

Hydraulic Analysis of Tiwanaku Aqueduct Structures at Lukurmata and Pajchiri, Bolivia

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(Received 22 December 1988, revised manuscript accepted 27 March 1989)

Recent archaeological research at the urban settlements of Lukurmata and Pajchiri on the southern shores of Lake Titicaca, Bolivia has revealed that open channel flow aqueducts were critical elements in the technological repertoire of Tiwanaku hydraulic engineers. Six stone faced and rubble-filled aqueducts directly linked to partially modified and canalized natural *quebradas* were identified, mapped, and dated by ceramic and structural associations to the Tiwanaku IV–V phases (c. AD 400–1100) at these two sites. A 50 m segment of the smallest aqueduct at Lukurmata was completely excavated to its base, while the intentionally shaped channel of a massive aqueduct at Pajchiri was surface profiled and mapped at intervals of 3 m over a nearly 90 m long, mid-structure segment. The modern theory of the hydraulics of open channel flows is applied to the analysis of aqueduct channel morphology, slope, structural characteristics, and design intent. The analysis concludes that these aqueducts functioned primarily as drainage structures which served to reduce the infiltration of surface water into the groundwater reservoir, and not to irrigate closely associated raised field, agricultural complexes. Additionally, the paper evaluates the general hypothesis that Tiwanaku state hydraulic engineering in the specific context of the urban environment concentrated on designing drainage systems that were intended to control local conditions of ground saturation, mass wasting and sediment transport.

Keywords: HYDROLOGY, TECHNOLOGY, AGRICULTURE, SOUTH AMERICA, ANDES, BOLIVIA.

Introduction

During the 1987 field season of the *Proyecto Wila Jawira*, a multinational, interdisciplinary research project investigating agricultural technology and organization in the ancient Andean state of Tiwanaku (Tiahuanaco), a series of hydraulic structures that were identified as aqueducts became the focus of analysis at the urban settlements of Lukurmata and Pajchiri. A total of six aqueducts, four at Pajchiri and two at Lukurmata, were documented during this season. These aqueducts represent the first surface water transport structures to be definitively associated with Tiwanaku urban sites.

Each of the aqueducts at Lukurmata and Pajchiri is associated with adjacent raised field complexes. Therefore, our initial hypothesis concerning the function of these hydraulic

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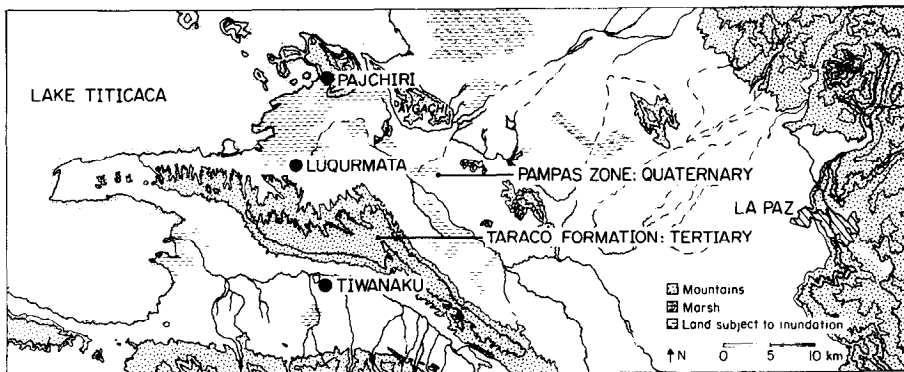


Figure 1. Region of site study: Catari sub-basin.

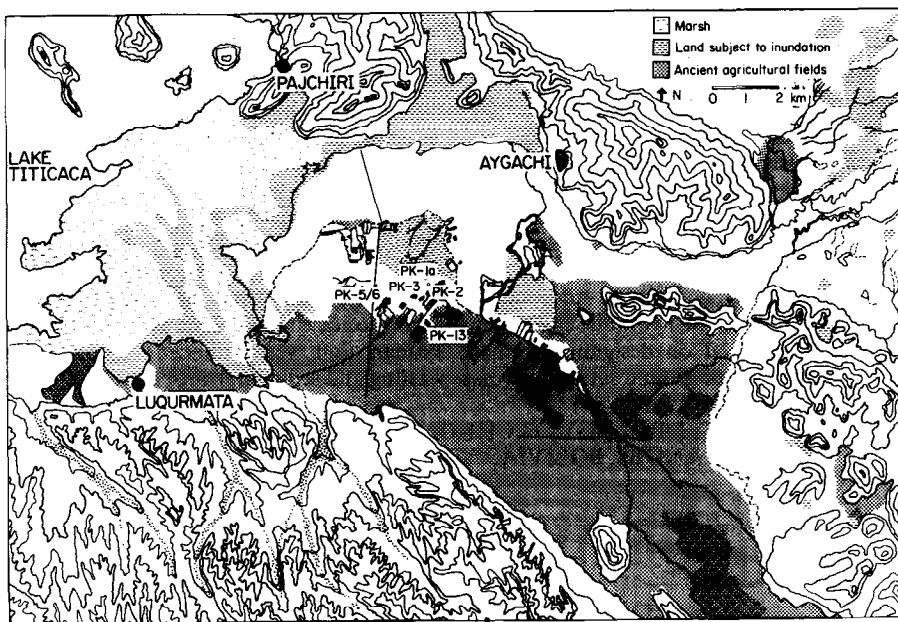


Figure 2. Detailed map of the Pampa Koani and adjacent mountain slopes.

structures was based on the assumption that they served the purpose of distributing fresh water to agricultural field systems. However, detailed analysis of local hydrological conditions and of aqueduct design suggests a second hypothesis: the hydraulic structures were intended primarily to reduce the infiltration of surface water into the groundwater reservoir, and to mitigate the impact of landscape erosion and mass sediment transport within the urban environment. In this paper we describe the results of detailed archaeological and hydraulic analyses of two aqueducts, one each at Lukurmata and at Pajchiri, and evaluate these alternative hypotheses with respect to aqueduct function.

Research Setting

The Tiwanaku secondary urban settlements of Lukurmata and Pajchiri are located on the southern shores of Lake Titicaca between $16^{\circ}45'S$ and $68^{\circ}30'W$, in an area denominated hydrologically as the Catari sub-basin (Unzueta, 1975). As indicated in Figures 1 and 2,

this hydrological unit is delimited on the north and east by a massif formed of Paleozoic rocks; on the south, by a low range of mountains of Tertiary origin (the Taraco formation) which forms a peninsula dividing the Catari sub-basin from that of the Valley of Tiwanaku; and on the west by Lake Titicaca.

Between these two principal mountainous outcrops, a vast plain that was the locus of massive Tiwanaku agricultural reclamation projects slopes gradually from the edge of Lake Titicaca towards the northeast (Kolata, 1986, 1987). This plain, referred to locally from west to east as the Koani, Wila Jawira, and Munkaña pampas respectively, consists of Quaternary fluvial and lacustrine clay, sand and gravel deposits with occasional outcrops of Tertiary rock, principally sandstones (Figures 1 and 2). Remnants of ancient lakebed terraces and multiple meander scars from rivers and streams are distinguishing geomorphological features of the Catari pampas zone. Large alluvial fans spread out from distal ends of the numerous *quebradas* that punctuate the mountain ranges on the north and south sides of the Catari drainage. Tremendous seasonal water flow in these *quebradas*, driven by sustained torrential downpours during the *altiplano* rainy season (December to March), transport enormous quantities of heterogeneous erosional products, ranging from fine-grained sediments and small clasts to boulders of substantial size.

Local Hydrology in the Catari Sub-Basin

Local hydrological conditions within the Catari sub-basin heavily influence contemporary land use, and have important implications for the interpretation of the pre-Hispanic agricultural landscape. Today over 90% of the Catari pampas zone is given over to pasturage with only scattered plots of small-scale, shallow furrow subsistence agriculture in areas of topographic highs (MACA, 1974, 1985). Local *campesino* informants explain that the pampas zone is “only good for forage for cattle, pigs, sheep and goats”. These informants (residents of the villages of Lakaya, Chokara, Quiripujo and Korila on the southern side of the Catari sub-basin near the archaeological site of Lukurmata) uniformly state that potatoes would “never grow” in the pampas because the swampy conditions of this zone would cause the plants to rot and to develop fungus.

Nevertheless, it is precisely in this pampas region that the state of Tiwanaku reclaimed extensive tracts (estimated at $> 70 \text{ km}^2$) of flat land for intensive agricultural production utilizing a system of raised field cultivation (Kolata, 1986, 1987; Kolata & Ortloff, 1989). This dramatic shift in land use and production systems over the past 800 years implies either a substantial change in local hydrological conditions that severely reduced the viability of subsistence crop cultivation, or differential technological response by populations to similar environmental conditions. To date, there is little evidence to suggest significant change in regional hydrological conditions over the past millennium. Here we explore the latter explanation for the radically different patterns of pre-Hispanic and contemporary land use evident in the pampas.

The elevated local water table characteristic of the marshy pampas zone in the Catari sub-basin results from the interaction of several geological and climatic features. There are two principal aquifers in this sub-basin: (1) one confined in the Tertiary sandstones of the mountainous Taraco formation, and (2) a second one beneath the Quaternary sediments of the pampas zone. The Tertiary aquifer is recharged by seasonal precipitation. The Quaternary aquifer, however, has several sources of recharge: (i) direct seasonal precipitation; (ii) permanent subterranean groundwater seepage from the higher Tertiary aquifer; (iii) permanent surface flow from springs in the adjacent mountain slopes, particularly from the Tertiary Taraco formation to the south; (iv) seasonal torrential surface flow driven by rainy season downpours on the relatively impermeable, adjacent

mountain slopes; and (v) deep seepage from surface rivers and streams that cross the pampas.

Given these five significant vectors of fresh water recharge, the geological characteristics of the Catari sub-basin, and the climatic cycle of a marked alternation between dry and wet seasons, it is not surprising that the pampas zone is perennially marshy and subject to seasonal inundation. Throughout the year, the merging of low gradient aquifer (calculated in the area of Lukurmata at 0.0003–0.0006) and ground surfaces creates pools of standing water. These pools gradually diminish in size through evapotranspiration during the height of the dry season (June–September) leaving behind thick alkaline crusts.

In this context of perennial high groundwater conditions, the absence of constructed drainage systems will result in a profoundly debilitating environment for agricultural production: we can predict under these circumstances a combination of waterlogging of plant root systems with surface deposition of soluble salts induced by high evapotranspiration rates of standing water. Efficient intensive agricultural production in this environment requires artificial drainage that will maintain a locally regulated water table level with respect to ground surface. Raised fields themselves, by elevating planting surfaces, represent one technological response to the problem of drainage and the regulation of relative groundwater level (see Darch, 1983 for a recent summary of research on drained field agriculture in the Americas). Aqueducts and major drainage canals that reduce the input of subterranean and surface water into local aquifers by directing flows away from reclamation zones represent a second technique for maintaining appropriate groundwater levels. It is this latter technological response that is the focus of this paper.

The Lukurmata Aqueduct/Canalized Quebrada System

The two aqueducts definitively identified at Lukurmata share important structural and design characteristics. These key construction features can be appreciated in the detailed archaeological analysis of one of these hydraulic structures presented in graphic form in Figures 3–7. The entire drainage system entailed construction, or modification of two linked, spatially contiguous components: (1) an upper channel that was formed by an artificially modified, natural *quebrada*; and (2) a lower channel that consists of a constructed, elevated aqueduct that provides passage for water over an open field for eventual discharge into a moat that encircles the base of the civic-ceremonial precinct at Lukurmata (Figure 3: aerial photograph).

The upper channel was stabilized by modifying the irregular natural sides of the *quebrada* through a cut and fill operation and then re-facing the new vertical channel walls with roughly-cut stone revetments (Figure 4). As might be anticipated, only the lateral walls of the channel were artificially stabilized; the *quebrada* floor was left unmodified. It is important to note that major residential terraces at Lukurmata are built up to the edge of the modified *quebrada*, and that subsidiary drainage canals from these domestic areas discharged excess water into this stabilized upper channel. However, the principal source of water flowing into the modified *quebrada* was seasonal precipitation that accumulated in a collection basin of at least one km² to the southwest of Lukurmata proper (Figure 3).

The modified *quebrada* articulated with the completely artificial lower channel (the aqueduct itself) at a point near the modern road that bisects the site (Figure 3). A good portion of this critical articulation point is obscured by fill of the modern roadbed, and excavation to clarify the structural features of this articulation was not feasible. As is apparent from Figures 5 and 6, the aqueduct is trapezoidal-shaped, consisting of parallel external walls made of river cobbles laid without mortar that were designed to retain an internal unconsolidated structural fill of river cobbles, earth, gravel, and sand. Tiwanaku ceramics incorporated in this construction fill suggest a general chronological placement of the structure in the Tiwanaku IV–V phases (c. AD 400–1100), a dating consistent



Figure 3. Aerial photograph of Lukurmata with canalized *quebrada* indicated.

with an associated, structurally compatible raised field complex for which we have one calibrated AMS radiocarbon date of $AD\ 950 \pm 100$ (ETH-3178).

The external walls of the aqueduct are cobble-lined throughout its entire length, but only the east wall, with a bonded corner that structurally joins the aqueduct orthogonally with a massive terrace retaining wall, remains in a good state of preservation (Figure 7). Excavations on the west edge of the aqueduct revealed fragments of the foundation stones for the parallel retaining wall, but the remaining portions of this wall were apparently destroyed by repeated erosional events. The distal end of the aqueduct curves toward the northeast to discharge into a large drainage canal (the Lukurmata moat illustrated in Figure 3) that flows into Lake Titicaca.

Since the flow rate of water through the aqueduct/canalized *quebrada* system depended upon the intensity of rainfall runoff, a standard, internally lined channel of a given shape and cross-sectional area was not constructed by the engineers of this ancient aqueduct. Instead, precipitation runoff was directed onto the surface of the fill and permitted to cut a natural erosional channel. Presumably if the channel was carved out during a near maximum flow rate event, this section would be sufficient to contain the current of all lesser flows. Although this technique of channel creation is unusual, it nevertheless represents a viable method for accommodating input flows that varied considerably in

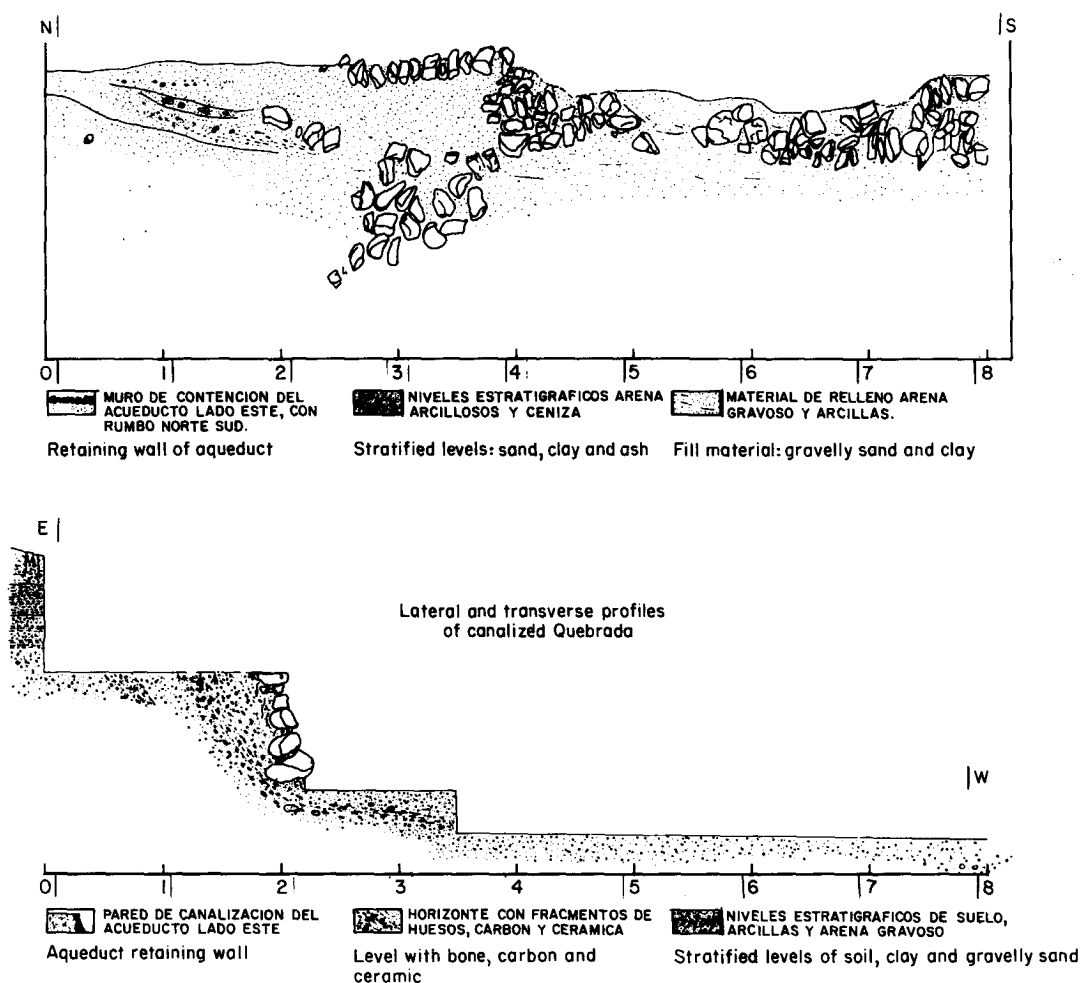


Figure 4. Transverse and lateral profiles of the Lukurmata canalized *quebrada*. As explained in the text, the canalized *quebrada* articulates with the lower, entirely artificially constructed aqueduct.

magnitude and duration: in theory, the eroded shape will be a stable, minimum-resistance profile for a given flow rate and specific energy. An excavated cross-section of the aqueduct clearly illustrates a sequence of such eroded, minimum-resistance channels within the construction fill (Figure 8). The reconstructed hydraulic characteristics of these internal aqueduct channels are analysed below.

Hydraulic Analysis of the Lukurmata Aqueduct

Throughout the following discussion we utilize concepts and analytical techniques developed in the modern theory of hydraulics of open channel flows. However, it should be noted that many of the same results may be obtained by observation and documentation of empirical results from multiple controlled tests. Indeed, many complex contemporary problems in open channel flow are only amenable to treatment by controlled tests since analytical solution techniques are not yet available. In this respect, hydraulics from ancient to modern times has much in common.



Figure 5. East wall of Lukurmata aqueduct.

Basic hydraulic concepts and definitions

Several parameters govern fluid flow through a canal: bed slope (θ_b), channel wall slopes (θ_R , θ_L), base width (B), and channel wall roughness (n , the Manning Roughness Coefficient). In addition, the flow classification (subcritical, critical and supercritical) describes whether a change in cross-sectional area of a canal has a generalized upstream influence (subcritical flow), no observable upstream influence (supercritical flow), or creates a localized disturbance zone, in the form of a transition between supercritical and subcritical flow, characterized by a downstream height increase and a velocity decrease of flow. This zone of transition is termed a hydraulic jump and is characteristic of choked supercritical flows.

To determine if subcritical or supercritical flow exists in a channel, one must compute the theoretical normal depth (D_n) and the critical depth (D_c) based on channel cross-section geometry (Henderson, 1966). The theoretical normal depth is defined as the hydraulic depth at which uniform flow exists in a channel, and is obtained by solution of the Manning equation for depth given a known flow rate, incorporating the bed slope for the head loss term. If $D_n > D_c$, then subcritical flow exists on moderate slopes; if $D_n < D_c$, then supercritical flow exists on steep slopes. In the case of *subcritical flow*, the Froude number (F), defined as $F = V/(gD)^{1/2}$ where V represents the mean water velocity, g the



Figure 6. Detail of basal construction of the Lukurmata aqueduct.



Figure 7. Aqueduct terraced retention walls.

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Lateral and transverse profiles of aqueduct

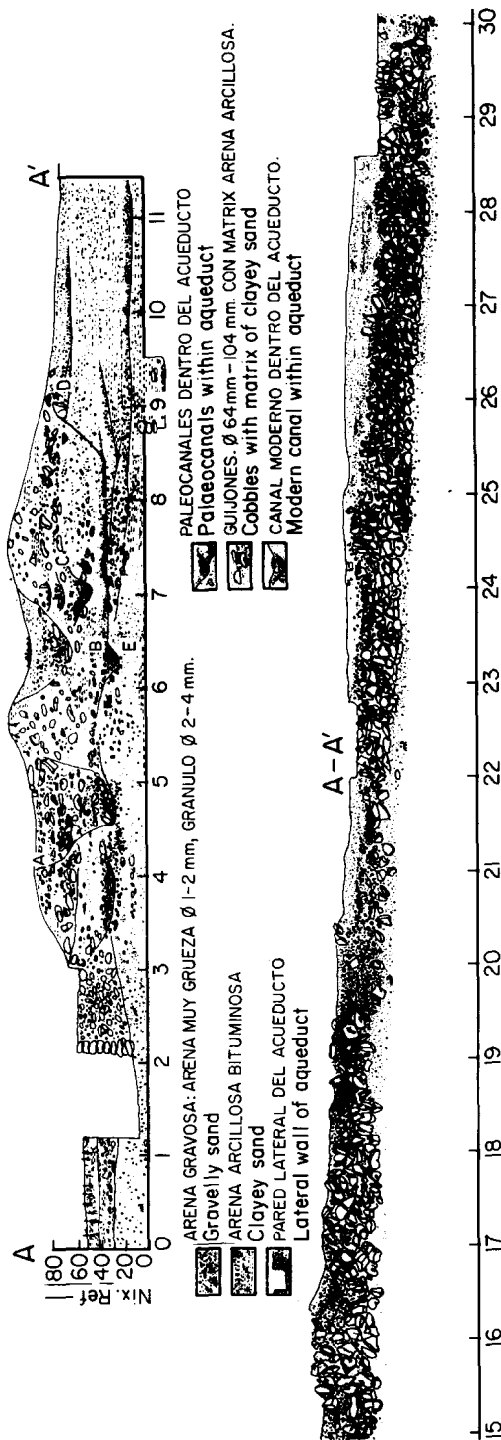


Figure 8. Lateral and transverse profiles of a 15 m section of the Lukurmata aqueduct.

gravitational constant, and D the hydraulic depth, is < 1 ; for *supercritical flow*, $F > 1$; and finally, for the special case in which $D_n = D_c$, denoted as *critical flow*, $F = 1$. Maintaining the Froude number close to unity, or theoretical normal depth close to critical depth, results in a maximum flow rate for a given intake-to-outlet height difference for a fixed channel area (Ortloff *et al.*, 1982: 581–94).

Formal hydraulic analysis

For purposes of analysis, the Lukurmata aqueduct/canalized *quebrada* may be idealized into a series of trapezoidal cross-sections along the streamwise length. In this idealized geometry, it is assumed that smooth transitions join successive trapezoids and that average section properties are used in flow computations. For each local section characterized by such a trapezoid, a bottom width B , side wall slopes θ_L and θ_R , and a bed slope θ_b can be measured, together with the streamwise distance between measurement stations, ΔS . Results of these measurements for the Lukurmata systems are given in Table 1. In this table, distances are recorded in meters, and angles in degrees. The first measured station Q' was taken at a point in the upper reaches of the channel in the canalized *quebrada*; subsequent stations are measured in the downstream direction.

The principal, as well as secondary eroded channel profiles in the aqueduct can be seen in the actual excavated cross-section of the aqueduct in Figure 8, and in abstracted form in the upper right corner of Figure 9. These profiles are slightly non-symmetrical due to fluid rotation induced by channel sinuosity upstream of the entrance region of the aqueduct. The rotation direction of the fluid is illustrated facing upstream. Stratigraphically, the profile segment designated A–B in Figure 9 is reasonably distinct, while the segment from B–D represents the remnants of an earlier channel that has been in filled. A small pocket of gravel E probably represents an erosional subcut induced by a later, low flow rate passing through the D–E–C channel.

The aqueduct fill material is composed of gravels, clays and mixtures of cobbles of various dimensions as indicated in Figure 8. The Manning Roughness Coefficient (n) is estimated to be 0.030 for aqueduct internal channel walls, and 0.025 for the modified *quebrada* walls of the upper channel (Chow, 1959). Assuming noncohesive, unconsolidated fill material and erosive behavior over the entire wetted perimeter of the channel, existing theories of hydraulics can be employed to predict the shape of the erodible section and to compare this prediction to the observed geometries. It should be noted that the erodible channel section, for a given angle of repose of fill material and a given discharge, yields a channel of minimum water area, minimum top width, maximum mean water velocity and minimum excavation. From assumptions stated in Chow (1959), then, the differential equation

$$(dy/dx)^2 + (y/y_0)^2 \tan^2 \theta = \tan^2 \theta \quad (1)$$

is derived for the erodible cross-section shape where $y = y_0$ at $x = 0$ (the channel centre: Figure 9). The angle θ is the angle of repose of fill material and the $y = f(x)$ solution represents the stable channel shape. A solution of equation (1) yields

$$y = y_0 \cos[(\tan \theta / y_0) (x)] \quad (2)$$

which is plotted in Figure 9 for various fill repose angles. Observing a repose angle of about 30–40° for excavated dry fill, the computed erodible profile is comparable in channel shape to the Lukurmata aqueduct segment A–F in Figure 9.

Of course larger flow rates would carve out larger channels and several remnant earlier channels of greater size may be observed below the latest one illustrated in Figures 8 and 9. This sequence of eroded channel profiles of varying geometries suggests the possibility of a

Table 1. Details of the upstream reach of the Luqurmata Canal. (Calculated slope, channel and flow characteristics for the lower aqueduct at Lukurmata)

Station	B	Θ_R	Θ_L	Θ_b	ΔS	D_N	D_C	F	Profile Type	h	n
Q'	5.0	15°11'	30°15'	4°48'	9.42	0.19	0.35	1.0	S-2	0.19	0.025
P	4.7	17°32'	18°10'	4°12'	23.0	0.18	0.37	2.5	S-2	0.18	0.025
O	4.7	17°32'	18°10'	4°10'	29.8	0.18	0.37	2.5	S-2	0.18	0.025
M	4.7	17°32'	18°10'	4°46'	29.7	0.17	0.37	2.7	S-2	0.17	0.025
N	7.6	11°30'	9°49'	4°46'	29.7	0.13	0.27	2.8	S-2	0.13	0.025
M'	4.0	18°50'	8°54'	3°03'	14.0	0.18	0.44	2.7	S-3	0.18	0.025
L	4.3	10°35'	9°13'	5°20'	30.0	0.17	0.43	2.8	S-2	0.17	0.025
K	8.3	17°07'	31°36'	4°02'	21.0	0.13	0.24	2.4	S-2	0.13	0.025
J	8.3	17°07'	31°36'	5°37'	3.8	0.12	0.24	2.4	S-2	0.13	0.025
I	7.2	15°47'	19°52'	5°37'	30.0	0.13	0.27	3.0	S-3	0.12	0.025
H	16.0	13°16'	9°18'	5°03'	16.0	0.08	0.16	3.7	S-2	0.12	0.025
G	5.5	9°56'	5°24'	3°36'	30.0	0.16	0.37	2.6	S-3	0.15	0.025
F	13.7	17°03'	7°54'	5°40'	30.0	0.09	0.50	2.6	S-2	0.09	0.025
E	1.8	9°56'	4°20'	5°37'	30.0	0.22	0.95	6.0	S-3	0.12	0.025
D	1.8	9°56'	4°20'	5°37'	5.0	0.22	0.95	2.9	S-3	0.22	0.025
D'				4°17'	40.8					1.01	0.03
B	4.06	10°	80°	2°24'	6.0	1.05	2.16		S-2	0.87	0.03

B = Base width (m), Θ_R , Θ_L = right and left channel sidewall slopes, Θ_b = bed slope, ΔS = streamwise distance (m), D_N = normal depth (m), D_C = critical depth (m), F = Froude number, h = water height at the end of the section (m).

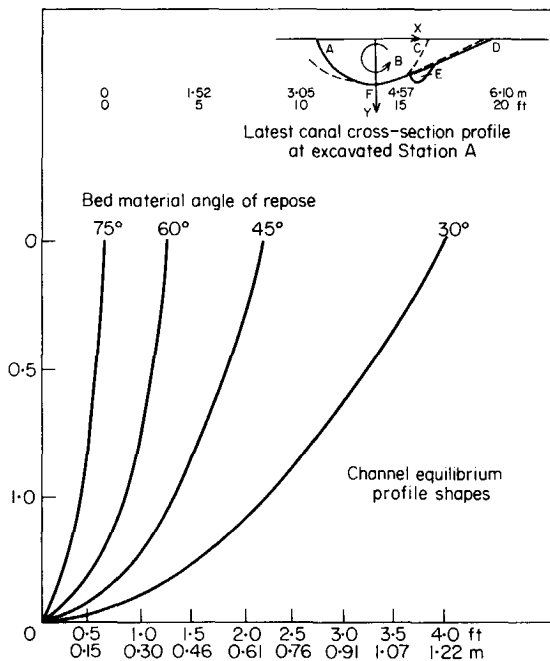


Figure 9. Channel equilibrium profile shapes.

sequence of heavy precipitation episodes in the past. For cases of large flow rate events, given stone packed lower bedding, side wall erosion leads naturally to wide, flat-bottomed profiles similar to those observed below the A-F-C channel in Figure 9 (compare also the actual profile in Figure 8). The erosion profile sequence of the observed aqueduct channels is discussed in more detail below.

In order to gain insight into the design purpose of the aqueduct, hydraulic calculations were made to assess the functioning of the system. Starting from point Q' at the uppermost reach of the drainage channel, a critical flow point is assumed together with a typical estimated flow rate of 108.5 ft s^{-3} . This value is arrived at from estimates of flow at the base of the drainage channel observed during the height of an average rainy season. Independent of the critical flow assumption at Q', the flow will soon accelerate to a supercritical flow regime due to the steep slopes (generally $> 4^\circ$) of the drainage channel (Table 1). Normal and critical flow heights are calculated for each measurement station listed in Table 1 (Chow, 1959; Henderson, 1966; Morris & Wiggert, 1972; Pashkov & Dolgachev, 1977).

For skewed trapezoidal-shaped approximations to the channel cross-sections in Table 1, channel area (A), wetted perimeter (P), hydraulic radius (R_h), hydraulic depth (D) and section factor (Z') are given by

$$A = By + (y^2/2)(Z_R + Z_L) \quad (3)$$

$$P = B + y[(1 + Z_R^2)^{1/2} + (1 + Z_L^2)^{1/2}] \quad (4)$$

$$R_h = A/P \quad (5)$$

$$D = [By + (y^2/2)(Z_R + Z_L)]/[B + y(Z_R + Z_L)] \quad (6)$$

$$Z' = [By + (y^2/2)(Z_R + Z_L)]^{3/2}/[B + y(Z_R + Z_L)]^{1/2} \quad (7)$$

where $Z_R = \cot \theta_R$, $Z_L = \cot \theta_L$ and y is the water height from the base of the trapezoid. Critical depth is determined from a solution for y_c from

$$Q = \sqrt{gZ'} \quad (8)$$

while normal depth Y_N is determined from the Manning Equation for a given flow rate (Q) and local bed slope (Chow, 1959; Henderson, 1966; Morris & Wiggert, 1972; Pashkov & Dolgachev, 1977). Because the flow is supercritical throughout, calculations can be performed in a downstream stepwise manner starting from the critical section. The Manning Equation is given by

$$Q = (1.49/n) (R_h^{2/3}) AS^{1/2}$$

where $Q = VA$ and V is the average velocity in a cross-section. From Morris & Wiggert (1972),

$$\Delta S = \int_{h_i}^{h_j} \frac{(1 - Q^2 B/g A_m^3) dD}{i_b - (nQ/1.49 R_{h,m}^{2/3} A_m)^2} \quad (9)$$

where integration is over y from substitution of equations (3)–(6) in (9). Here $dD = (dD/dy)dy$. Integration is performed numerically for ΔS .

In equation (9), Q is the channel flow rate and i_b the local bed slope. The i, j subscripts on h denote a y height increment over which the integration is carried out to produce a streamwise length increment, ΔS . The quantity g represents the gravitational constant of 32.2 ft s^{-2} . For rectangular cross-section canals, $D = Y$. The m subscript represents a mean value of parameters between integration limits.

Calculations produce details of the flow profile and velocity in the drainage canals. Results from the measured stations on the Lukurmata drainage system summarized in Table 1 indicate that the flow height of a given station approaches the local value of normal depth Y_N for each adjacent station. As is typical for supercritical flows on steep slopes, the normal depth asymptote is approached rapidly at each measurement station. As indicated in Table 1, the local Froude number ranges between 2.5–3.7 in the upper channel, while the Froude number at station D, the entrance region to the aqueduct, is 2.9. Flow depth at D is equal to the normal depth (0.73 ft) at the aqueduct entrance region. Thus, the channel flow upstream of the aqueduct is smoothly transitioning in depth between stations, close to the local normal depth, and locally uniform throughout its length. The upper drainage channel, then, efficiently transports water such that *locally* the fluid surface is parallel to the channel bottom. A check of the Vedernikov number (Chow, 1959) produces values less than unity, indicating flow stability on the steeply sloped upper channel.

The purpose of these computations is to determine typical flow conditions at the entrance region to the lower aqueduct. The design of this entirely man-made hydraulic structure offers the deepest insights into the expertise of Tiwanaku engineers, and the nature of the problems they perceived in attempting to contain seasonal flows within the drainage channels of the Lukurmata urban environment. Since the aqueduct channel is erodible, the first calculations are designed to determine if the flow in the lower aqueduct is still uniform and supercritical for the parabolic channel sections shown in Figure 8. To this end, equations (3) to (9) are employed, and the results reproduced in Table 1.

The principal problem with respect to the hydraulic characteristics of the lower aqueduct is to determine whether the geometry and structural features of this channel generate a hydraulic jump, or support a smooth transitional supercritical flow. If a hydraulic jump developed at the juncture of the upper and lower channels, then severe erosion would have (periodically) destroyed the aqueduct fill. To answer this question, Figures 10 and 12–14 are constructed utilizing modern hydraulic theory (Chow, 1959).

On plots of flow rate versus lower aqueduct slope, the critical slope line demarcates the presence of critical flow. According to Figure 8 results, assuming a rectangular canal cross-section, a channel with a slope of 0.015 supporting a flow rate of 26 ft s^{-3} with a flow depth of $Y = 1 \text{ ft}$ would then be subcritical for a $B = 5 \text{ ft}$ bottom channel width. A slope greater than 0.020 would support a supercritical flow. For a subcritical flow on the lower aqueduct, a hydraulic jump *must* occur to transition the upper channel supercritical flow to subcritical flow. Since the measured aqueduct slope is 0.043, it can be concluded that the lower aqueduct supports a supercritical flow for the class of $B = 5 \text{ ft}$ channel widths. Continuation of the search for possible subcritical flows among the class of wider eroded profiles characterized by a bottom width of $B = 2.5 \text{ ft}$ (Figure 12), $B = 10 \text{ ft}$ (Figure 13) and $B = 15 \text{ ft}$ (Figure 14) for a range of possible Y flow depths that the aqueduct could support reveals that subcritical flow does not exist on the lower aqueduct at the measured slope of 0.043.

A calculation made for a non-rectangular channel cross-section (Figure 10) reveals that the rectangular cross-section calculations are conservative with respect to slopes at which the critical flowline exists. Therefore over a variety of eroded channel bottom widths, flow depths and flow rates, the lower aqueduct *always* supports a supercritical flow. Accordingly, for a wide class of eroded profile shapes (Figure 8), there exists a smooth supercritical transition from the exit normal depth on the upper channel to a higher normal depth on the lower aqueduct. The value of this depth can be obtained by solution of the Manning equation for given Q , n and B for any eroded profile shown in Figure 8.

Past station A, the aqueduct curves smoothly to the northeast while maintaining its full structural width. Coordinates (x, y) of the curved section are given in Table 2. By

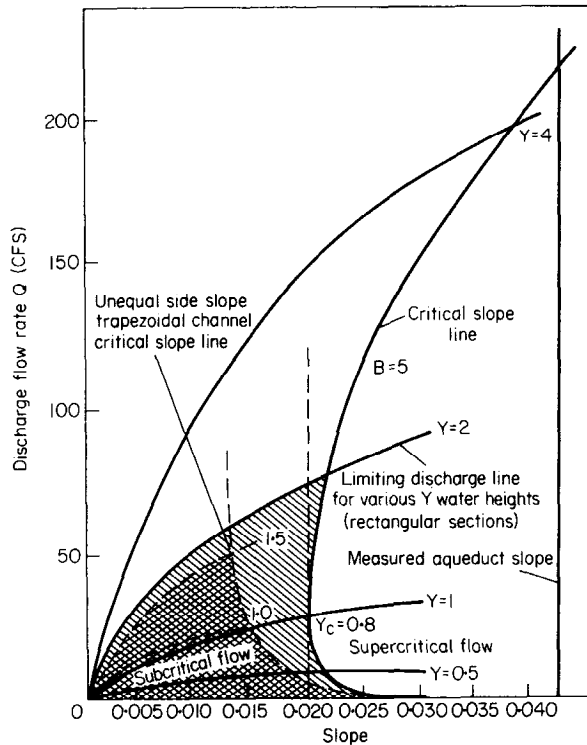


Figure 10. Calculated flow rates and flow characteristics for various channel slopes.

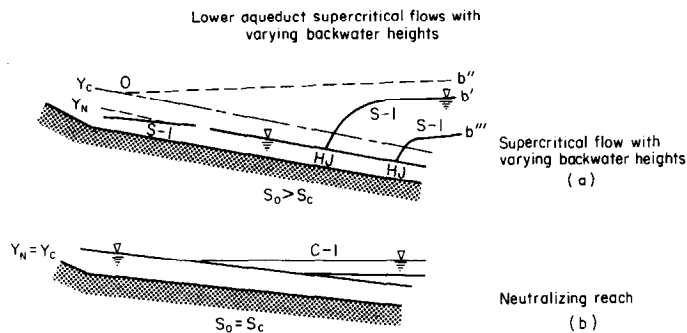
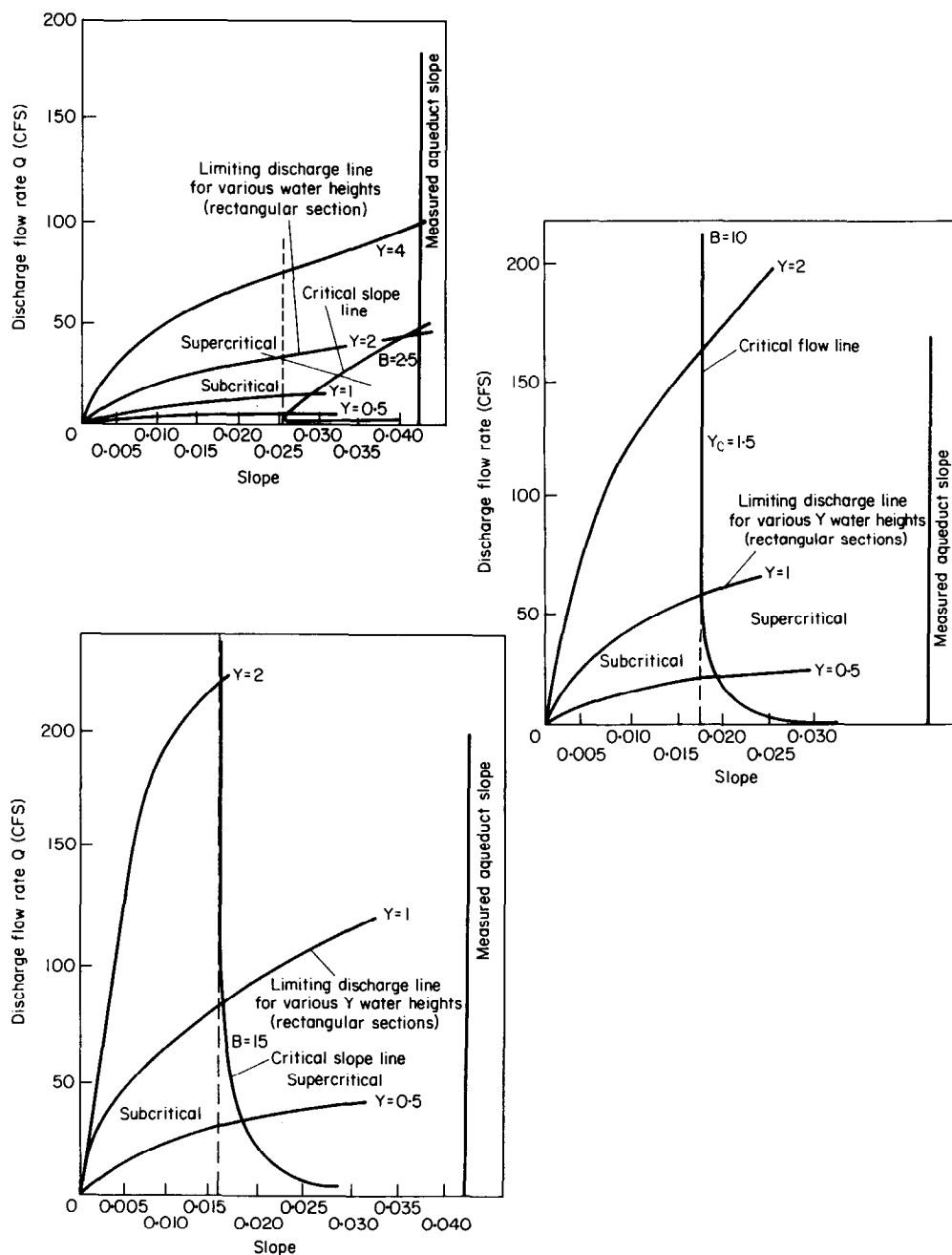


Figure 11. Lower aqueduct supercritical flows with varying backwater heights, Y_N , Normal depth; Y_c , critical depth; S , slope; HJ , hydraulic jump; S_c , critical slope; S_0 , actual slope.

characteristic methods, the supercritical turning of the flow and height change can be determined (Chow, 1959; Henderson, 1966; Morris & Wiggert, 1972). However, the key problem here is the effect of the backwater height on aqueduct flow. This height is basically the current lake level which is known to vary considerably in historical times: the lake



Figures 12-14. Critical slope line for varying channel depths. The critical slope line demarcates the presence of critical flow.

level has fluctuated as much as 5 m within a 2 year period [Carmouze & Aquize Jaen, 1981: Figure 3(b)]. For the case in which this height is high and intersects the supercritical aqueduct flow, serious erosion problems potentially destructive to the structural integrity

Table 2. Coordinates of the aqueduct curved east wall (distal end)

ΔS (from A) (m)	x (m)
0	0
0.50	0
1.00	0.30
1.50	0.56
2.00	0.92
2.50	1.38
3.00	1.92
3.50	2.24
4.00	3.33

of the aqueduct could be generated by the development of a highly turbulent hydraulic jump. Referring to Table 1 and Figures 10 and 11(a), the supercritical flow on the aqueduct approaches the normal depth line Y_N on an S-2 profile (see Chow, 1959 for a discussion of these profiles). If the backwater height is below Y_N , then no upstream effect is induced in the supercritical flow. If the backwater level is high (b'''), however, then a hydraulic jump (HJ) will occur on the aqueduct leading to massive erosion of the unconsolidated, unlined canal fill in the vicinity of the jump's water height increase. For instance, for the reconstructed hydraulic characteristics of the lower portions of the aqueduct in which $F = 2$, and the bed slope of the channel $= 0.043$, the hydraulic jump height can approach three times that of the pre-jump height. As the backwater height increases, the jump moves up the aqueduct, causing additional damage to the hydraulic structure through turbulent fluid action in the erodible channel fill. Moreover, in this situation of high backwater levels, we would anticipate that this already unconsolidated structural fill would be further compromised by the wetting action of the intersecting lake level. The merging of aqueduct flow and lake backwater will destroy the local fill region by the action of unsteady, turbulent mixing within the hydraulic jump.

As the backwater height further increases [Figure 11(a)], the theoretical limit $0-b''$ on an S-1 curve (Chow, 1959) is reached, and the aqueduct is submerged. The geometry of the curved, distal portion of the Lukurmata aqueduct slightly raises the height of the flow over the normal depth through Froude wave interaction, while slightly decreasing the supercritical velocity (Table 2). Here the effect of backwater height change is to produce an *intermediate* step-up in fluid height before the hydraulic jump adjustment to the backwater height described above. This effect limits the erosional potential of the hydraulic jump on the lower reaches of the aqueduct, and suggests that the Tiwanaku engineers may have explicitly incorporated this curved section of the aqueduct into their design to mitigate structural deterioration.

Another elegant solution to eliminate, or mitigate the impact of potentially destructive hydraulic jumps on the lower reaches of an aqueduct is to construct a neutralizing reach (Chow, 1959; Bakhmeteff, 1932). This entails establishing the aqueduct bed slope equal to the critical slope, thereby causing backwater levels to approximate horizontal lines (C-1 profiles) which intersect the surface of the flow without causing turbulence [Figure 11(b)]. This is equivalent to an hydraulic jump of zero height. The curve for the trapezoidal section critical slope (S_c) is found from selection of Y values in equation (8) to produce Q (or discharge rate of flow) values. The Q and Y values are then substituted into the Manning Equation to determine S_c values. This method is used to determine S_c curves in Figures 10, and 12–14.

For the A-F-D channel (Figure 9), the intersection of the critical slope line with a $Y = 2$ ft height flow occurs at a critical slope of approximately 0.02 (Figure 10). This critical slope is well below the measured aqueduct bed slope of 0.043, therefore no neutralizing reach was possible for the A-F-D parabolic channel as a preventive measure against canal erosion during conditions of high rainfall and high lake levels intersecting the lower reaches of the aqueduct.

Incomplete remains of earlier erosional channels appear below the A-F-D channel, (see the cross-section illustrated in Figure 8). Many of these channel remnants are partially destroyed by later erosional and filling events. Consequently, no secure time sequence of channel configurations can be readily discerned. However, it is clear that a sequence of wide-base, parabolic channels characterized by high rates of flow are intercalated with low flow rate incised channels.

Calculations based on rectangular canal sections have been made for a variety of base widths ($B = 2.5, 5, 10$, and 15 ft) in Figures 12–14 that show the critical slope line for different possible water depths in the channel. For all cases for $B > 5$ ft, a neutralizing reach is impossible for the assumed flow rate. The limiting discharge line is calculated from the Manning Equation using S_c and a given depth. The flow is still supercritical for these cases, and hydraulic jump phenomena can still be expected to occur in the lower reaches of the aqueduct channel under conditions of elevated backwater height intersecting the lower reaches of the aqueduct.

Interestingly, for hypothesized massive channel flow rates generated by torrential rainfall events, there are some possibilities of a neutralizing reach condition for $B < 5$ ft channels, if this base width can be maintained. For $B = 2.5$ ft and $Y = 2$ ft for example (Figure 12), the intersection point A lies close to the measured aqueduct bed slope of 0.043. This indicates that the channel can function as a neutralizing reach for this set of flow rate, base width and water depth conditions, thereby eliminating an on-aqueduct hydraulic jump for any backwater depth.

The basic problem encountered in the aqueduct design is that the input flow rate, which depends fundamentally on precipitation intensity and duration, varies radically from event-to-event. Therefore, a standard design channel cannot be built to perform transport tasks efficiently for a wide range of input flow rates. In this hydrological context, the erodible channel technique evident in the Lukurmata aqueduct serves design purposes well since it carves out the “necessary” channel to support a given flow rate, maintains a nearly uniform flow throughout, and, in inundation conditions when the lake level is high, can function as a neutralizing reach to eliminate on-aqueduct hydraulic jumps near the exit region of the aqueduct. The presence of several deep, flat-bottomed channel profiles with base widths in the range of 5–10 ft in the Lukurmata aqueduct suggests massive rainfall events in the early history of this hydraulic structure. The lower flow rate events are more likely to produce “permissible” velocities (Henderson, 1966) and achieve the equilibrium parabolic profiles illustrated in Figure 9.

Hydraulic Analysis of the Pajchiri Aqueduct

To gain further insight into the design of Tiwanaku hydraulic structures, we analyse one segment of an aqueduct system at the site of Pajchiri (Figure 1). Similar to the aqueduct at Lukurmata, three of the four aqueducts constituting the Pajchiri hydraulic system are characterized by two distinct components: (1) an upper channel consisting of an artificially canalized *quebrada*, and (2) a lower channel that conducted water over an open plain on an elevated structure that was constructed of parallel stone retaining walls infilled with earth, stone and gravel. As at Lukurmata, the modified *quebrada* channels of these aqueducts reach into high montane catchment basins where they were charged by precipitation runoff and by permanent springs and subterranean seeps. The fourth Pajchiri aqueduct

was not structurally associated with an artificially modified *quebrada*. Rather, it consisted of an elevated, stand-alone hydraulic structure linked by a surface canal to an artificial spring-charged reservoir adjacent to the principal civic-ceremonial precinct of Pajchiri (Figure 15). Unlike the Lukurmata aqueduct analysed above, the hydraulic structures at Pajchiri are much larger running over a kilometer in length and achieving maximum structural heights between 4–7 m. They are carefully built with large cut-stone retaining walls, rather than with the simple river cobbles that characterize the structure at Lukurmata (Figure 16). In addition, unlike the Lukurmata aqueduct, those at Pajchiri have well-constructed stone drop structures linked to smaller feeder canals that were apparently intended to distribute fresh water to local raised field systems arrayed between the aqueducts themselves. The Pajchiri aqueducts clearly demonstrate fine structural detailing, and a massive investment of labor.

Field work on the Pajchiri aqueduct chosen for analysis entailed surface measurements on preserved channel widths, lateral shapes, and canal bed slopes on an approximately 90 m mid-structure segment. Measurements were taken at 3 m intervals over this segment, commencing at a point approximately 300 m from the spring/reservoir water intakes. The results are shown in Table 3. As observed from the base width (B) column, the canal shape profile of this aqueduct exhibits an expansion-contraction design. Upstream and downstream of the analysed section the channel width is relatively uniform. The question presents itself: does this design feature of a shaped canal that expands and contracts in channel width perform a hydraulic function, or was the canal shape simply the product of non-standardized construction techniques?

Table 3 lists the normal (Y_N) and critical (Y_C) depths for assumed typical flow rates in the range of 1 m s^{-3} and 0.5 m s^{-3} from the spring/reservoir water sources. For both flow rates, $Y_C > Y_N$ indicating the presence of supercritical flow. In most cases, the difference between Y_N and Y_C is on the order of 20–30%. The flow depth is therefore close to the local value of the normal depth. Because the critical depth is not excessively divergent from the normal depth, and the slope not excessively greater than the critical slope, the flow rate will be close to the maximum possible for the given slope, channel area, and initial specific energy. To avoid flow at the critical state ($F = 1$) and its inherent instability, the flow is somewhat into the supercritical region, with a Froude number ranging from 1.5 to 2.5.

The lower normal water depths in the vicinity of the expanded channel “bulge” compared to upstream and downstream water depths, enhanced the function of the drop structures and smaller feeder canals with low intake heights constructed perpendicular to the main channel in this region. The flow velocity is small into the drop structures when the water height is only slightly above the intake height. This may aid in irrigation strategies that required slow water input over long periods rather than rapid flow over a short time with large volume flow rates. The lower intake heights of the drop structures, made possible by the locally lower water height in the expansion region, permitted easy manipulation of the secondary water distribution system: simple sluice gate structures of small stones could be used to periodically block the intakes to regulate water supplies to the feeder canals.

Despite the presence of occasional well-elaborated drop structures and secondary feeder canals, it appears that the principal function of the Pajchiri aqueducts was to transport excess water from surrounding spring and *quebrada* systems to the lake basin. There are a total of four aqueducts running through the field systems at Pajchiri. Each appears to channel water from specific local drainage basins and spring systems. The presence of near critical slope canals reinforces the transport rather than distribution model of canal function. In this respect, it is important to note that several segments of raised fields at Pajchiri were cut on laboriously constructed terraces between the



Figure 15. One of four aqueducts at the site of Pajchiri.

aqueducts. This is a clear indication of design response to high groundwater conditions and supersaturation of soils that are potentially deleterious to agricultural production.

The aqueducts at Pajchiri, then, can be best interpreted as hydraulic structures that supported continuously functioning drainage canals designed to remove excess water from areas of field reclamation, thereby maintaining a stabilized water table at a point below that of crop root systems. The occasional drop structures and feeder canals along the aqueducts may have been incorporated into the structures to address the problem of periodic droughts that afflict the Andean *altiplano*, or, perhaps somewhat more speculatively, to mitigate the potentially disastrous problem of hypersalinization on field systems by providing a direct method of suffusing them with fresh, flowing water thereby cleansing them of soluble salts. In either case, it is clear that the various features of the Pajchiri hydraulic structures represent a case of designed multifunctionality responsive to the severe inundation-drought cycles that are characteristic of the Andean *altiplano* climatic regime.

Summary and Conclusions

Two aqueduct systems at Tiwanaku urban settlements have been analysed to interpret their design features. Both appear to employ supercritical open channel flows, and serve as



Figure 16. Construction details of one Pajchiri aqueduct.

drainage systems intended to prevent excess surface water from entering the groundwater reservoir. For the Lukurmata aqueduct, an erodible channel system was constructed to collect runoff from a large, local basin and divert it directly into Lake Titicaca. This aqueduct may have features related to a neutralizing reach concept which would eliminate on-aqueduct hydraulic jumps in the presence of flood conditions and/or elevated lake levels. The aqueduct supports a flow at local normal depth values under typical usage conditions, implying a constant velocity and a constant area flow parallel to the canal bed on the aqueduct. Under flood conditions, the lower channel of the aqueduct/canalized *quebrada* system may support critical flow enhancing rapid drainage.

The Pajchiri aqueduct analysed here also exhibits a design intent of diverting excess surface water to Lake Titicaca to reduce the problem of supersaturated soils that would damage agricultural production. Since the aqueduct slopes are constructed near the critical slope, maximum flow rates from spring and drainage basins to the lake outlet of the aqueducts can be achieved. This slope configuration, along with the otherwise puzzling presence of four long, parallel aqueducts within 100 m of each other, indicates that the major hydrological concern of the Tiwanaku engineers at Pajchiri was water diversion and drainage, rather than distribution to agricultural fields, or to residential populations.

Table 3. Structural details of the Pajchiri expansion-contraction aqueduct section

Station (m)	B (m)	S_b	$Z_L = \text{ctn } \Theta_L$	$Z_R = \text{ctn } \Theta_R$	$Q = 1 \text{ m s}^{-3}$		$Q = 0.5 \text{ m s}^{-3}$	
					D_c (m)	D_N (m)	D_c (m)	D_N (m)
0	1.05	0.37	1	1	0.41	0.28	0.26	0.19
3	3.00		1	1	0.22	0.15	0.14	0.10
6	3.25		1	1	0.21	0.15	0.13	0.10
9	3.00		1	1	0.22	0.15	0.14	0.10
12	3.30		1	1	0.21	0.14	0.13	0.09
15	3.00		1	1	0.22	0.15	0.14	0.10
18	3.00		0	0	0.20	0.16	0.14	0.10
21	3.10		0	0	0.21	0.16	0.14	0.10
24	3.20		0	0	0.20	0.15	0.13	0.10
27	2.70		1	0.58	0.23	0.16	0.15	0.10
30	2.30		1	0.58	0.20	0.18	0.17	0.12
33	3.10		1	0.58	0.22	0.15	0.14	0.10
36	3.20		0	1	0.21	0.15	0.13	0.10
39	2.80		0	1	0.23	0.16	0.15	0.11
42	2.65		0	1	0.23	0.17	0.15	0.11
45	2.65	0.033	1	1	0.24	0.16	0.15	0.11
48	2.65		1	1	0.24	0.16	0.15	0.11
51	2.65		1	1	0.24	0.16	0.15	0.11
54	2.26		0	0.58	0.25	0.19	0.16	0.12
57	2.26		0	0.58	0.25	0.19	0.16	0.12
60	1.90		0	0.58	0.28	0.21	0.18	0.14
63	1.90		0.58	1.19	0.29	0.20	0.19	0.13
66	2.02		0.58	1.19	0.28	0.20	0.18	0.13
69	2.38		0.58	1.19	0.25	0.18	0.16	0.12
72	1.72		0	0	0.28	0.24	0.19	0.16
75	1.61		0	0	0.28	0.26	0.19	0.16
78	1.42		0	0	0.31	0.28	0.20	0.18
81	1.40		0	0	0.31	0.29	0.21	0.18
84	2.26	0.041	0	0	0.25	0.20	0.21	0.13
87	2.30							

Therefore, the evidence from both Lukurmata and Pajchiri implicates a design intent related to a transport and drainage, rather than distributional, strategy for hydraulic structures within these urban environments. The rationale behind this strategy can be associated directly with specific climatic, hydrological, and topographical circumstances that induced elevated local groundwater conditions in and around these Tiwanaku cities. The hydraulic engineers of Tiwanaku were apparently responding to potentially disastrous problems generated by seasonal inundation of urban and agricultural landscapes by building appropriate aqueduct systems linked to modified natural *quebrada* channels. These systems effectively served to reduce the infiltration of surface water into the groundwater reservoir, thereby lowering the local water table with respect to the ground surface. This regulation of water table height was essential to insure the long-term sustainability of agricultural fields within the boundaries of these cities as well as within their immediate hinterlands.

The canalized *quebrada*/aqueduct systems simultaneously prevented, or mitigated the impact of massive erosion and sediment transportation within residential zones of Lukurmata and Pajchiri. A substantial proportion of the populations in these two urban settlements resided on artificially constructed terraces carved into hillsides with slopes approaching 20° (Kolata, 1987). Currently, unprotected slopes in the region of these two

sites experience tremendous local erosion and, at times, mass wasting induced by seasonal torrential downpours. By canalizing this heavy seasonal flow and diverting it to the lake basin, Tiwanaku hydraulic engineers were, in effect, conserving the habitability of the urban environment. Monumental, subterranean sandstone drains, or more aptly, sewer systems, recently (1988) excavated at the capital city of Tiwanaku dramatically emphasize that state's preoccupation with efficient, large-scale drainage within the urban landscape. We anticipate that our continuing investigations into the hydraulic systems of the Tiwanaku state will reveal additional examples of sophisticated, multifunctional technological response to the environmental exigencies and potentials of the Andean high plateau exemplified in the aqueducts and canalized *quebradas* of Lukurmata and Pajchiri.

Acknowledgements

The results of the research discussed here are based on the project entitled "The Technology and Organization of Agricultural Production in the Tiwanaku State". This project is supported by research grants from the National Science Foundation (BNS 8607541; BNS 8805491) and the National Endowment for the Humanities (RO 21368-86; RO 21806-88), Alan L. Kolata, Principal Investigator. The United States Government has certain rights in this material. Additional funding was provided by grants from the Inter-American Foundation (BO-252, 1987; BO-273, 1988). We gratefully acknowledge support for computation from FMC Corporation, Santa Clara, CA.

The research was authorized in Bolivia by the Instituto Nacional de Arqueología (INAR), La Paz, Dr Carlos Urquiso Sossa, Director. We are particularly indebted to Dr Carlos Ponce Sanginés, Director Emeritus of INAR and to Oswaldo Rivera Sundt (INAR), Co-Director of the project for facilitating research activities in Bolivia. Leocalia Ticcla, Staff Geologist (INAR), recorded and drafted the aqueduct system profiles in this paper. His painstaking work and discerning eye are acknowledged and deeply appreciated.

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