University of New Mexico
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Piedras Marcadas Spillway

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A Report Prepared for the
Albuquerque Metropolitan Arroyo
Flood Control Authority

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Note: This is an expanded version of the February 5, 1996, report that focused on the proposed development engineered by Isaacson & Arfman, P.A. This document contains clarifications potentially useful to AMAFCA and/or the State Engineer in overall spillway assessment. This document does not alter the original finding that the proposed Eagle Ranch Road does not jeopardize the spillway.

Introduction

The Piedras Marcadas Dam was completed in 1984. The emergency spillway is a trapezoidal earthen channel of 200-foot bed width b and 3:1 sideslope Z. PMF design discharge is 26,500 cfs. The spillway crest is a 100-foot (flow length) horizontal apron, elevation 5032. The gradually varied profile is S2. Table 1 summarizes the hydraulics, approximating flow down the slope as normal depth (n = 0.027).

Table 1. Spillway Hydraulics

		Crest	Slope
Slope S		0.0000	0.0228
Depth y	(ft)	7.84	5.18
Velocity V	(fps)	15.12	23.72
Froude Number F		1.00	1.90

From downstream lip of the crest, the spillway turns approximately 95^{o} to the right (downstream-facing perspective) with a 400-foot radius of curvature r_c.

The spillway is stabilized by horizontal radial sills, 2-feet wide. Drop between sills is 3 feet. The sills follow the geometry of step corners on a spiral staircase, as illustrated by Fig. 1.

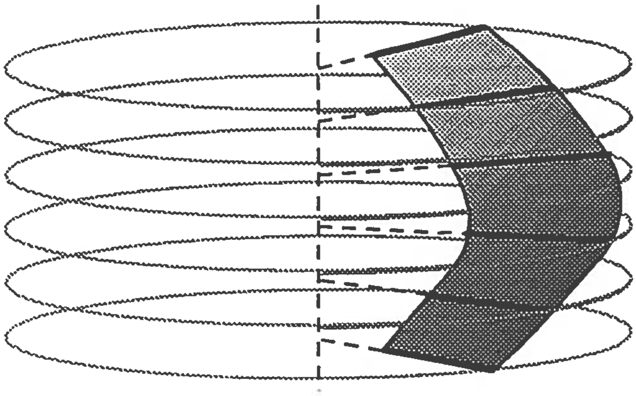


Figure 1. Sill orientation (no scale)

The sills are designated in Table 2 and depicted in Figure 2.

Table 2. Sills

Sill	Elevation (feet)	Station (feet)
32	5032	10+70
29	5029	12+02
26	5026	13+33
23	5023	14+65
20	5020	15+96
17	5017	17+28

The streamline length along the left sideslope is 67 percent greater than the streamline length along the right sideslope. Flow along the left sideslope follows a 0.0182 slope; flow along the right sideslope follows a 0.0304 slope.

The spillway is carved into the reservoir north abutment with an approximate 15-foot sidewall cut. The terrain bordering the right bank generally slopes away from the channel. No right bank exists downstream of Sill 17. Upstream of Sill 20, the terrain bordering the left bank slopes toward the channel. Downstream of Sill 20, the terrain bordering the left bank slopes away from the channel. No left bank exists 200 feet downstream of Sill 17.

As the spillway has no discharge at the 100-year event, the spillway bed is the proposed site of Eagle Ranch Road west of its intersection with Coors Blvd. The road alignment follows the spillway bed up to Sill 26 and then cuts through the spillway left sideslope and proceeds to the north.

The breach of the left sideslope by the road poses three potential hydraulic difficulties:

1) Possible egress of flood flow through the breach and into adjacent development.
2) Wave action in the spillway.

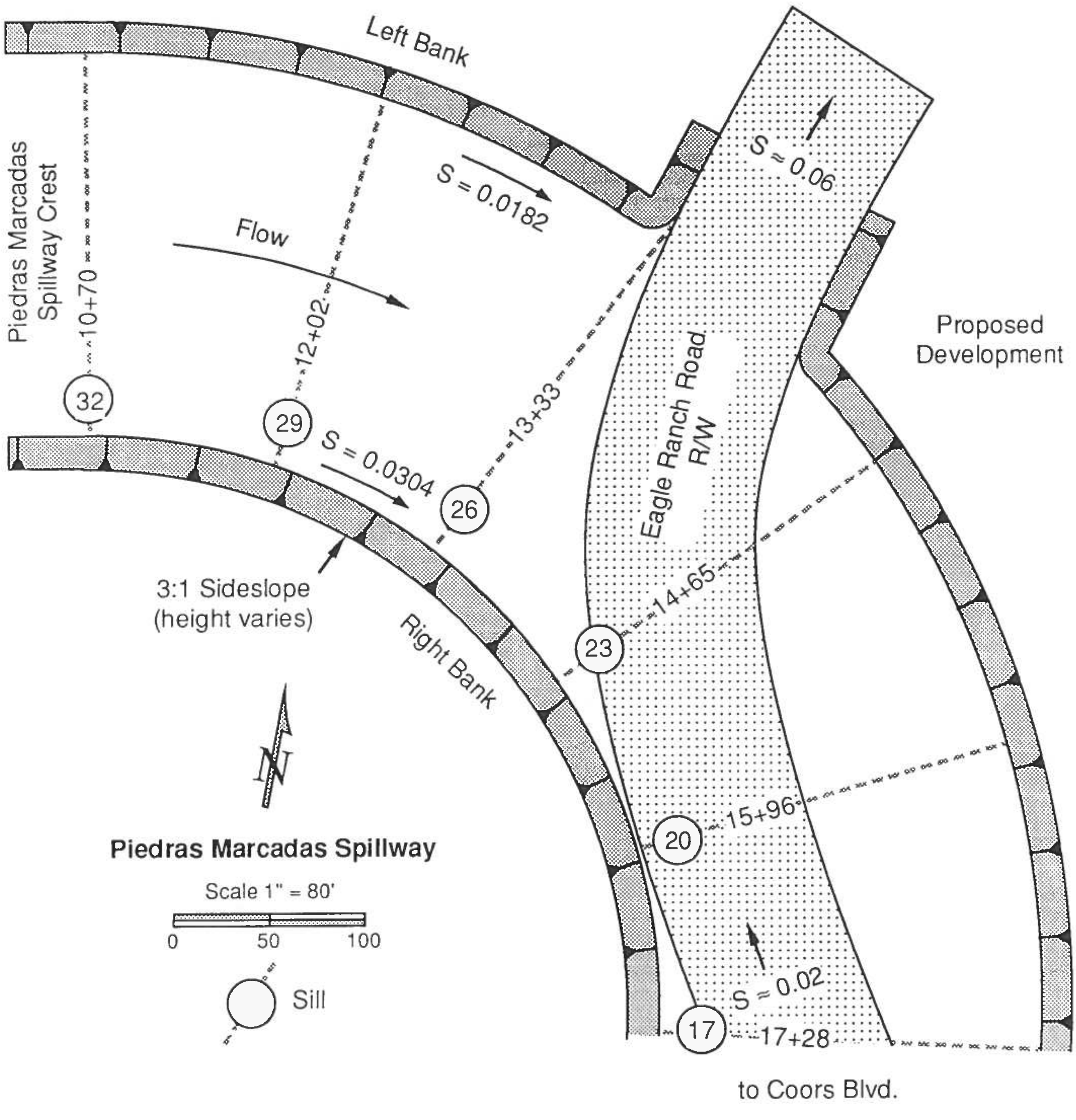


Figure 2. Layout

3) Possible redirection of flow against the downstream toe of the dam.

Hydraulic Study of Eagle Ranch Road in the Emergency Spillway of the Piedras Marcadas Detention Dam, Wilson & Co., 1987, provides preliminary HEC-2 analysis of the spillway/road. As HEC-2 cannot simulate two-dimensional flow, Wilson performed a 1:120 model study for visual appraisal. As flow depth at 1:120 is approximately 0.5 inch, the 1:120 model was too small for design verification. Wilson proposed a 1:30 scale for subsequent study.

Problem Statement

Isaacson & Arfman, P.A., on behalf of land development along the left spillway bank and northeast of the proposed breach, engaged UNM to model the spillway to:

1) Determine the hydraulic profile, with particular regard to superelevation.
2) Evaluate the hydraulic consequences of the breach.
3) Explore design modification as indicated.

The model study should address the following concerns:

1) Isaacson & Arfman's site grading.
2) The City of Albuquerque's establishment of road grade and alignment.
3) AMAFCA's and the State Engineer's satisfaction that:
   a) Flow will be conveyed within the designated path,
   b) Spillway function is not jeopardized, and
   c) Safety of the embankment is not compromised.

The Model

The study employed the UNM Civil Engineering/AMAFCA Open Channel Laboratory. The 50 foot articulated table served as a platform for a three-dimensional spillway model constructed of wood and sheet metal. The model extended from the Sill 32 to Sill 17.

Discharge measurement was by a pressure differential Annubar Flow Sensor, model GCR25 in the model water supply.

Geometric similitude was 1:48, scaling the 200-foot prototype bed to a model bed width of just over 4 feet. The curved nature alignment makes a model at Wilson's 1:30 proposal wider than the laboratory.

The inertial-gravity force ratio was the same for both model and prototype, making Froude numbers equal, and thus wave action geometrically proportional.

Experiments

Four alternative designs were modeled. Alternative 1 is the as-built condition with no breach. Superelevation in a trapezoidal channel in a simple circular curve has an analytic solution, subsequently discussed.

Alternative 2 is the current road plan:

0.0256 slope at grade up the lower spillway.
A slight slope decrease as the road crosses the channel bed.
0.06 slope through the breach.

Alternative 3 elevates the roadbed to deflect flow. The road slopes at approximately 0.04 from spillway center, forming a triangular sill approximately 3 feet high where the road encounters the sideslope.

Alternative 4 is the current road plan plus a deflector approximately 14 feet high on the upstream side of the breach. The deflector extends above the sideslope to deflect flow away from the breach. The deflector could be a vertical reinforced concrete wall or could be a roller compacted concrete berm. The deflector could lie along the edge of the road right-of-way as a structural retaining wall, or could lie roughly 40 feet upstream, notched into the sideslope. The hydraulics principally depend on the projecting area perpendicular to the flow, the same in all cases.

Fig. 3 summarizes the alternatives.

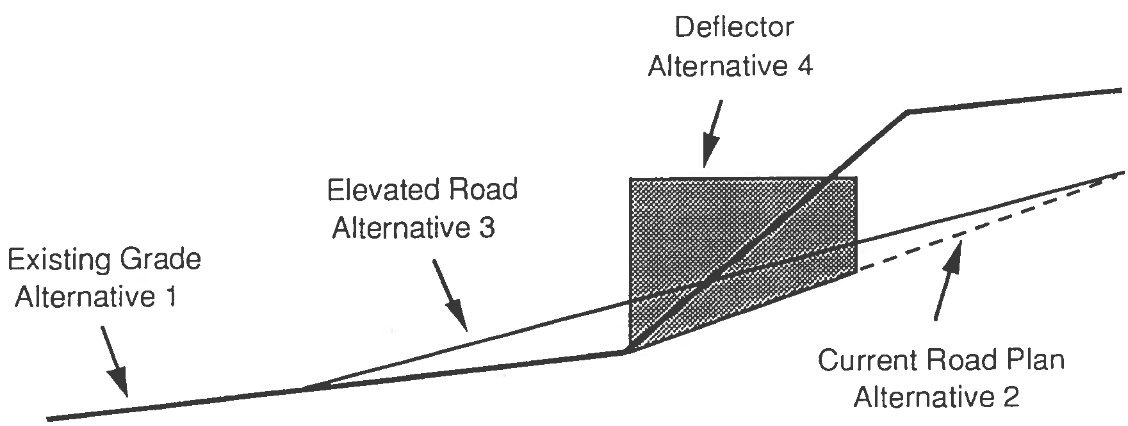


Figure 3. Alternatives (no scale)

Flow depths were measured along the left bank at stations listed in Table 3. “Bank Top Elevation” refers to the top of the 3:1 sideslope.

Table 3. Measurement Locations

Station	Bed Elevation (feet)	Bank Top Elevation (feet)	Remarks
10+70	5032.0	5039.0	Sill 32
11+36	5030.5	5038.0	
12+02	5029.0	5038.0	Sill 29
12+72	5027.5	5038.0	
13+33	5026.0	5038.0	Sill 26, Upstream corner of breach
13+99	5024.5	5038.0	Downstream corner of breach
14+65	5023.0	5038.0	Sill 23
15+31	5021.5	5038.0	
15+96	5020.0	5038.0	Sill 20
16+62	5018.5	5032.0	
17+28	5017.0	5029.0	Sill 17

All alternatives were modeled at the PMF (26,500 cfs). As subsequently noted, Alternative 2 emerged as this report's recommendation. Alternative 2 was tested at 0.25 and 0.5 PMF discharge (6,625 and 13,250 cfs) to check for wave action at intermediate discharges.

As a future hydrology revision might call for an increase in reservoir capacity, Alternative 2 was tested for the PMF with a 1-foot crest sill (Sta. 10+70) extension elevation to 5033.

Results

Table 4 indicates flow elevations for the full PMF.

Table 4. Left Bank Water Surface Elevation (feet)

	Elevation (ft) Alternative				Depth (ft) Alternative			
Station	1	2	3	4	1	2	3	4
10+70	5039.9	5039.9	5039.9	5039.9	7.9	7.9	7.9	7.9
11+36	5038.5	5038.2	5038.5	5038.2	8.0	7.7	8.0	7.7
12+02	5037.4	5037.4	5037.4	5037.1	8.4	8.4	8.4	8.1
12+67	5037.1	5037.1	5037.1	5037.1	9.6	9.6	9.6	9.6
13+33	5036.1	5036.1	5036.7	5038.0	10.1	10.1	10.7	12.0
13+99	5034.8	5035.5	5038.6	5032.9	10.3	11.0	14.1	8.4
14+65	5036.1	5033.9	5031.0	5033.3	13.1	10.9	8.0	10.3
15+31	5033.9	5030.4	5029.5	5031.4	12.4	8.9	8.0	9.9
15+96	5033.9	5031.4	5031.0	5030.7	13.9	11.4	11.0	10.7
16+62	5030.7	5030.0	5030.0	5029.7	12.2	11.5	11.5	11.2
17+28	5029.0	5028.4	5028.7	5028.4	12.0	11.4	11.7	11.4

Fig. 4 summarizes the left bank profile.

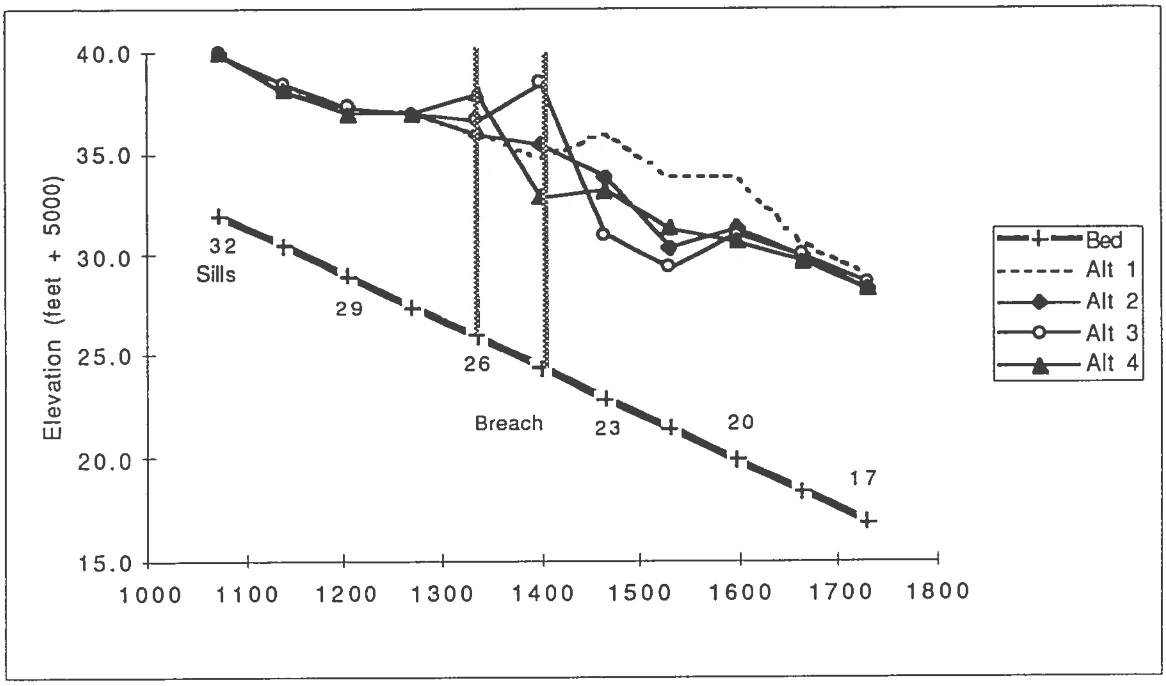


Figure 4. Left Bank Profiles

Table 5 indicates flow depths at 0.25 and 0.50 PMF discharges, Alternative 2. No unusual waves or jumps occur.

Table 5. Left Bank Water Surface Depth (feet)

Station	Discharge 0.25 PMF	0.50 PMF
10+70	3.5	5.2
11+36	2.2	4.0
12+02	2.2	4.0
12+67	2.5	4.2
13+33	2.5	4.2
13+99	2.7	4.8
14+65	2.7	5.0
15+31	2.5	4.5
15+96	2.7	5.2
16+62	2.7	5.2
17+28	2.7	5.2

The extended crest sill necessarily raises all flow elevations at Sta. 10+70 by 1 foot. By Sta. 11+36, however, the crest sill extension has no effect on flow depth.

Discussion

Design Standards

PMF hydraulics should not be envisioned in terms of conventional floodplain administration. A standard 100-year design (a 2.2 inch 24-hour storm) must satisfy freeboard, scour and setback criteria established to reflect acceptable risk. Such conservative design for a PMF (20 inch PMP) is not mandated. The PMF must not cause catastrophic dam failure in high hazard areas, but the overflow through the emergency spillway is likely to destroy downstream conveyance structures and crossings and endanger life.

Radius of Curvature

The 400-foot r_c was dictated by the terrain and available right-of-way. This r_c is low by conventional norms. The standard minimum r_c/b ratio of 3.0 for subcritical flow yields a 600-foot minimum radius of curvature r_{min} (*Hydraulic Design of Flood Control Channels*, EM 1601, U.S. Army Corps of Engineers, 1991). For supercritical flow without spiral transitions,

$$r_{\min} = \frac{4V^2W}{gy} \tag{1}$$

where V is velocity, W is the top width, g is gravity and y is depth (EM 1601). Using normal depth, r_{min} is 3120 feet.

The spillway turns more sharply than that specified by either standard. Superelevated flow, standing waves and helicoidal flow are anticipated consequences.

Water Surface Elevations

Alternative 1, as built, contains the PMF. The spillway functions properly.

Impinging Wave

The downstream corner of the breach (Sta. 13+99 between Sills 26 and 23) is a critical location for the proposed development. Fig. 5 illustrates the susceptibility of this location to an impinging wave. The breach sideslope must be sufficiently elevated to prevent spill into the development. Alternative 2 indicates that the breach sideslope should be at least at elevation 5035.3 to hold the impinging wave.

Reflected Wave

Fig. 5 illustrates a possible reflected oblique wave initiated at the breach. Reflected waves are common in nonlinear concrete supercritical channels. Should such a wave break through the low right bank near Sill 20, flow might begin to erode the dam embankment. Experimentally, the impinging wave reflects back into the channel as a distinct and significant wave. The reflected wave rapidly dissipates, however. There is no evidence of the reflection on the opposite bank. The loss of waveform is likely due to the 200-foot channel width plus the bed roughness.

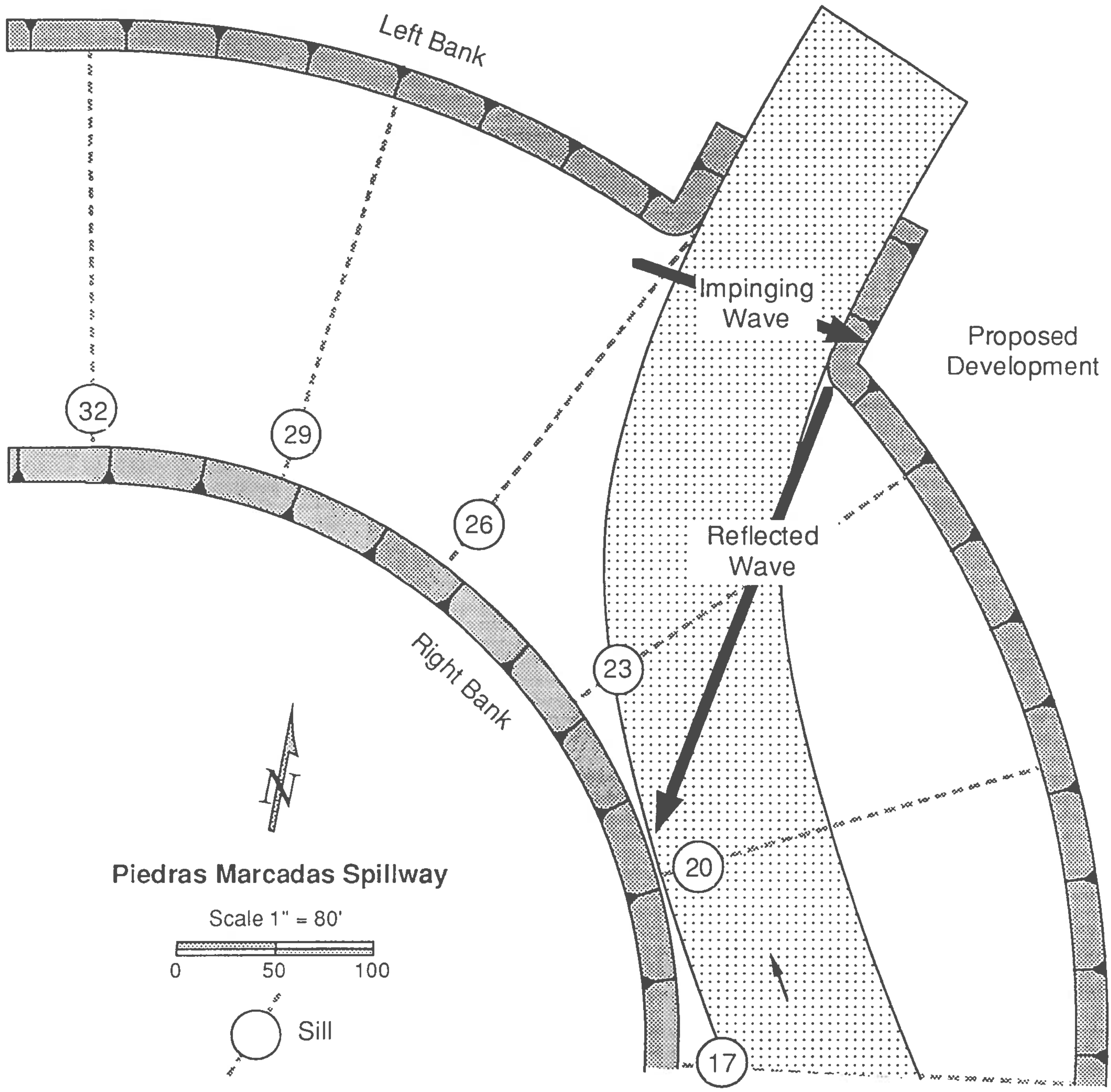


Figure 5. Wave Action

Superelevated Flow

Both experimentally and analytically, superelevated flow is a dominating freeboard factor on the left bank. Superelevated flow essentially eliminates the possibility of flow escaping over the right bank.

Wilson estimates superelevated flow to be “4 feet above normal depth”. EM 1601 estimates superelevation,

$$\Delta y = \frac{CV^2W}{gr_c} \tag{2}$$

where Δy is the rise in water surface between the theoretical water surface at the centerline and the outside water surface and C is 1.0 for supercritical simple circular trapezoids. At normal depth, Δy is 4.83 feet. Added to the centerline depth, total depth on the left bank is 10.01 feet. On the right bank, depth is 0.35 feet.

Wilson left the channel bed horizontal, possibly because a superelevated channel bed (sloping up to the east) would:

(1) Negatively superelevate the road's bend and be unacceptable from the traffic standpoint, and/or

(2) Concentrate flows less than the PMF along the right bank.

Alternatives 3 and 4 in this report are experimental “patches”, should a countermeasure to superelevated flow be mandated. The Appendix of this report discusses superelevation in a more fundamental manner, alternatives not necessarily appropriate for the Piedras Marcadas, but options that might be considered in similar situations.

The Roadbed as a Remedy for Superelevation (Alternative 3)

Alternative 3 is counterproductive. While the elevated roadway may turn some flow to the right, the elevated roadbed also deflects water higher into the breach. The impinging wave at Sta 13+99 is approximately 3 feet higher than at Alternative 2.

A Deflector as a Remedy for Superelevation (Alternative 4)

Alternative 4 lowers the impinging wave at Sta. 13+99, but backs flow at Sta. 13+33. While the deflector initiates a reflected wave somewhat more pronounced than the reflected wave of Alternative 2, there is no reflected wave on the opposite bank. From the perspective of freeboard, a deflector is advantageous.

From the perspective of regulatory flood protection, a deflector is not justified. Development in the region requires 100-year event protection, for which the Piedras Marcadas emergency spillway will be dry.

Were a vertical wall needed for other purposes, e.g., as a retaining wall for the breach, it might be reasonable to reinforce and/or extend it for flow deflection. As a wall is not otherwise needed, it seems unwarranted now.

Intermediate Discharges

At the lesser flows, superelevation is relatively minor with no indication of hydraulic problems.

Crest Sill

Other than the 1-foot increase in the crest water surface itself and a short S2 adjustment, the crest sill extension has no significant effect on flow downstream. By the next gaged station (11+36), the hydraulic significance of bed slope, roughness and the channel bend masks the influence of the upstream boundary condition.

Conclusions

1) As suggested in the literature, standard estimates of superelevation underestimate actual depths.

2) The spillway functions properly in the as-built condition.

3) The spillway functions under the current road plan. Wave action is most severe where an impinging wave rides up on the downstream corner of the breach.

4) Elevating the road to deflect flow enhances wave action on the bank.

5) A deflector reduces wave action on the bank by turning flow away from the breach, but is not required by flood protection standards.

6) The proposed development should be protected from flow at elevation 5035.5 in the breach.

7) The oblique wave reflecting from the breach does not persist to the opposite bank. The breach does not jeopardize the reservoir embankment.

7) Flows at less than the PMF do not exhibit hydraulic behaviors meriting special concern.

8) The crest sill may be elevated at least 1 foot without significant downstream effect.

Appendix

Notes:

1. Multiply model distance by 48 to get prototype distance.
2. Multiply model velocity by 6.93 to get prototype velocity.
3. Multiply model discharge by 15960 to get prototype discharge.

Superelevation

Eqs. 1 and 2 presume that some flow remains against the inner bank. Modeling indicates the possibility of superelevated flow leaving the inside of the bed dry, as illustrated in Fig. 6.

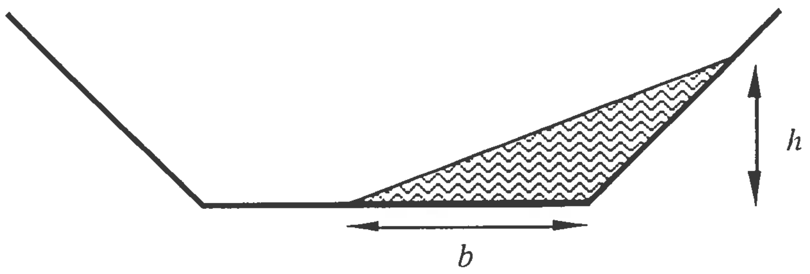


Figure 6. Severe Superelevated Flow (no scale).

Eq. 3 estimates superelevation in the case of Fig. 6.

$$h = \frac{3r_c}{2Z}\left[\sqrt{1 + \frac{8Zb}{3r_c^2}\frac{V^2}{2g}} - 1\right] \tag{3}$$

where h is the total superelevated depth, b is the width of the wetted bed, and Z is not 0. Taking b as the full bed width, h is 9.5 feet. For the same reasons that Eq. 2 underestimates Δy, Eq. 3 underestimates h.

The modeled bed was roughened to maintain some flow on the inside bank, in keeping with the Wilson analysis. From the performance standpoint, it makes little difference whether the bed is fully covered, as channel adequacy is determined by h on the outside bank.

The model indicates that superelevation without breach complications tends to reach a depth of approximately 13.1 feet, roughly 30 percent greater than the values predicted by equations. French notes that the assumption of average velocity causes Δy to be underestimated by as much as 50 percent *(Open Channel Hydraulics*, 1985, p. 278). An additional cause for Δy underestimation is the propensity of waves to run up inclined sideslopes.

The bed could be banked as illustrated in Fig. 7 to maintain a constant depth across the cross-section.

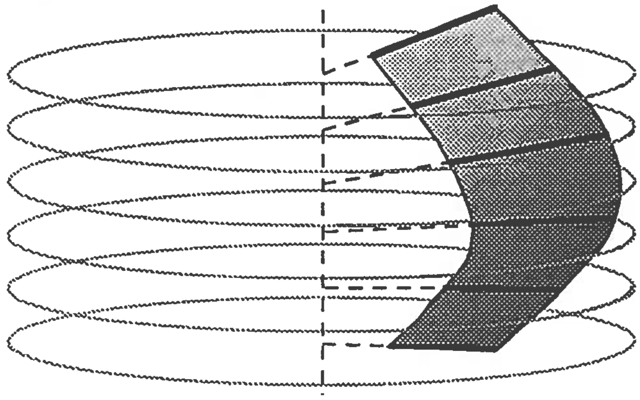


Figure 7. Banked Spillway (no scale)

where the cross slope S_t, (Chow, *Open Channel Hydraulics*, 1959, p. 457) is,

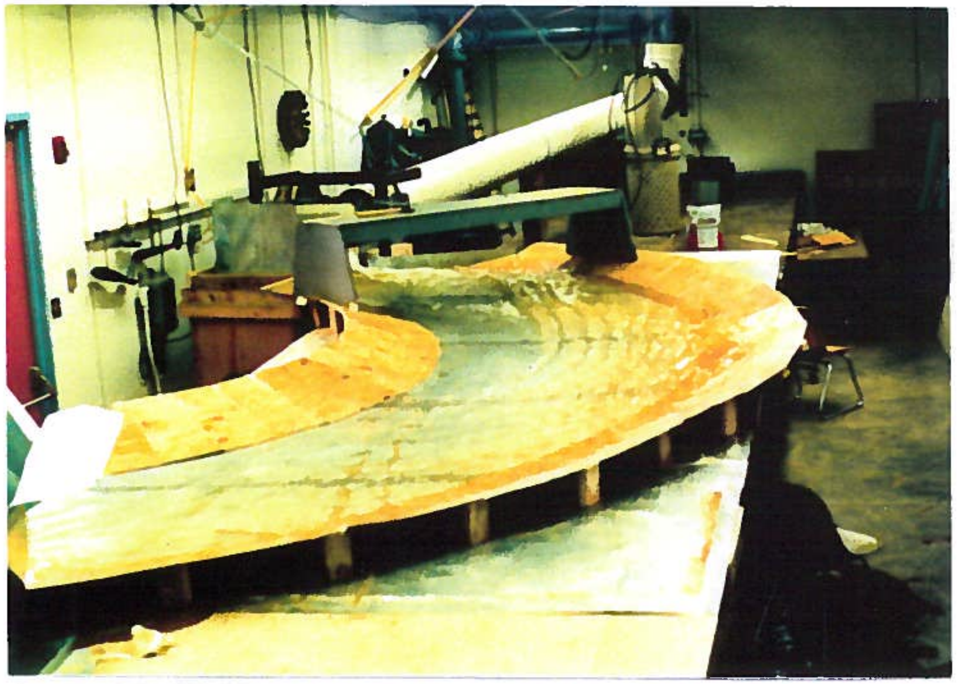
$$S_t = \frac{V^2}{g r_c} \tag{4}$$

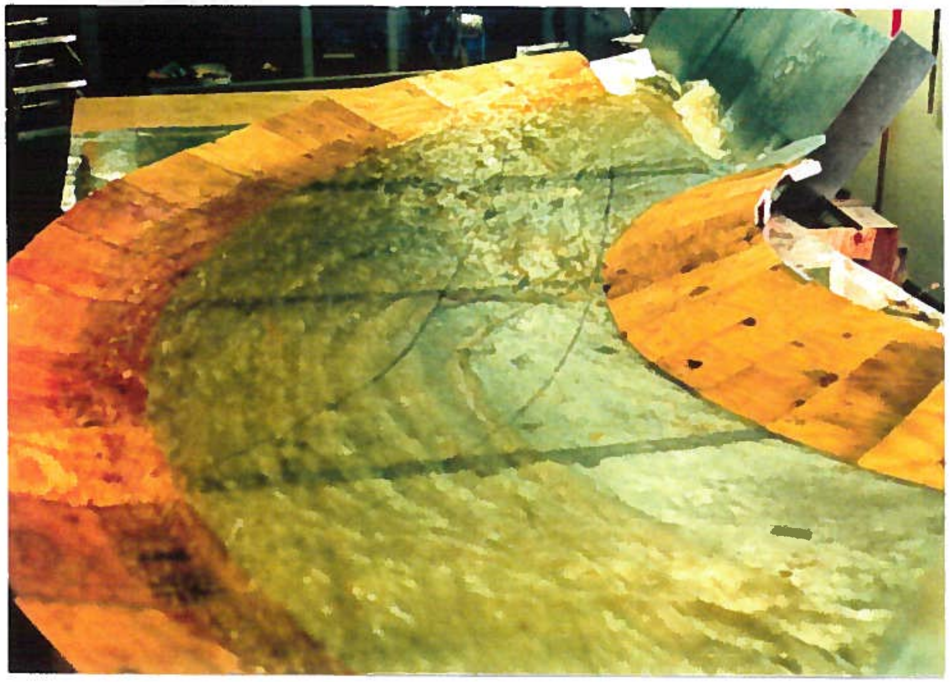
To maintain normal depth, S_t is 0.044. Horizontal sills are already in place. Such banking would require significant concrete and earth work.

A curved vane, approximately 8 feet high and 600 feet long down the center of the spillway would reduce Δy on the left sideslope. Such a vane must both structurally and geotechnically withstand full hydrostatic pressure on one side. The vane must be breached for the road and thus is infeasible.

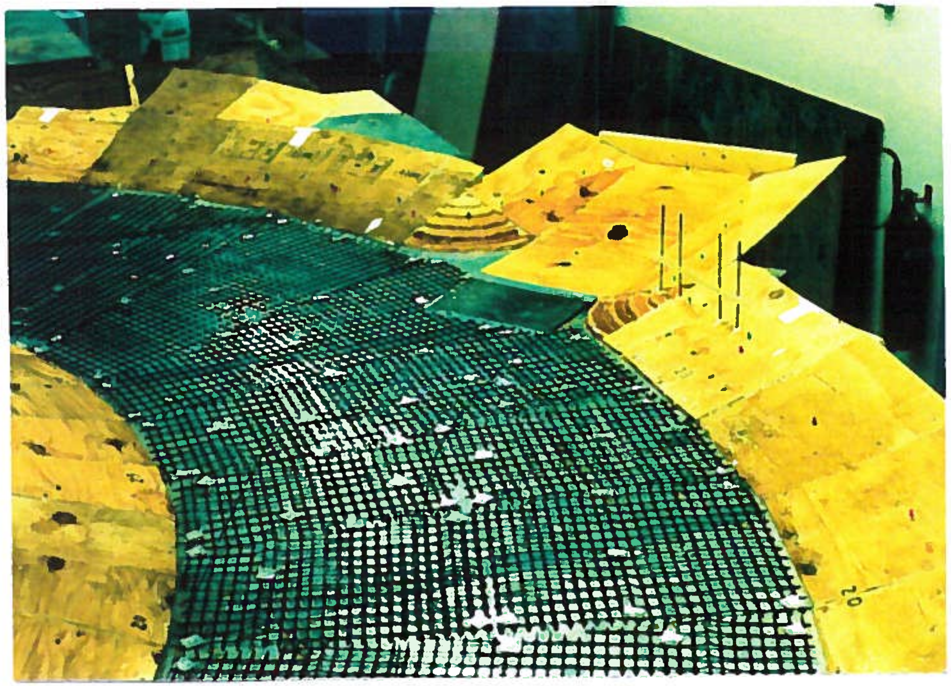
Diagonal sills protruding above the bed could help turn the flow. Wilson included one such sill, 2 feet x 2 feet above grade (plus 7 feet below grade), to turn some flow away from the breach. One modeled sill has no discernible effect. Were sills larger and/or more in number, they might reduce superelevation. Modeling suggests that a multiple-sill scheme might require in the order of 30 diagonal sills to have significant consequence on superelevation. Given the need to anchor each sill in the erodable channel bed, the concrete required for adequate sills is in the order of magnitude of the concrete required to entirely line the spillway. Diagonal sills thus do not appear to be economically feasible and were not further modeled.

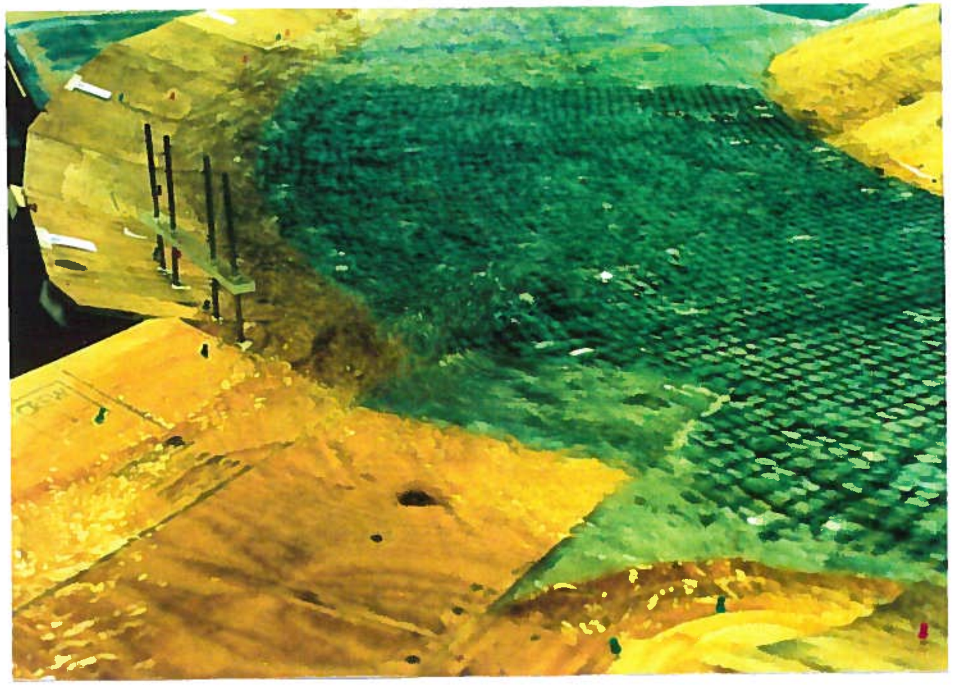
Changing the outer bank from a 3:1 earthen sideslope or 2:1 roller compacted concrete to a vertical wall would reduce some of the superelevation runup, but would not reduce the fundamental Δy. This alternative is cost ineffective.



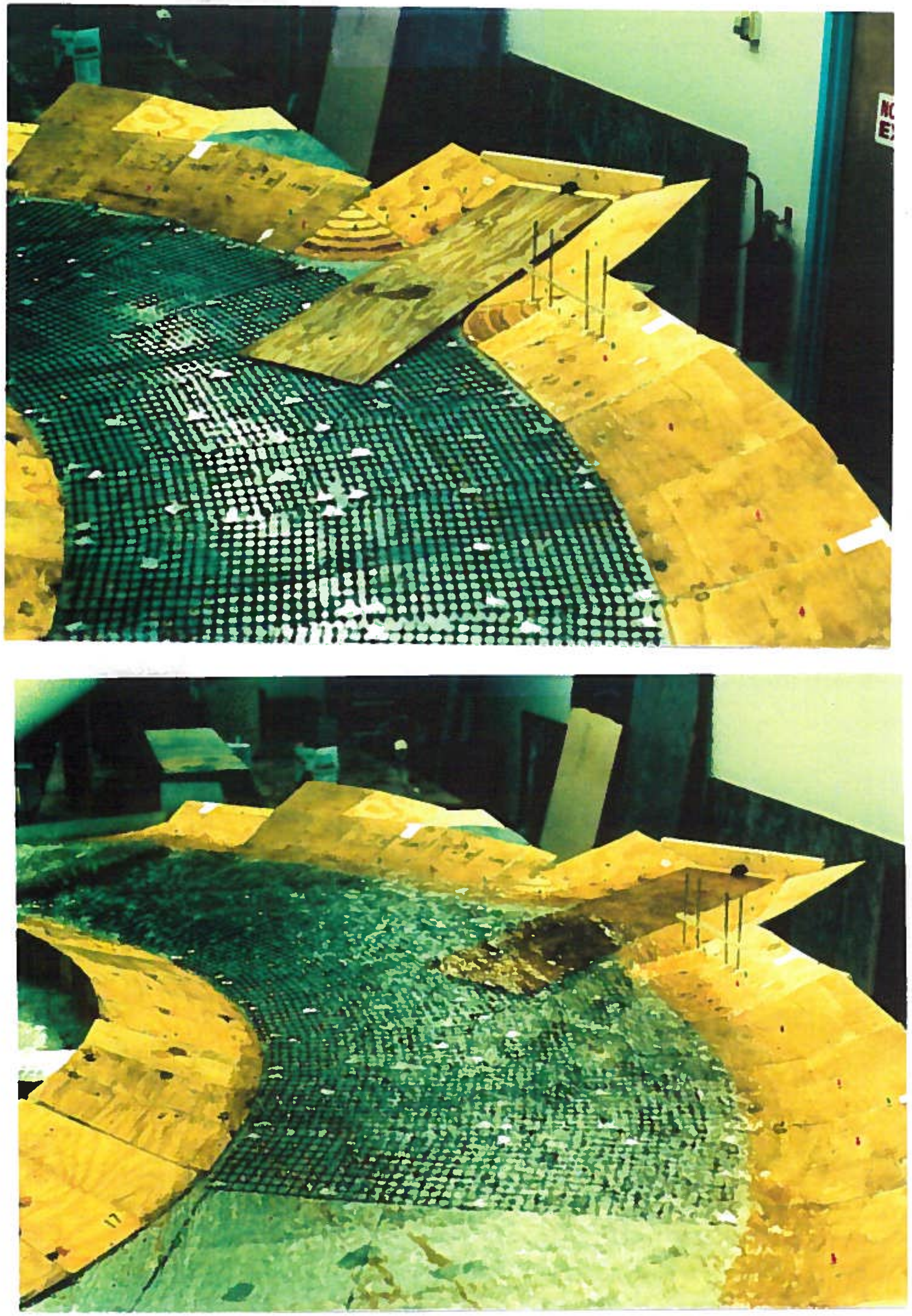


Alternative 1 prior to bed roughening

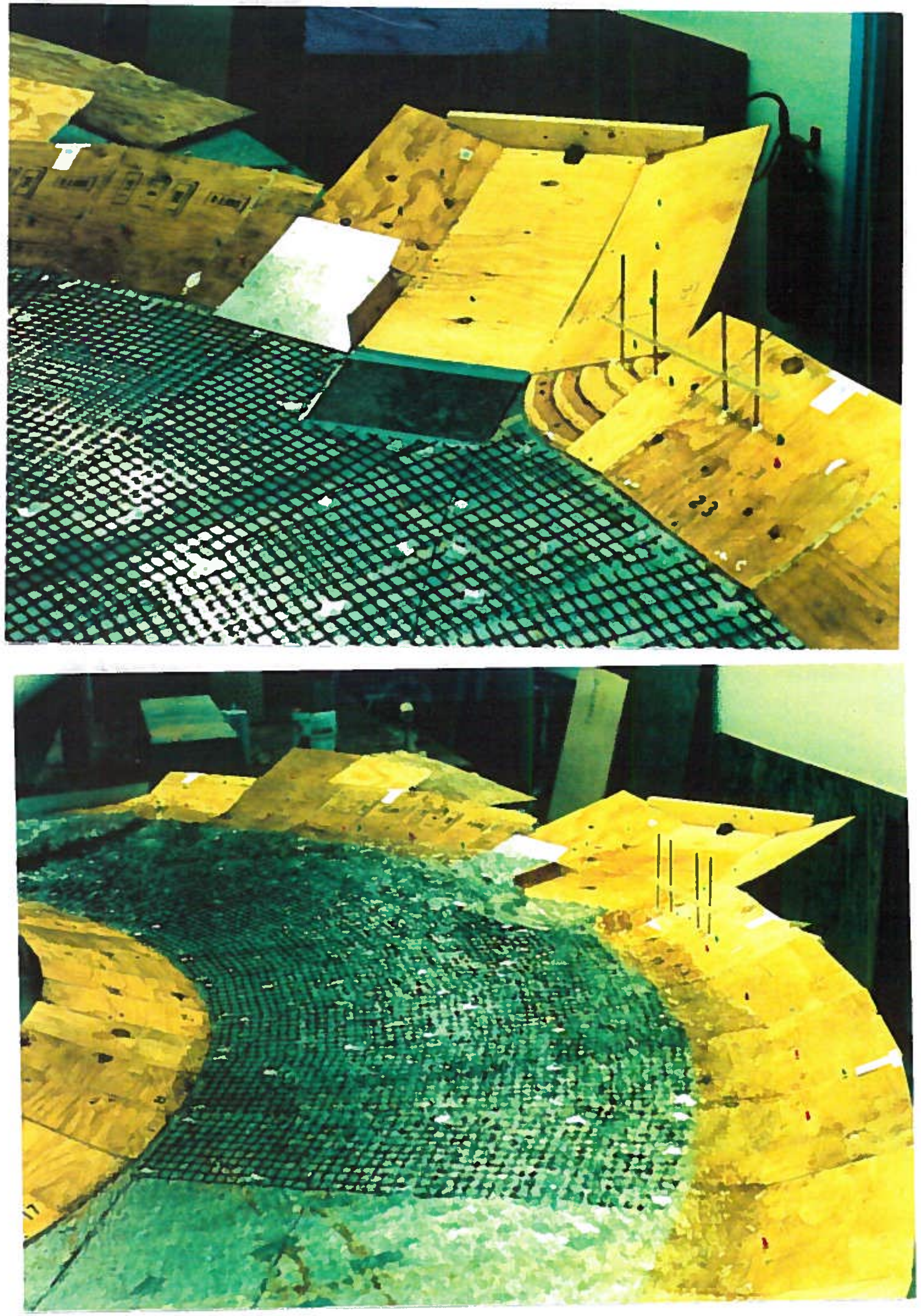




Alternative 2



Alternative 3



Alternative 4