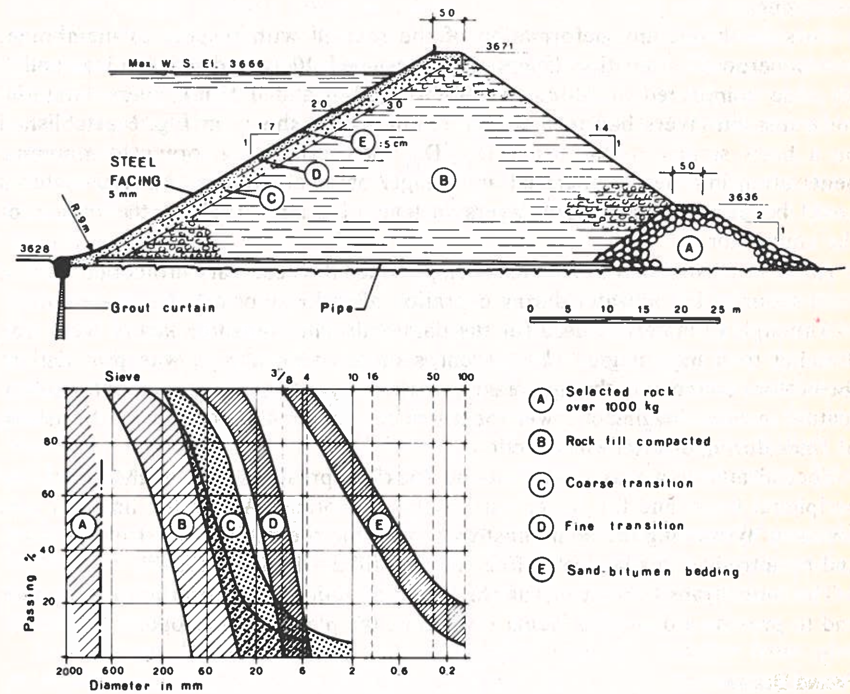

FIG. 6.—Aguada Blanca Dam—Rockfill Cross Section and Specified Gradations for Construction Materials

the proper grouting of the highly jointed rock mass near and below the dam.

A peripheral concrete beam of special design, (see Fig. 5) tied to the rock by rebars and sealed against it by short suture grouting was the answer to the first problem. The beam would also serve as a plug when grouting the main curtain through the rock.

On the other hand, it was soon evident that stability was not a problem. The section adopted offered the best conditions as far as hydrostatic pressure distribution and drainage of both the dam and its abutments. In addition, rockfill was going to develop arching between canyon walls, increasing its own stability. Stability analysis computations made for the downstream slope with the wedge method, using horizontal lower sliding surfaces, neglecting arching effect and assuming a rock friction angle, $\phi = 35°$, gave a minimum factor of safety of 2.1 with a seismic acceleration of 0.15 g (15 m/s^2). In order to reduce settlements the requirement was set for a rockfill compacted in 3-ft (1-m) lifts with four passes of 5-t (5-Mg) vibratory roller.

The final section, shown in Fig. 6, resulted in a uniform rock body with slopes 1.7/1 upstream and 1.4/1 downstream. The rock had to meet specified grain size requirements.

Deformation uniformity was considered a priority requirement over deformation magnitude. Dry masonry layers were eliminated with the aim of avoiding localized deformations and maintaining construction entirely within loose fills current techniques.

To smooth out any deformation of the rockfill with respect to membrane, two superposed transition layers were designed 10 ft and 6.5 ft (3 m and 2 m) wide compacted in 20-in. and 40-in. (0.50-m and 0.25-m) layers. Material for transition layers had to meet the requirements shown in Fig. 6 established on a basis similar to the usual D_{15}/D_{85} filter criteria, in order to minimize penetration into nearby material and danger of local cavities. Alluvial material could be used for transition layers instead of quarry rock at the option of the contractor.

At the downstream toe, selected rock provided the necessary protection against local scouring by tailwater during operation of spillway or bottom sluice outlet.

Although all materials used for the dam body and transition layers were free draining to a high degree (k = 1 cm/s or greater), a pipe was provided at the bottom connecting the upstream transition layer to the toe as an extra safety feature in case clogging of lower rockfill occurs as a result of downward washing of fines during or after construction.

Special attention was paid to avoid rockfill spreading that might affect the peripheral beam and the lower part of the membrane. At the toe line this was obtained by moving the beam upstream from the theoretical offset of the slope and by introducing a wedge of fine selected fill as shown in Fig. 6.

The outer transition layer was thickened as required to form a crest camber and to provide a domelike facing rise above the plane of the slope.

Facing Design

Once the decision was made to use an impervious deck on the upstream face of the dam, the actual type and details of it had to be selected.

Rigid facings with only localized lines of flexibility were not considered adequate to meet movements of a flexible body whose deformations were to a large extent difficult to anticipate because of material characteristics and placement conditions. Nearing the slope, while material characteristics are better controlled through gradation, placement decays because of actual condition unfavorable to proper compaction.

The deck had to withstand temperature variations in the highest ranges which occurred with unusual frequency and were met by a similar structure. Moreover, practically any variation was such as to represent a freezing and thawing cycle. Local thermal gradients, at sun-shade or water-air boundaries, were extremely high. Gradients across the waterline could reverse within a few hours from the daytime condition of facing warmer than water, to the nighttime condition of facing colder than reservoir water. Lines stressed by such high temperature gradients were going to move fast over the deck surface because of the canyon morphology and possible reservoir rate of filling and drawdown.

High heat conductivity was a desirable feature in helping to equalize temperatures through the whole facing. All this also required a material with a maximum inner strength, unaffected by frost, temperature variations, and gradients, and offering maximum stability of characteristics and performance over a long time span.

The possibility was also anticipated that a contractor might not master the required degree of the rather complex technologies of hot mix bituminous concrete or of thin, tight jointed, concrete slabs. This nearly ruled out anything that could not be made with a clear cut process and, in a way, practically independent from local materials, site installations, and qualified manpower. Thorough checks had to be easy and repairs possible in a fool-proof form. In other words, the best facing was the one that could be, so to say, prefabricated and assembled with a minimum of qualified work at the site.

In considering concrete, bituminous mixes, plastics, and metals, the latter met the aforementioned requirements best. The decision was made to use a metal facing.

Once the metal was chosen, steel seemed the best option both in terms of technique and economy. The major difficulty to cope with is, in fact, corrosion. To fight it, one either uses expensive nonrusting metals or alloys, or a steel with a minimum percentage of foreign components. Due to the fact that steel corrosion is mainly promoted at points of unbalanced electric potential originating from microvoltaic elements formed by crystals of different nature, the less impurities present, the lower the rate and degree of corrosion susceptibility.

The chemical composition specified for the facing steel is given in Table 1.

Several such steels are standard production and are commercially available in most countries. At Aguada Blanca ARMCO ingot iron, which has mechanical properties similar to other standard steels, was used and these properties are listed in Table 2.

To eliminate facing punching and to equalize pressure transfer from plate to rockfill, a fine grained, yielding bedding was considered necessary. A sand bitumen mix was specified. The bitumen was added to help both the placement and smoothening of the surface and to protect the airside face of metal. The bedding sand grain size is shown in Fig. 6.

The applications of bare steel with a low percentage of foreign components against fresh water are numerous and positive. Nevertheless, a paint coating was chosen for extra protection and to detect possible spots of degradation. One main role assigned to water side paint was to form a semireflecting surface reducing solar radiation retained by the facing. Both the water and the air side of the facing had to be painted with acrylic paint: two coats on the water side and one coat on the air side over sandblasted surfaces with a 0.003-in. (0.07-mm) minimum coat thickness. Bitumen from the sand-bitumen bedding

TABLE 2.—Mechanical Characteristics of Aguada Blanca Facing Steel

Characteristic (1)	Data (2)
Failure stress, in pounds per square inch (kilonewtons per square meter)	43,000 (295,000)
Yield stress, in pounds per square inch (kilonewton per square meter	27,000 (185,000)
Elongation, as a percentage	38
Thermal expansion coefficient	0.0000067° F^{-1} (0.000012° C^{-1})
Thermal conductivity, in joules per meter second · Kelvin	0.175

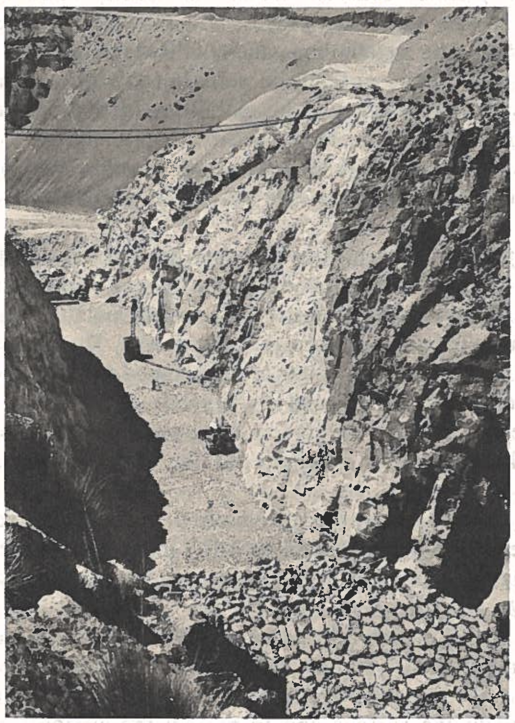

FIG. 7.—Aguada Blanca Dam—Rockfill During Construction; Perypheral Beam is Visible in Upper Part of Picture

was also intended to provide a partial recoating of burned off paint in welding areas.

A 3/16-in. (5-mm) metal thickness was chosen mainly on the basis of the actual possibilities of satisfactorily welding commerical gages and the lower

range of thickness adopted in the past.

No computation was made, since it was assumed the steel membrane stretched over a hole underneath, as sometime suggested, because this was considered a rather academic hypothesis and not likely to occur given the way in which the upstream support layers were designed and had to be built.

Welding, especially delicate because it was made on thin plates, was to ensure a mechanical strength comparable to the plate proper and had to be absolutely tight. Welding procedures were specified according to ASME codes and electrodes according to ASTM A 233 specifications.

Butt welding was permitted only on the assembly yard. Welding on the slope was prescribed as lap only.

Steel profiles set on the slope were provided along all in situ seams. After serving as rails to slide in place each plate they also permitted the lap type welding to be done. Welding seams had to be tested according to ASTM E-94 and E-114 specifications following the engineer's instructions.

The maximum temperature variation assumed for design was 175° F (80° C). A 33-ft (10-m) slab would shrink from its maximum size at highest temperature, approx 3/8 in. (10-mm). Based on commerical plate modular widths, joints were placed at 33 ft × 53 ft (10 m x 16 m). Each joint was designed for a play of 9/8 in. (30 mm) to take into account the partial rigidity of cross points. Plate behavior between partially restrained points, such as profiles at in situ lap welds, was studied in order to ensure that whenever thermal expansion force was large enough to produce buckling it also overcame the resistance of the plate to slide on the slope, thus buckling stresses would be relieved by joints play.

The facing shown in Fig. 5 is composed of individual, preassembled plates of 8.2 ft × 13.1 ft (2.5 m × 4.0 m), welded in place to form a continuous sheet. Expansion joints are U shaped with a bottom plate to prevent the material underneath from creeping in and locking the joint. The U joints, if crossed straight, would leave a grid of singular points with no deformability. To overcome this, each crossing was made with one of the two meeting accordeons comtinuous throughout while the other one was interrupted and side covered with a plate a short distance away. This same disposition was repeated two crossings away while at the next crossing the arrangement was rotated 90°. By so doing the rigidity at each cross point is only partial and the movement component prevented can be absorbed at the next joint fully effective in that particular direction.

The grid of U-shaped joints ends, all around, in a peripheral joint running so as to make the facing shape symmetrical around the slope center line.

Some distance outside the peripheral joint the facing ends welded to a tie-down profile anchored to a concrete beam sealed into canyon rock. A bituminous plastic seal fills a groove alongside the steel concrete matching line. Steel and concrete should dilate here substantially at the same rate as and similar to the canyon wall rock.

The facing at the dam crest is rolled upstream to form a wave parapet in a way often seen in other dams. This method was adopted after a consideration of facing smoothness and consequent increased wave run up.

To limit the possibility, however remote, of excessive facing shifting in some preferential direction, anchors have been provided near the center of each 33-ft × 53-ft (10-m × 16-m) panel. Each anchor ties down the facing while allowing freedom for 8 in. (20 cm) of lateral movement. Each anchor bolt is waterproofed by a welded cover box.

Dam Construction

The bulk of the dam, shown in Fig. 7, was obtained from an andesite-dacite outcrop quarried approx 2.2 miles (3.5 km) from the site. The contractor chose to obtain the rock by coyote holes and shafts loaded with an average of 0.85 lb/cu yd (300 g/m^3) of Amnonal with 8% Semexa (60% dynamite). This method, given actual rock jointing, led to a substantial amount of oversize blocks which had to be drilled and split.

Loading at the quarry was with Cat 944 and hauling to the fill by 9 cu yd (7 m^3) dumpers. Rock was spread in 2.5-ft (80-cm) lifts by an HD 21 bulldozer. The practical impossibility of cutting access ramps into the canyon steep abutments forced the contractor to build the last 100 ft (30 m) of the dam by chuting the rock from the crest down to fill level.

This procedure, in connection with bulldozer spreading, instead of loader transport from surge pile to placement site, led to some segregation and accumulation of fines in limited areas. As a remedial measure, spots of accumulated fines were surrounded and interlayered with well-graded, clean rock in order to guarantee an efficient drainage throughout the entire rockfill body.

Transition layers were placed after completion of the main rockfill (see Fig. 6) lowering class C and D rock along the slope by chutes to two separate hoppers and moving it by 3/4 cu yd (0.5 m^3) tippers. Both layers were raised at the same time and compacted in 10 in. (0.25 m) lifts with four passes of a 1-t (1-Mg) vibratory roller. River gravel was used only to compensate for gradation lacks of crushed quarry rock.

Dam construction was totally unaffected by climatic conditions. No substantial departure from specifications can be mentioned.

Facing Placement

Facing placement, shown in Fig. 8, started with locating and welding the 8.2-ft × 13.1-ft (2.5-m × 4.0-m) grid of A 36 steel T profiles, sandblasted and protected with two covers of red lead except for the upper flange face.

The spaces between profiles were then filled flush with a sand-bitumen hot mix. Preheated aggregates were mixed with 6.5% by weight of RC 2 cut-back, for 5 min in a 3/4 cu yd (0.5-m^3) concrete mixer equipped with a burner to maintain a mix temperature of 140° F (60° C). The mix, transported by crane hoppers, was hand placed and smoothed by light steel drums operated manually.

Facing units were preassembled, welding, together two 4.1-ft × 13.1-ft (1.25-m × 4.0-m) plates to form an 8.2-ft × 13.1-ft (2.5-ft × 4.0-m) piece; the edge which was butt welded was previously champfered. Welding was carried out in three stages. One root pass and one filling pass were applied successively in the champfer groove. A third filling pass was applied after turning the plate upside down and cleaning the back of the gap with a thin grinding wheel. This third step was not foreseen in the original specifications but was decided on after observing that weld penetration was not complete at all points. Slight irregularities in plate edge straightness, due to improper factory cutting, were

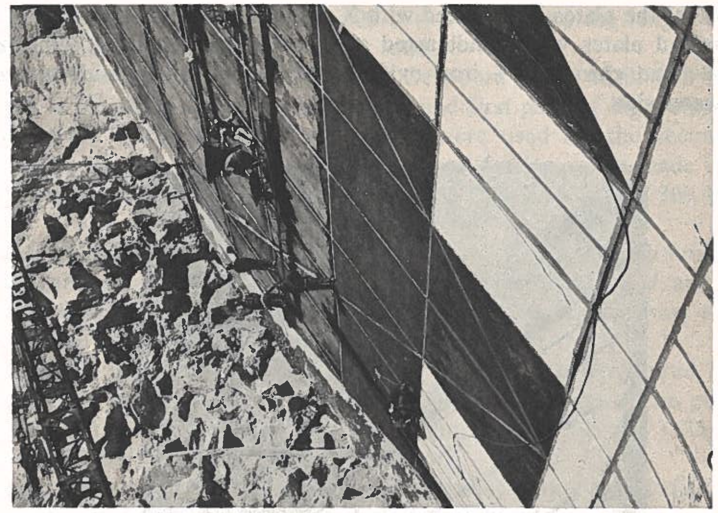

FIG. 9.—Aguada Blanca Dam—Close-Up View of Facing Plates Being Welded to Pre-set Profiles

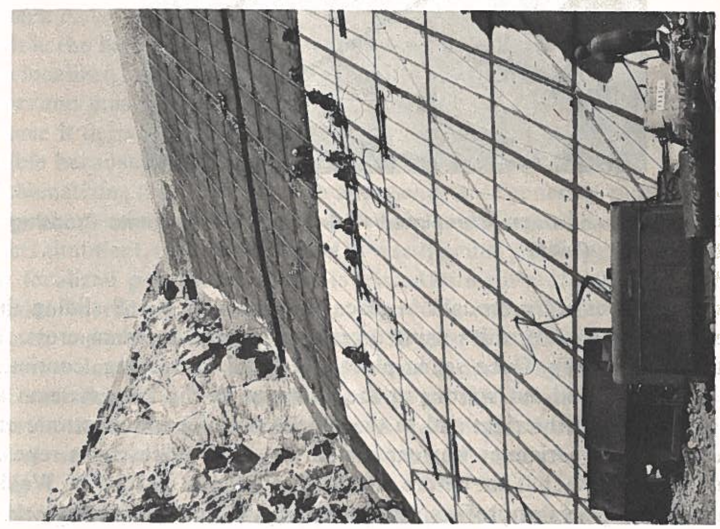

FIG. 8.—Aguada Blanca Dam—Facing During Placement; Sand-Bitumen Bedding Layer is Visible Partially Completed

also a reason for the adoption of a third pass.

To avoid thermal warping, the plates were first tack welded. Final welding was started from midpoint of each seam outward, burning one electrode each way.

Over 18% of the plates were tested with X-rays for welding quality.

Preassembled plates were sandblasted and coated with antirust paint (zinc chromate + lead chromate + iron oxide), two covers on dry side and one cover on water side.

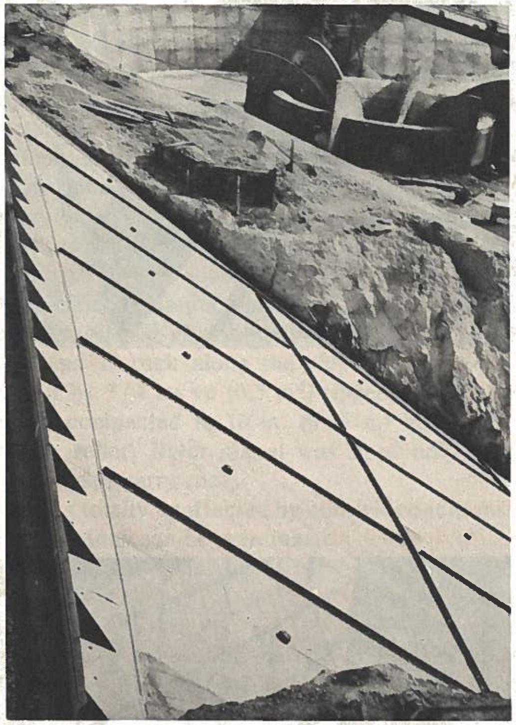

FIG. 10.—Aguada Blanca Dam—Completed Facing and Crest; Joints Crossing Arrangements is Clearly Visible

Preassembled plates were brought in place as shown in Fig. 9 sliding them over the corresponding T profiles using a crane placed on the dam crest. The work started from toe up. Once set in place and fixed by tacking, continuous final welding was carried out starting at the midpoint of the longest seam and advanced alternating to the right and to the left by burning one electrode each way. Once half of the perimeter was completed the same process was repeated on the second half length, but not before the heat had entirely dissipated. Welding was not allowed on plates or profiles that were still hot from previous work.

Lap welding on the slope was carried out in one pass. The lower six plates, about a quarter of the entire facing, were welded with two passes. Welding in tow passes was also used for all horizontal seams at the lower end of each

plate to obtain a better filling of overhead groove.

The U joints, cold bent, were welded to the facing as the final step. As an extra safety feature, U shaped backstrips were placed under all butt welds between the joint elements.

The overall weld length was 16,500 ft (5,000 m), and the length of rejected and replaced welding was 1.5% of the total.

The electrodes used were Oerlikon "Cellocord P" E 6010, 1/8 in. (3 mm) for the first and third pass of butt welds and first pass of lap welds. Oerlikon "Overcord F" E 6012, 3/16 in. (4 mm) were used for the second pass of butt welds, the second pass of lap welds, and for lap welds made in on step. The welding machines used were Lincoln SA 200 F-162, SAE 300 F 226, and F 162.

After completion of all weldings on facing and after a through brush cleaning of all damaged or burned paint, the water face was touched and a second cover of antirust paint was then applied. The final paint (acrilic resins + pigments) was applied in two coats.

The actual placement of 34,000 sq ft (3,150 m^2) of facing required 75 working days with a crew of six welders plus 12 unskilled laborers. The plate weight was 138 ton (125 Mg) and the overall facing weight 195 ton (175 Mg), i.e., 11.4 psf (56 kg/m^2) of facing, was placed.

Environment was the main difficulty. The welding routine was, in fact, disturbed frequently by repeated facing deformation produced by temperature changes. Fig. 10 shows details of the completed facing and dam crest.

Abutment Treatment

Water and grouting tests carried out during the exploratory stage led to the assignment of average permeability characteristics to local formations as shown in Table 3.

While the high values of permeability in fluvio lacustrine formations originate from localized lenses and streaks of coarue alluvium interbedded with a relatively impervious mass, the permeability of ignimbrite is generalized through the mass because it derives from intense jointing. At several spots water tests were not feasible because of excessive fracturing and total water loss.

Schematizing the situation, both abutments were generally pervious and locally extremely pervious rock, because of several crossing, wide open joints. Beyond the left abutment, rock was replaced by a sedimentary mass generally semipervious with localized pervious gravelly levels. Abutment waterproofing scheme was intended to reduce permeability in the rock immediately near the dam, and to catch and control localized seepage beyond left abutment rock. For the first purpose a double grout and drainage curtain, departing sideways along the dam axis, was adopted in both abutments while for the second purpose only drainages at the downstream end of seepage paths were decided. An extended piezometric control network was foreseen as an integral part of the scheme, as shown in Fig. 1.

The right abutment was grouted from ground surface to a distance of 330 ft (100 m) from the dam. Drainage holes were drilled both upward and downward from a drainage tunnel 250 ft (75 m) long at El. 11,932 ft (3,637 m).

The left abutment was grouted from a grouting tunnel extending 350 ft (105

m) into canyon wall. As in the opposite bank, a drainage tunnel, 82 ft (25 m) downstream from grout curtain, enters 730 ft (223 m) into the abutment's rock at El. 11,932 ft (3,637 m). The drainage tunnel length was extended during

TABLE 3.—Average Permeability Characteristics of Aguada Blanca Formations

Formation (1)	Average lugeon units, L (2)	Estimated permeability coefficient, K, in centimeters per second (3)
Ashes + Lapilli	38	10^{-4}
Fluvico Lacustrine	41	10^{-4}
Ignimbrite	over 37	10^{-4} or more
Intertuffaceous	16	10^{-5}
Primary tuff	19	10^{-5}

Note: L and K values are for ungrouted materials.

TABLE 4.—Data on Grout and Drainage Curtains for Aguada Blanca Dam

Item (1)	Grout curtain (2)	Drainage curtain (3)
Net area, in square feet (square meters)	213,000 (19,800)	183,000 (17,000)
Overall drill-hole length, in feet (meters)	32,500 (9,950)	6,100 (1,850)
1st stage drill-hole length, in feet (meters)	14,700 (4,480)	
2nd stage drill-hole length, in feet (meters)	16,800 (5,125)	
3rd stage drill-hole length, in feet (meters)	1,150 (345)	
1st stage cement + sand + bentonite average take, in pounds per foot (kilograms per meter)	365 (547)	
2nd stage $c + s + b$ average take, in pounds per foot (kilograms per meter)	165 (246)	
3rd stage $c + s + b$ average take, in pounds per foot (kilograms per meter)	37 (56)	

consturction in order to ensure that the alluvial mass, beyond the abutment's ignimbrite, was attained.

The dam itself was protected with a grout curtain drilled from the peripheral concrete beam well into intertuffaceous formation.

Drainage of alluvial levels beyond abutment rock was obtained with five

subhorizontal holes 260 ft (80 m) long, daylighting on the left river bank at El. 11,914 ft (3,631 m).

The piezometric network consisted of 15 open piezometers with an overall length of 3,400 ft (1,040 m) and a maximum individual depth of 270 ft (82 m). To guarantee unrestricted access, all piezometers were drilled from the ground surface.

Drilling was only rotary with AX (30 mm) holes for grout curtain and NX (75 mm) holes for drains.

All piezometers and drains, drilled downward or in loose materials, were lined with plastic pipe 2 in. (48 mm) OD with 1/32 in. (1 mm) slots. Other drains were unlined.

TABLE 5.—Final Construction Costs for Aguada Blanca Rockfill Dam

Item (1)	Cost, in dollars (2)
Rockfill in place including transition layers	3.75/cu yd (4.9/m^3)
Facing in place including joints, profiles, paints, and bedding	4.0/sq ft (43.0/m^2)

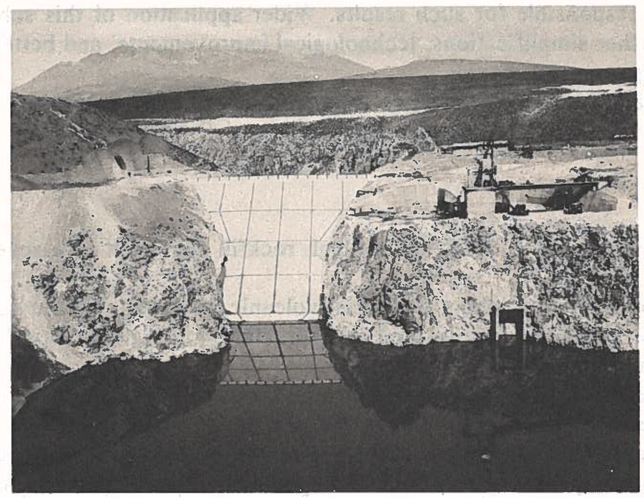

FIG. 11.—Aguada Blanca Dam in Operation

The entire curtain was done using the split spacing procedure of drilling and grouting. Primary hole spacing was 16.5 ft (5 m) and minimum final spacing was 4.1 ft (1.25 m) at sporadic locations.

Stop grouting was the rule. Areas of weak materials or very deep holes were stage grouted.

A water-cement mix was adopted with varying grout consistency. The water-cement ratio was gradually decreased from 6/1 to 2/1. After taking 50 cement bags, sand, and volcanic glass from nearby, pyroclastic deposits were added in the same quantity of cement, reducing the water-solids ratio to 1/1. The grain size of sand and volcanic glass was uniform with 100% passing no. 16 (1.2 mm) sieve. Alluvial and lacustrine pyroclastic deposits were grouted with a water-cement-bentonite mix to reduce viscosity and increase penetration. Bentonite was added up to 6% of cement weight.

Grouting pressures were 5.6 psi (39 kN/m^2) per m of depth from grouting surface.

The main data on grout and drainage curtain are given in Table 4.

Conclusions

Considering the unusually severe conditions at the Aguada Blanca site and comparing this job with others similar in general area and built under analogous premises, the writers believe that Aguada Blanca can be considered a dam of straightforward and simple construction. The general confidence in work quality was also higher than experienced by the writers in other jobs.

During the first years of operation, although stressed with rather fast fillings and drawdown rates the structure, shown in Fig. 11, performed in a satisfactory manner. Piezometers indicate 100% head loss across the facing and 50% to 90% across the grout curtain.

The writers believe that the metal facing, in connection with a rockfill support, is largely responsible for such results. Wider application of this scheme could permit further simplifications, technological improvements, and better construction timing.

The final construction costs, on a 1970 basis, are given in Table 5. The Bank for Inter American Development financed the civil works.

Summary

Aguada Blanca is a 148-ft (45-m) high rockfill dam built in a severe climate area.

Site geology resulting from alternating volcanic formations and fluvio-lacustrine sediments, unusually complex and unfavorable to any rigid structure, suggested the adoption of a fill type dam.

Because of site morphology, anticipated construction difficulties, and quality requirements, a rockfill type dam with metal facing was chosen.

The dam's outer slopes are 1.7/1 upstream and 1.4/1 downstream. The fill material is compacted quarry rock throughout, except for two transition layers of finer gradation supporting a 3/16-in. (5-mm) thick metal facing.

Metal has been specified as iron with a low level of foreign components to reduce rust susceptibility. In addition, metal has been protected with acrilic paint.

Facing flexibility, needed to absorb thermal expansions, has been provided through a system of accordeon type joints with special crossing arrangement to avoid localized rigidity.

The final overall construction cost of facing, including paint and bedding, was \$4/sq ft (\$43/m^2).

Satisfactory construction and performance of the Aguada Blanca Dam is believed to be largely due to the adoption of a metal facing.

Acknowledgment

Among the many persons whose confidence and support were decisive to the acceptance of unusual design concepts and determinant at several stages of its realization, the writers wish to remember George E. Bertram and Alessandro Gallico. They equally wish to express their gratitude to Raul Flores Gonzales Director de Infraestructura de Riego of Ministerio de Agricultura del Perù for granting permission for publication of this paper.

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Aguada Blanca Rockfill Dam with Metal Facing[a]

Closure by Piero Sembenelli,[4] M. ASCE and Marco Fagiolo[5]

The writers would like to provide the following answers to Doughty's questions which essentially dealt with two points: (1) Rockfill placement technique; and (2) dam post-construction deformations.

The rockfill forming the bulk of the dam was actually placed by bulldozer in 2.5-ft (0.8-m) lifts and compacted with four passes of a 6-ton (6,000-kg) smooth drum vibratory roller (see Fig. 7). No sluicing was attempted. The same placement procedure was used for the entire dam, the only difference being that for the lower 49 ft (15 m) rock was hauled by truck to the fill level while for the upper 99 ft (30 m) rock was chuted down to the fill level from the crest road and then spread using a bulldozer and compacted as described.

Dam deformations were anticipated as rather limited because of compaction and arching phenomena, greatly favored by the narrowness of the gorge and the roughness of the rock walls. Crest settlements on the order of 0.002/0.003 H were our guess and a crest camber of 15 in. (380 mm) = 0.009 H was designed. Actual settlements obtained leveling a line of three references between 2 BM as shown in Fig. 12 are on the order of 5/8 in. (16 mm) nearly 0.0003 H or one-tenth of our guess.

By far more important deformations were expected on the metal facing.

A domelike rise above the ideal plane of the slope was introduced in the design by thickening the 380 mm outer transition layer. A maximum rise of 15 in. (380 mm) was designed.

Facing deformations were surveyed with a deformometric torpedo traveling in a plastic casing placed just under the metal plate. The torpedo measures the angle between two successive portions of the casing each 1.969 in. (50 mm) long. The instrument (manufactured by Officine Galileo of Florence, Italy) is composed of a reference body and a measuring arm and senses two components of the deformation through a couple of vibrating wires with the accuracy of 1/10 000 (0.1 mm/m). By exploring the entire casing length from crest to toe in successive steps of 1.969 in. (50 mm), a profile of the casing can be obtained at any time and any reservoir level. The error accumulated over the total casing length is eliminated, allowing the profile to pass through two points of known coordinates: the lower casing end (fixed because embedded in rock) and the upper casing collar with settlements and horizontal movements that are obtained by leveling and goniometry. Accumulated permanent facing deformations, in a direction normal to the metal plate (Fig. 13), are on the order of 15 in. (380 mm), as the profiles obtained in July, 1972 and November, 1973 indicate.

[a] January, 1974, by Piero Sembenelli and Marco Fagiolo (Proc. Paper 10270).

[4] Head, Soil Mechanics Dept., ELC-Electroconsult, Milano, Italy.

[5] Head, Site Supervision Office, ELC-Electroconsult, Milano, Italy.

Recoverable deformations produced by full reservoir load are on the order of 4 in. (100 mm), as the profiles of November, 1973 and March, 1974 indicate.

Measurements of facing deformation also confirmed the anticipated toe spreading phenomenon. We estimate a total spreading on the order of 8 in. (200 mm) based on an actual outward movement of the toe of the slope of nearly 6 in. (150 mm) between June, 1971 and November, 1973.

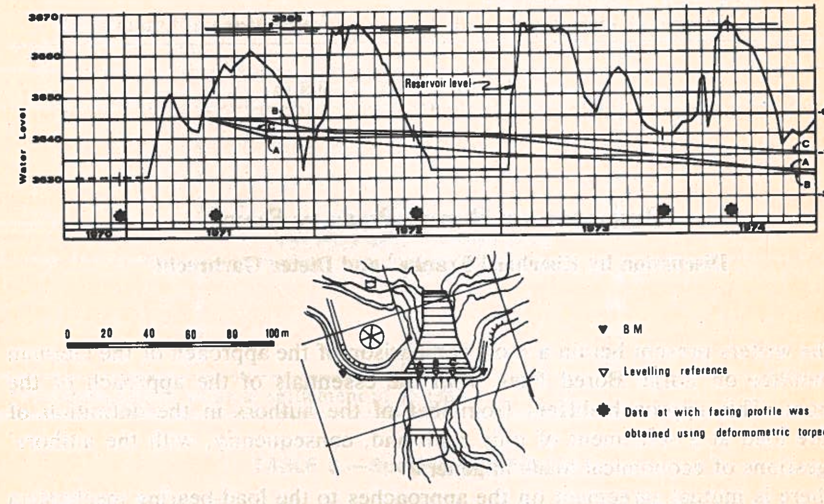

FIG. 12.—Aguada Blanca Dam—Reservoir Operation and Crest Settlements

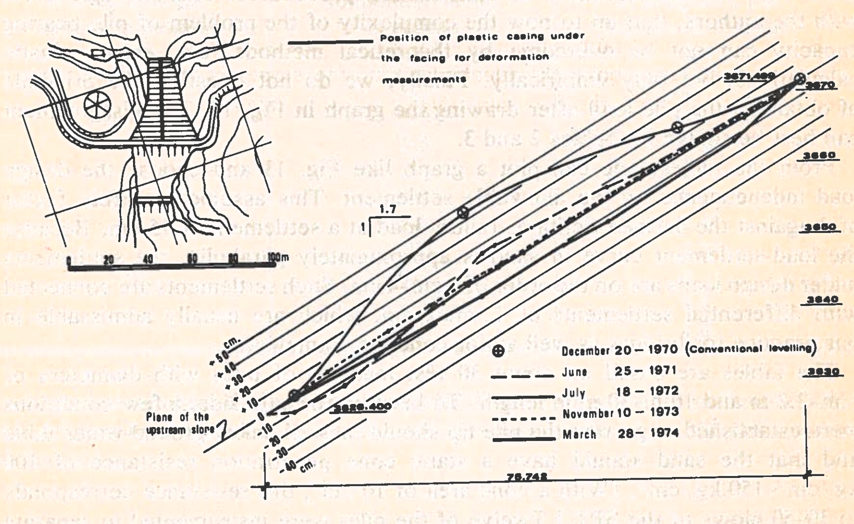

FIG. 13.—Aguada Blanca Dam—Deformations of Upstream Metal Facing during First 4 yr of Operation Obtained through Deformometric Sonde along Plastic Casing Placed under Facing

Errata.—The following corrections should be made to the original paper:

Page 50, paragraph 4, line 3: Should read "Piezometers readings indicate head losses, across the two-line grout curtain, varying from 30%-60% of the total head." instead of "Piezometers indicate 100% head loss across the facing and 50% to 90% across the grout curtain."

BEHAVIOR OF BORED PILES IN SAND[a]

Discussion by Eberhard Franke[3] and Dieter Garbrecht[4]

The writers present herein a short comparison of the approach of the German Committee on Large Bored Piles, with the essentials of the approach of the authors. This approach differs from that of the authors in the definition of failure load at a settlement of only 1 in. and, consequently, with the authors' suggestions of economical loads in general.

There is mutual agreement on the approaches to the load-bearing mechanism in Fig. 1, the substantial interdependence of load-settlement behavior of bored piles, and the fortuitousness of construction procedures. There is agreement with the authors, that up to now the complexity of the problem of pile bearing capacity can not be overcome by theoretical methods (e.g. elastic plastic calculations) but only empirically. Finally, we do not question the principle of obtaining the pile load after drawing the graph in Fig. 13. The disagreement can best be shown by Tables 2 and 3.

From the tables, one can plot a graph like Fig. 13 and choose the design load independence on the allowable settlement. This assumes a safety factor of 2 against the defined fictitions failure load at a settlement of 15 cm. Because the load-settlement curve in sand is approximately parabolic, the settlements under design loads are on the order of 2 cm-4 cm. Such settlements are connected with differential settlements of 1 cm-2 cm, which are usually admissable in our practice for bridges as well as for concrete framework.

The tables are based on about 30 test loadings of piles with diameters of 1 m-2.2 m and 10 m-30 m in length. To keep on the safe side, a few conditions were established, e.g., that the pile tip should be well below ground-water table and that the sand should have a static cone penetration resistance of 100 kg/cm^2-150 kg/cm^2. [With a cone area of 10 cm^2, this resistance corresponds to 30-50 blows of the SPT.] Twelve of the piles were instrumented to separate tip resistance from skin friction. But it has proved that the accuracy of all kinds of instrumentation, based on strain measurement of concrete is rather poor, because of the variability of the modulus of compressibility of the concrete with both stress and locality. Provided a sufficient number of results of test loadings is available, there is another possibility to separate tip resistance and skin friction by dimensional analysis, which is now prefered for the refinement of the tables.

TABLE 2.—Specific Tip Resistance

Settlement,[a] in centimeters (1)	Tip resistance, in kilograms per square centimeter (2)
(a) Piles without Enlarged Base	
1	9
2	14
3	19
15	40
(b) Piles with Enlarged Base	
1	6.5
2	10
3	12
15	27

[a]Fictitions failure value at settlement of 15 cm.

TABLE 3.—Specific Skin Friction

Static cone resistance, in kilograms per square centimeter (1)	Depth below foundation level, in meters (2)	Skin friction, in kilograms per square centimeter (3)
50	—	0
50-100	0-2	0
	2-5	0.3
	5	0.5
100-150	0-2	0
	2-7.5	0.45
	7.5	0.75
150	0-2	0
	2-10	0.6
	10	1.0

[a]July, 1974, by Fadlo T. Touma and Lymon C. Reese (Proc. Paper 10651).

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