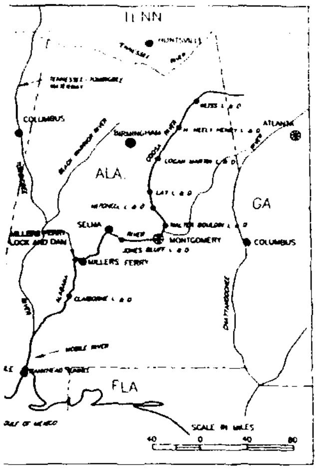
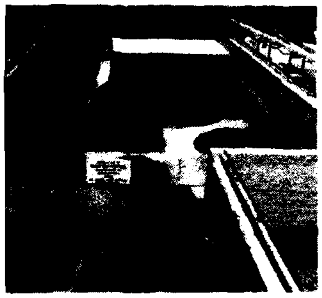
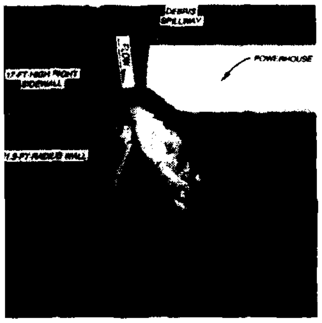






HYDRAULICS LABORATORY

(2)

TECHNICAL REPORT HL-92-16

DEBRIS SPILLWAY AND CHUTE FOR MILLERS FERRY POWERHOUSE ALABAMA RIVER, ALABAMA

Hydraulic Model Investigation

by

Deborah R. Cooper

Hydraulics Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199

October 1992

Final Report

Prepared for US Army Engineer District, Mobile
Mobile, Alabama 36628-0001

REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

1. AGENCY USE ONLY *(Leave blank)*

2. REPORT DATE
October 1992

3. REPORT TYPE AND DATES COVERED
Final report

4. TITLE AND SUBTITLE
Debris Spillway and Chute for Millers Ferry Powerhouse, Alabama River, Alabama; Hydraulic Model Investigation

5. FUNDING NUMBERS

6. AUTHOR(S)
Deborah R. Cooper

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)
USAE Waterways Experiment Station, Hydraulics Laboratory, 3909 Halls Ferry Road, Vicksburg, MS 39180-6199

8. PERFORMING ORGANIZATION REPORT NUMBER
Technical Report HL-92-16

9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)
USAE District, Mobile, PO Box 2288, Mobile, AL 36628-0001

10. SPONSORING/MONITORING AGENCY REPORT NUMBER

11. SUPPLEMENTARY NOTES
Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

12a. DISTRIBUTION/AVAILABILITY STATEMENT
Approved for public release; distribution is unlimited.

12b. DISTRIBUTION CODE

13. ABSTRACT *(Maximum 200 words)*

A 1:25-scale model of the Millers Ferry Lock and Dam, Alabama River, Alabama, reproduced a 24-ft-wide by 170-ft-long ogee spillway and existing powerhouse, 200 ft of the approach immediately upstream of the spillway crest, and 600 ft of exit channel. The hydraulic design of the debris spillway was evaluated. Model tests consisted of observing flow conditions throughout the structure, determining how effectively debris was being passed through the structure, and determining if scour of the riprap protection downstream occurred. A horizontal apron terminated by a curved right sidewall diverted flow into the deeper tailwater area and provided needed energy dissipation for passage of debris through the chute spillway. It was determined that the debris spillway would operate efficiently without causing unacceptable flow conditions and scour downstream from the spillway chute with tailwater el 32.0-50.0. Operation of the debris spillway would have to be suspended at tailwaters greater than el 50.0, and waterlogged debris and debris larger than 50 ft would have to be physically removed.

14. SUBJECT TERMS
Chute spillway
Debris removal
Debris spillway
Millers Ferry
Powerhouse

15. NUMBER OF PAGES
57

16. PRICE CODE

17. SECURITY CLASSIFICATION OF REPORT	18. SECURITY CLASSIFICATION OF THIS PAGE	19. SECURITY CLASSIFICATION OF ABSTRACT	20. LIMITATION OF ABSTRACT
UNCLASSIFIED	UNCLASSIFIED		

PREFACE

The model investigation reported herein was authorized by the Headquarters, US Army Corps of Engineers (HQUSACE), on 3 August 1989 at the request of the US Army Engineer District, Mobile.

The studies were conducted in the Hydraulics Laboratory (HL) of the US Army Engineer Waterways Experiment Station (WES) during the period August 1989 to April 1992 under the direction of Messrs. F. A. Herrmann, Jr., Director, HL; R. A. Sager, Assistant Director, HL; and G. A. Pickering, Chief, Hydraulic Structures Division (HSD), HL. The tests were conducted by Mrs. D. R. Cooper and Messrs. W. B. Fenwick, E. L. Jefferson, and R. Bryant, Jr., of the Spillways and Channels Branch, HSD, under the direct supervision of Mr. N. R. Oswalt, Chief of the Spillways and Channels Branch. This report was prepared by Mrs. Cooper.

During the course of the investigation Messrs. M. Thompson, B. Felder, J. Stuckey, J. Couey, W. Odom, D. Otto, C. Snow, P. Gagliano, D. Sessions, J. Shelley, E. Harris, L. Do, and L. Harper, Mobile District; and S. Powell and T. Munsey, HQUSACE, visited WES to discuss test results and correlate test results with current design studies.

Messrs. M. Bolden and J. Lyons, Engineering and Construction Services Division, WES, constructed the model. Mrs. M. C. Gay, Information Technology Laboratory, WES, edited this report.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Leonard G. Hassell, EN.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
acres	4,046.873	square metres
acre-feet	1,233.489	cubic metres
cubic feet	0.02831685	cubic metres
degrees (angular)	0.01745329	radians
feet	0.3048	metres
inches	2.54	centimetres
miles (US statute)	1.609347	kilometres

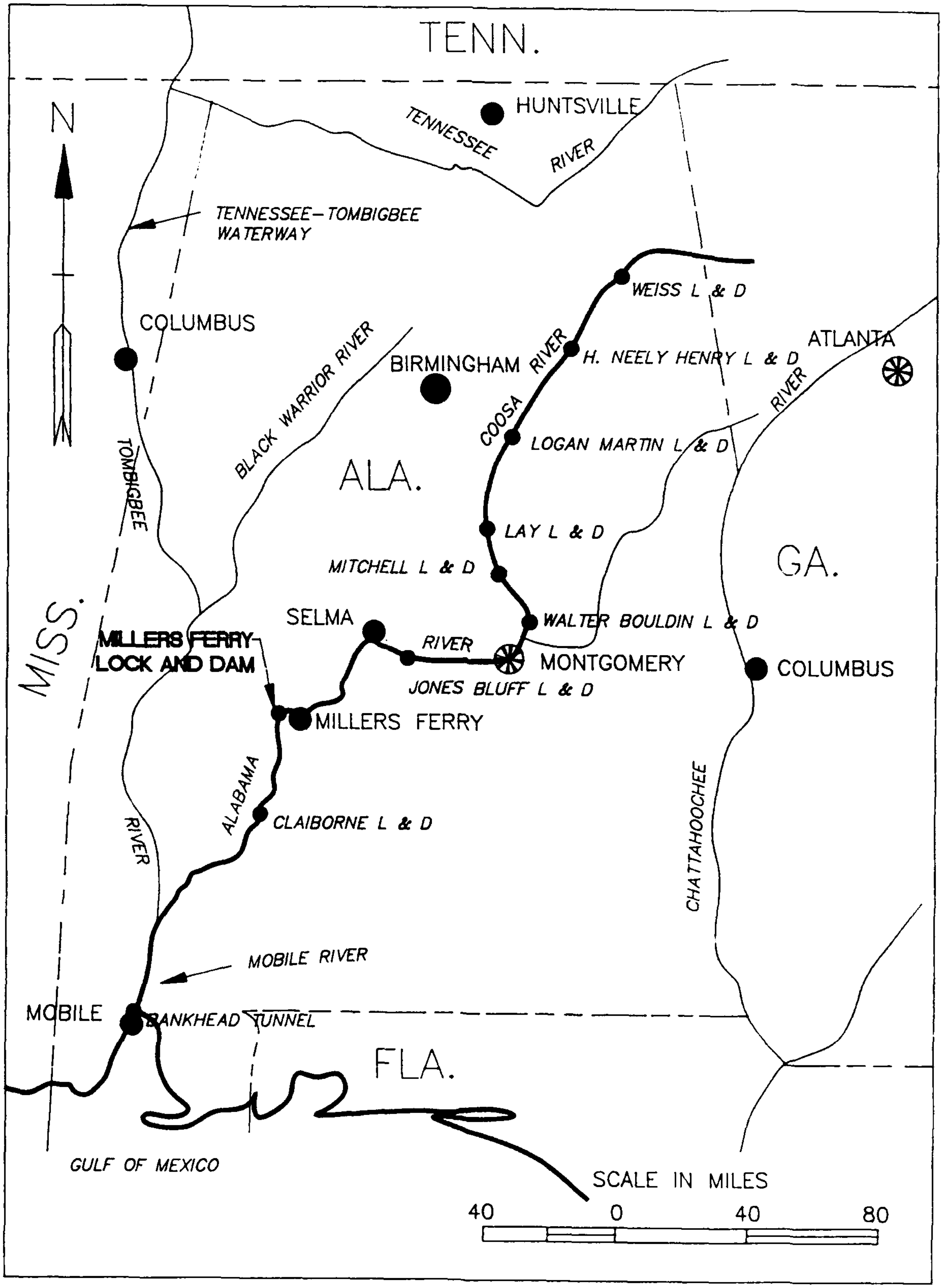


Figure 1. Vicinity map

DEBRIS SPILLWAY AND CHUTE FOR MILLERS FERRY POWERHOUSE

ALABAMA RIVER, ALABAMA

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. Millers Ferry Lock and Dam is located on the Alabama River near Millers Ferry, AL (Figure 1). It was authorized by the River and Harbor Act of 2 March 1945. The existing project consists of an earth dike on the right bank, a gated spillway with seventeen 50-ft*-wide gates in the river channel, a lock at navigation mile 178.0 above the Bankhead Tunnel, Mobile, AL, lock mound on the left bank, an earth dike extending downstream paralleling the lock to the powerhouse intake structure, a powerhouse, and an earth dike extending to high ground on the left bank (Plate 1). The powerplant contains three 25,000-kw units. The lock has chamber dimensions of 84 by 600 ft and a depth of 13.0 ft over the miter sills. The 103-mile-long lake created by this dam, the William "Bill" Dannelly Lake, has an area of 17,200 acres, a capacity of 331,800 acre-ft at normal pool elevation 80.0**, and a 9- by 200-ft navigation channel extending its entire length.

2. Floating debris has been a continual problem at the Millers Ferry Powerhouse since the project was completed 20 years ago. Various debris removal schemes have been tried through the years, such as a powerhouse debris removal crane, upstream trash fence, snagboat dipping, professional divers, push boats, and a floating catch pen. A spillway located adjacent to the existing powerhouse was designed by the US Army Engineer District, Mobile, as the best alternative for removing debris from the forebay at Millers Ferry. Due to high costs associated with constructing a deep stilling basin through the embankment, a chute type spillway was proposed for prototype construction.

---

* A table of factors for converting non-SI to SI (metric) units of measurement is presented on page 3.

** All elevations (el) cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD).

Purpose and Scope of the Model Study

3. Because of the uncertainties reg··ding downstream energy dissipation, flow patterns, and unusual exit geometries, this model study was conducted to evaluate the hydraulic design of the debris spillway by measuring velocities and evaluating scour in the exit area. A spillway configuration minimizing excavation of the existing dam embankment was developed. The extent of scour and the need for protection downstream of the structure to adequately protect the project and minimize future maintenance requirements were of interest.

Presentation of Data

4. In the presentation of test results, no attempt was made to introduce the data in the chronological order in which the tests were conducted on the model. Instead, as each element of the structure is considered, all tests conducted thereon are discussed in detail. All model data are presented in terms of prototype equivalents. All tests are discussed in Part III.

PART II: THE MODEL

Description

5. The 1:25-scale model reproduced the proposed 24-ft-wide by 170-ft-long ogee spillway and existing powerhouse, 200 ft of the approach immediately upstream of the proposed spillway crest, and 600 ft of exit channel (Figure 2, Plates 2 and 3). The portions of the model representing the powerhouse approach, exit, and overbank areas were molded of sand to sheet metal templates and covered with a 24-in.-thick blanket of riprap. The spillway and spillway gate were constructed of sheet metal. The powerhouse, sidewalls, and chute apron were made of wood.

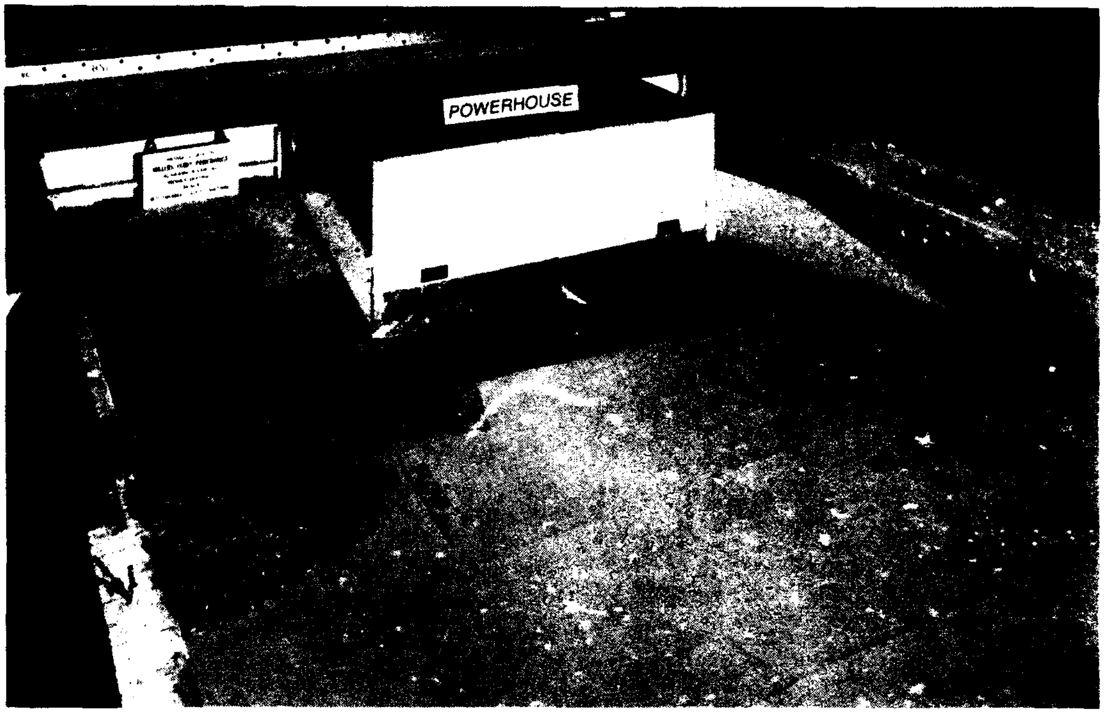
Appurtenances and Instrumentation

6. Water used in the operation of the model was supplied by pumps, and discharges were measured with orifice meters. The tailwater in the downstream end of the model was controlled by an adjustable tailgate. Steel rails set to grade provided reference planes. Water-surface elevations were obtained with staff gages.

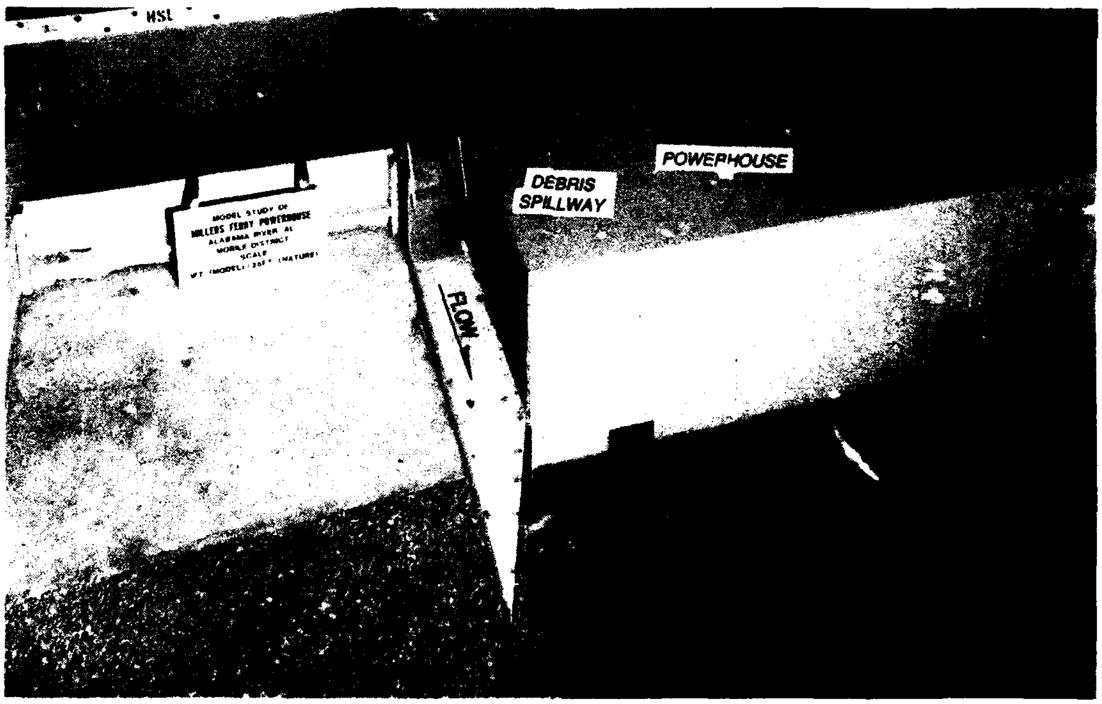
Scale Relations

7. The accepted equations of similitude, based upon the Froudian relations, were used to express the mathematical relations between the dimensions and hydraulic quantities of the model and the prototype. General relations for the transference of model data to prototype equivalents are presented in the following tabulation:

Dimension	Ratio	Scale Relation Model:Prototype
Length	$L_r = L$	1:25
Area	$A_r = L_r^2$	1:625
Velocity	$V_r = L_r^{1/2}$	1:5
Discharge	$Q_r = L_r^{5/2}$	1:3,125
Time	$T_r = L_r^{1/2}$	1:5
Weight	$W_r = L_r^3$	1:15,625
Force	$F_r = L_r^3$	1:15,625



a. Overall view looking upstream



b. Closeup view looking upstream

Figure 2. 1:25-scale model

PART III: TESTS AND RESULTS

8. As previously stated, the purpose of the model study was to evaluate the hydraulic design of the debris spillway. The function of the debris spillway is to remove floating debris from the powerhouse forebay as efficiently as possible without causing unacceptable flow conditions and scour downstream from the spillway chute in the powerhouse tailrace area. Model tests consisted of observing flow conditions throughout the structure, determining how effectively debris was being passed through the structure, and determining if scour of the 24-in.-thick riprap protection downstream occurred. All tests were conducted with a discharge of 2,640 cfs and the upper pool at el 80.8. For tests to determine riprap stability, the tailwater was gradually lowered until failure occurred or the tailwater reached a minimum el of 32.0.

Type 1 (Original) Design

9. The type 1 design debris spillway consisted of a 24-ft-wide ogee crest and chute adjacent to the powerhouse and a sidewall on the right of the spillway (Plate 3). No energy dissipator was provided at the end of the chute, and energy was dissipated by flow plunging into the tailwater. A tainter gate was located at the upstream end of the structure to cut flow off when passage of debris was not necessary.

Approach

10. With a discharge of 2,640 cfs, there was considerable contraction of flow at the left entrance to the spillway. Also, debris introduced into the model became snagged on the right upstream approach wall.

Chute

11. A water-surface profile through the structure with a discharge Q of 2,640 cfs is shown in Plate 4. Water-surface data are tabulated in Table 1.

Exit

12. The tailwater elevation was varied starting at el 66.0 and then gradually decreasing to el 32.0. Flow conditions in the exit area with various tailwater elevations are shown in Photos 1-4. Minimal damage to the tailrace riprap occurred when the tailwater was held above el 36.0. A large

ır hole formed in the tailrace riprap when the tailwater was lowered from 36.0 to el 32.0 (Photo 5). Flow exited the spillway at an angle to the ıt (looking downstream), causing increased turbulence and scour potential the right downstream embankment.

Type 2 Design

13. A sloping wall was added to the right upstream approach wall ate 5) to alleviate debris collecting on the wall. Also, a 10-ft-diam ›totype) semicircular deflector was attached to the face of the powerhouse decrease flow contraction at the left entrance to the spillway (Plate 5). s was designated the type 2 design. Debris passage was improved and less traction of flow was observed. However, exit flows continued to cause ır on the right downstream embankment.

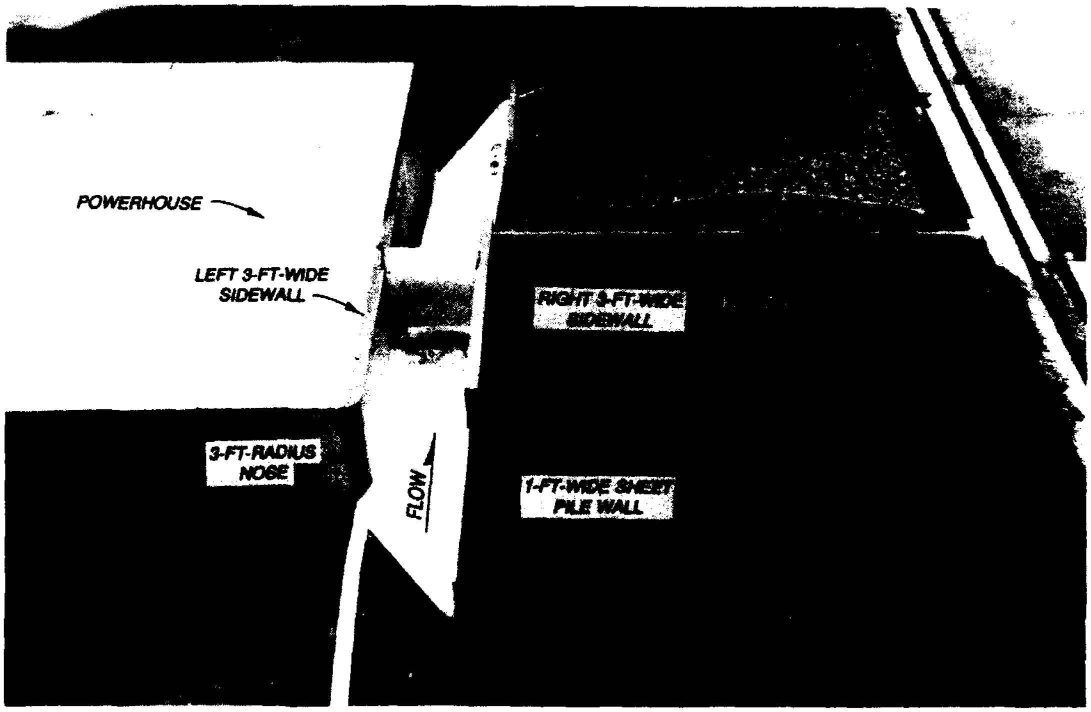
Type 3 Design

14. A 3-ft-wide wall with a 3-ft-radius upstream nose was added along right upstream face of the powerhouse to improve entrance flow into the te, the trash chute geometry was modified, a 3-ft-wide wall was placed acent to the powerhouse for structural stability, and the tailrace slope changed to 0.195 ft/ft (Figure 3, Plates 6 and 7). A 67.3-ft-long zontal apron at el 40.0 was provided to increase energy dissipation. A tant discharge of 2,640 cfs was introduced into the debris spillway until upper pool stabilized at el 80.8. Tailwater elevations were varied start- at el 66.0 then gradually decreasing to el 32.0 (Photos 6-9). Riprap dam- on the berm at el 47.5 (Photo 10) was caused by flow overtopping the right ewall when the tailwater ranged from el 49.0 to el 50.0 (Photo 7). No dam- to the tailrace riprap occurred when the tailwater was held above el 34.5. arge scour hole formed in the tailrace riprap when the tailwater was low- d from el 34.5 to el 32.0 and held at el 32.0 for 2.5 hr (Photo 11). Flow ed the spillway at an angle to the right (looking downstream), causing reased turbulence and scour potential on the right downstream embankment.

Alternate Designs

15. Several modifications to the energy dissipator (horizontal apron at

a. Overall view looking upstream



b. Closeup view looking downstream

Figure 3. Type 3 design

el 40.0) were tested to alleviate scour of the riprap in the tailrace. The type 4 and 5 designs consisted of two deflector blocks staggered 15 and 5 ft from the right sidewall on the horizontal apron (Plates 8 and 9, respectively). The type 4 design deflectors were 4 ft high by 4 ft long by 5 ft wide and were oriented so the sloping face was parallel to flow. The type 5 design deflectors were 5 ft high by 4 ft long by 4 ft wide and were oriented so that the sloping face was toward the existing left retaining wall. Each design was tested with pool el 80.8 and tailwater el 32.0 for 2.5 hr. Flow flipped over the deflectors and plunged into the riprap in the tailrace causing riprap failure. Observation of the flow trajectory over the deflectors indicated that insufficient flow in the jet was being spread in the deeper tailwater area. Subsequent tests concentrated on diverting the flow from the shallower tailwater to the deeper tailwater immediately downstream of the powerhouse.

16. A 10-ft-high wall with a 25-ft radius (type 6 design) was located at a point tangent to the right training wall 30 ft downstream of the powerhouse (Plate 10). This design was tested at tailwater el 32.0. Flow was forced upward about 35 ft above the chute invert. More of the flow was diverted in the deeper tailwater, but the riprap in the sloping area downstream of the powerhouse was displaced.

17. A 10-ft-high wall with a 50-ft radius (type 7 design) was located at a 130-deg angle to the right training wall 21.75 ft downstream of the powerhouse (Plate 11) and tested for 2.5 hr at tailwater el 32.0. More of the flow was diverted in the deeper tailwater, but the riprap in the sloping area downstream of the powerhouse was displaced.

18. The type 8 design (Plate 12) was the same as the type 7 design but with the wall moved 16 ft downstream of the powerhouse. The test was conducted for 2.5 hr at tailwater el 32.0. More of the flow was diverted in the deeper tailwater, but the riprap in the sloping area downstream of the powerhouse was displaced.

19. In the type 9 design (Plate 13), a straight 10-ft-high wall with 1-ft chamfers on the top and bottom of the wall was located at a 122-deg angle to the right training wall 11.75 ft downstream of the powerhouse. This design was tested for 2.5 hr at tailwater el 32.0. More of the flow was diverted in the deeper tailwater, and the riprap remained stable. Locating the wall so close to the powerhouse resulted in substantial splash against the downstream

corner of the powerhouse and limited clearance between the extension of the left training wall (downstream of the powerhouse) and the angled wall. Because the potential for damage to the powerhouse from large debris and/or jamming of the chute was increased, increasing the clearance from the powerhouse became necessary (type 10 design).

20. The type 10 design (Plate 14) was the same as the type 9 design but with the wall 27.25 ft downstream of the powerhouse. This design was tested for 2.5 hr at tailwater el 32.0. Some flow was diverted upward with most falling in the deeper tailwater, and the riprap remained stable. Relocating the wall 27.25 ft downstream of the powerhouse alleviated the splash against the downstream corner of the powerhouse and provided substantial clearance between the extension of the left sidewall (downstream of the powerhouse) and the angled wall.

21. To provide additional clearance for passage of debris through the chute, the extension of the left sidewall beyond the downstream face of the powerhouse was removed. This was designated the type 11 design (Plate 15), and a test was run for 2.5 hr at tailwater el 32.0. Some flow was again diverted upward with more being diverted in the deeper tailwater, and the riprap remained stable. Debris simulating timber logs 2.5 ft square and 25, 50, and 100 ft long were introduced upstream of the powerhouse to observe how well the debris spillway would "draw" and pass debris (Photo 12). At tailwater el 32.0-50.0, the 25- and 50-ft-long logs were drawn into the chute, passed through into the tailrace area, and recirculated downstream of the powerhouse. The 100-ft-long logs became snagged on the spillway crest. Once debris became waterlogged, it was not drawn into the chute. It was concluded that the larger and waterlogged debris would not be passed through the chute and provisions would have to be made for physical removal of such debris.

22. In the type 12 design (Plate 16) the 1-ft chamfers on top and bottom of the wall were removed and replaced with 2.5-ft chamfers, and a test was run for 2.5 hr at tailwater el 32.0. Although the riprap remained stable throughout this test, flow splashed against the powerhouse, increasing the potential for damage to the powerhouse from debris.

23. A 10-ft-high, 45-deg sloping wall located at a 122-deg angle to the right sidewall 27.25 ft downstream of the powerhouse (type 13 design, Plate 17) was tested for 2.5 hr at tailwater el 32.0. Very little of the flow was diverted to the left into the deeper tailwater. Most of the flow

overtopped the wall and plunged into the shallower tailwater. Damage to the riprap in the downstream tailrace was severe.

24. Although the type 11 design energy dissipator (Plate 15) performed satisfactorily at minimum tailwater el 32.0, overtopping of the right sidewall with tailwater el 49.0 and above damaged riprap on the berm adjacent to the right sidewall. Placing a partial roof over the angled wall (type 14 design, Plate 18) prevented overtopping and alleviated riprap damage on the berm along the right sidewall at higher tailwaters, and did not cause any riprap damage in the tailrace at minimum tailwater el 32.0. Engineers from the Mobile District, however, felt that a partial roof would present problems in passing large pieces of debris, and other means of refining the type 11 design energy dissipator were explored.

25. At the request of the Mobile District, the clearance between the 122-deg angled wall with 1-ft chamfers on the top and bottom and the downstream face of the powerhouse (type 15 design, Plate 19) was increased 5 ft (to provide 42.5 ft of clearance from the powerhouse). Riprap in the tailrace was displaced after 2.5 hr of operation at minimum tailwater el 32.0.

26. Efforts were concentrated on alleviating overtopping of the right sidewall and streamlining the 122-deg angled wall for improved hydraulic flow conditions. In the type 16 design, the 10-ft-high right sidewall height was increased 7 ft (Plate 20) and the angled wall was replaced with a 17-ft-high and 21.5-ft-radius wall downstream of the powerhouse. One-ft chamfers were placed at the top, bottom, and end of the curved wall (Plate 21). The excess length of the right sidewall and the apron at el 40.0 were removed and replaced with topography (Figure 4, Plate 21). Overtopping of the right sidewall for tailwater el 49.0-50.0 was alleviated, with only intermittent splash over the curved wall occurring at the higher tailwaters. Although the riprap remained stable in the tailrace after 2.5 hr of operation at minimum tailwater el 32.0, simulated 50-ft-long by 2.5-ft-square logs became snagged on the chamfer on the end of the curved wall.

Recommended Design

27. The 1-ft chamfer was removed from the end of the curved wall (type 17 design, Figure 5 and Plate 22). The tailwater was varied from el 66.0 to 32.0 (Photos 13-20). The model was operated for 2.5 hr at minimum

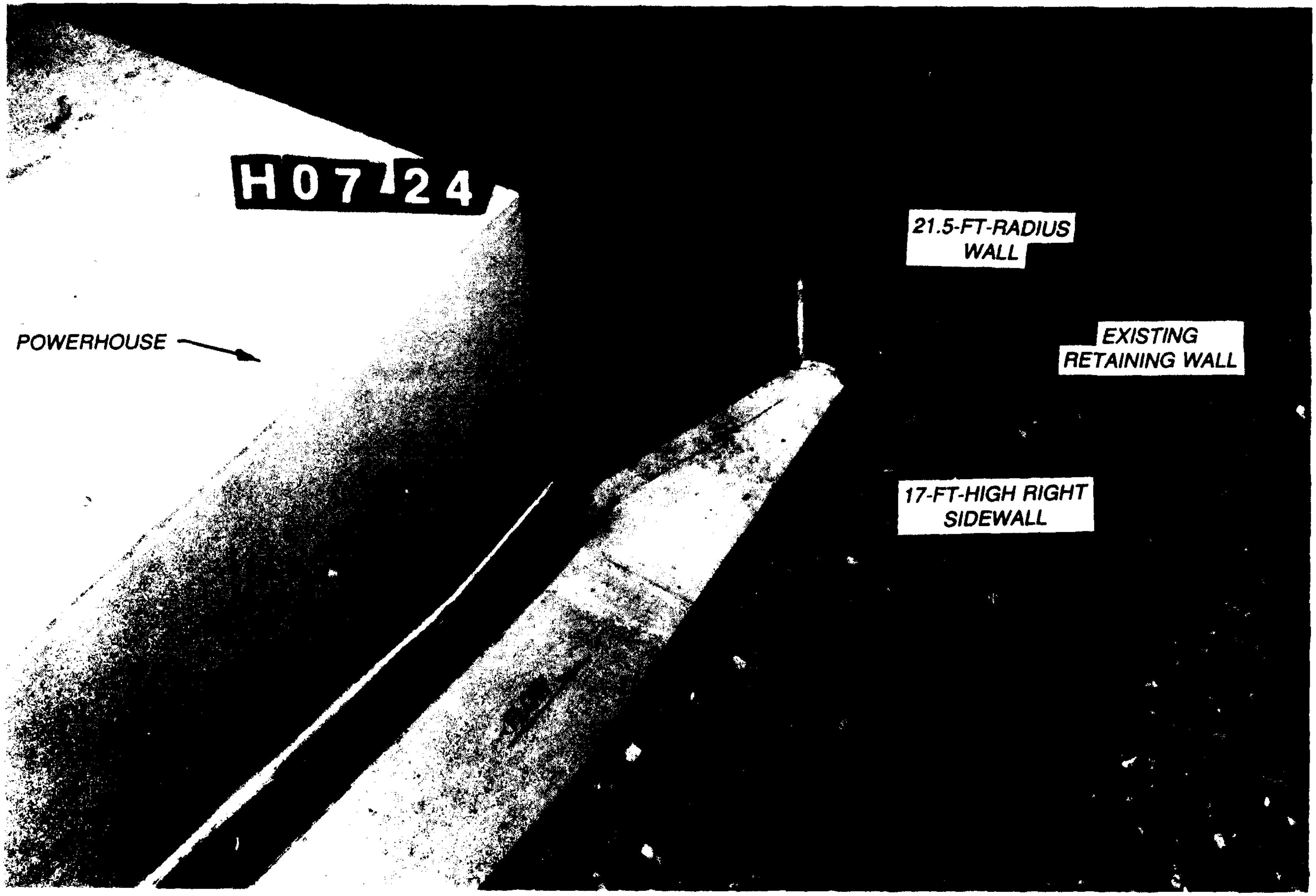
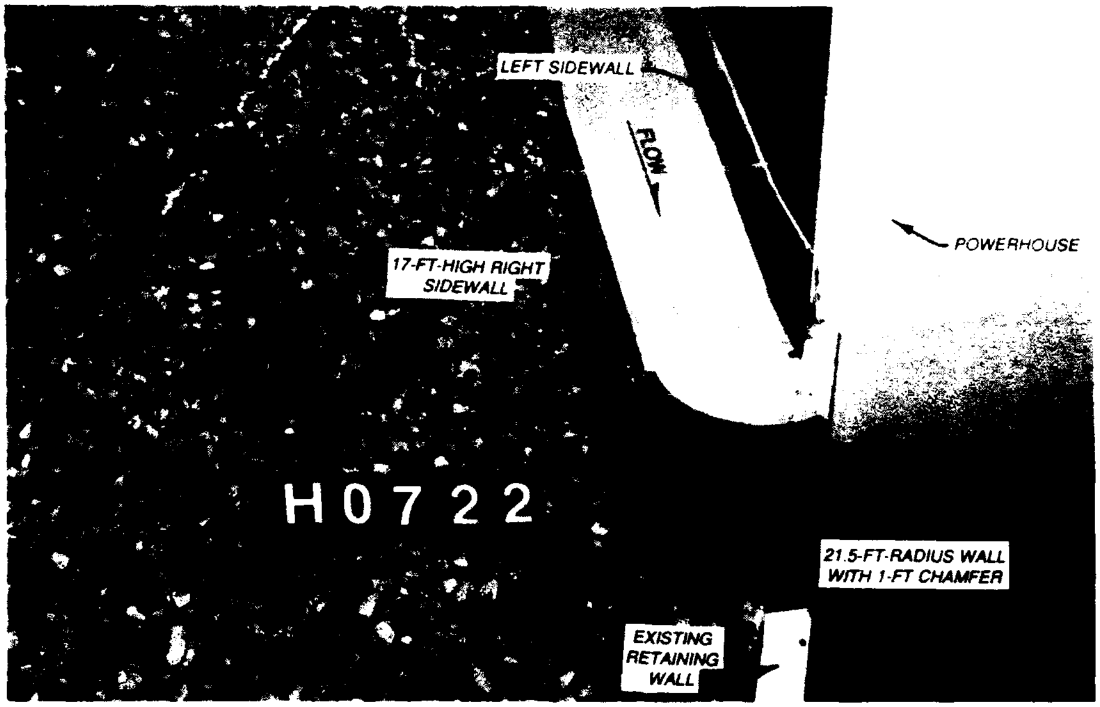
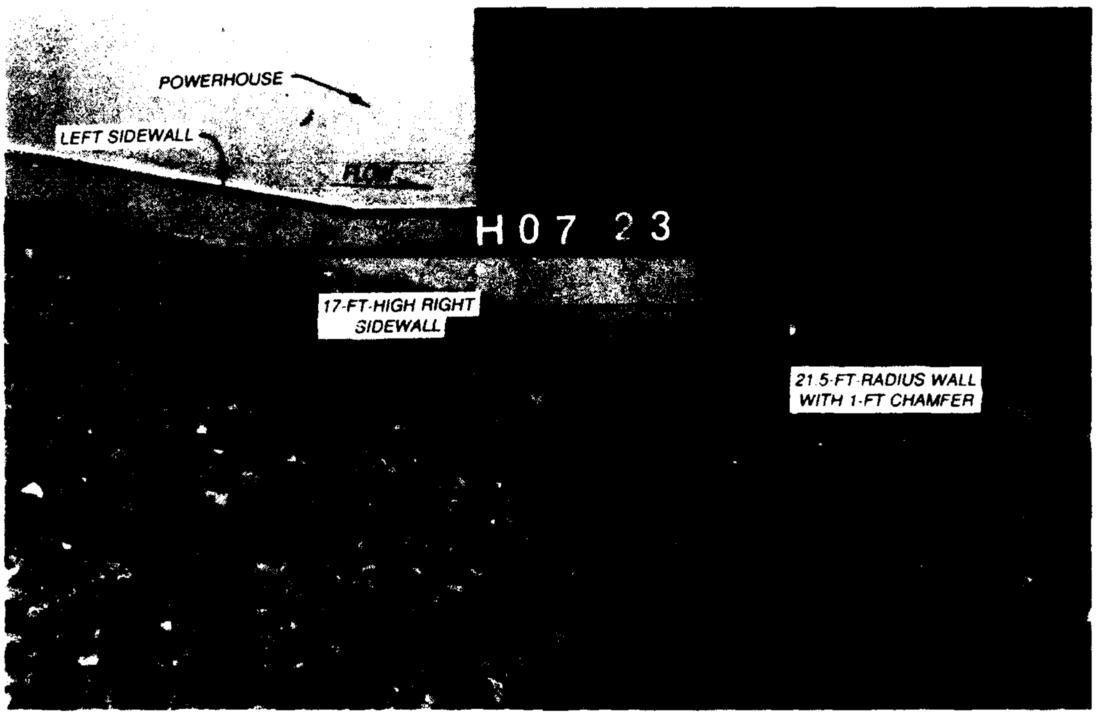


Figure 4. Type 17 design, 21.5-ft-radius wall without 1-ft chamfers and modified topography



a. Looking upstream



b. Profile view

Figure 5. Type 17 design

tailwater el 32.0. The riprap remained stable throughout testing. No overtopping of the walls was observed. All debris except the 100-ft-long debris passed through the chute at tailwater el 32.0-50.0. At tailwater elevations above el 50.0, debris became trapped in a roller at the toe of the debris spillway. As the logs recirculated on the chute, some logs became jammed in the chute, causing overtopping of the right sidewall and washing out of riprap along the wall. It was concluded that the debris spillway could safely and efficiently be operated for tailwaters between el 32.0 and 50.0. Once the tailwater exceeds el 50.0, it is recommended that the debris spillway <u>not</u> be operated. Therefore, the type 17 design energy dissipator (Figure 5, Plate 22) was recommended for prototype construction with the specification that the debris spillway not be operated at tailwaters greater than el 50.0.

28. Velocities measured 0.5 ft above the horizontal apron ranged from 45.5 to 49.0 fps at tailwater el 32.0. The velocity data are listed in Table 2. Water-surface profiles were measured along the right and left training walls and along the center line of the chute for the recommended design. The center-line water-surface profile is plotted in Plate 23 and the data are listed in Table 3.

PART IV: DISCUSSION AND RECOMMENDATIONS

29. Floating debris in the Millers Ferry Powerhouse forebay has been a continual problem since the project was completed. Various methods of removal of the debris have been used over the years with limited success. A chute type debris spillway was designed by the Mobile District and proposed for construction along the right side of the existing powerhouse. Model tests indicated that with some modifications this structure could efficiently pass floating debris without causing adverse flow conditions downstream.

30. A 3-ft-radius upstream nose was added to the face of the powerhouse to decrease flow contraction at the left entrance to the spillway.

31. Velocities of 45.5 to 49 fps were measured in the chute exit at minimum tailwater (el 32.0). To avoid damage to the existing 24-in.-thick riprap blanket, an energy dissipator had to be designed that would soften the jet by spreading the flow in the deeper tailwater downstream of the powerhouse. Type 4 and 5 deflectors 4 and 5 ft high, respectively, did not dissipate enough energy. Overtopping of the 10-ft-high right sidewall (types 3-15) caused damage to a berm along the right wall. A 17-ft-high, 21.5-ft-radius curved wall with 1-ft chamfers at the top and the bottom of the wall located downstream of the powerhouse was recommended for prototype construction (type 17). Removing the extension of the left sidewall beyond the powerhouse provided more clearance between the curved and left sidewalls for passage of large debris.

Table 1

Water-Surface Elevations Along

Debris Chute Center Line

Type 1 (Original) Design

Q = 2,640 cfs

Distance from Crest, ft	Water-Surface El
Upstream	
79.5	80.9
54.5	80.4
29.5	79.5
4.5	79.1
Downstream	
20.5	61.5
45.5	55.1
70.5	50.3
95.5	45.5
120.5	40.7
145.5	35.9
167.0	31.8

Table 2

Velocities at Chute Toe

Type 17 Design

Q = 2,640 cfs

Distance from Chute Center Line ft	Left or Right of Center Line	Velocity fps
9.5	Left	46.5
4.5	Left	48.5
0	-	46.3
4.5	Right	45.5
9.5	Right	49.0

Note: Velocities measured 0.5 ft above apron.

Table 3

Water-Surface Elevations Along

Debris Chute Center Line

Type 17 Design

Q = 2,640 cfs

Distance from Crest, ft	Water-Surface El
Upstream	
79.5	81.0
54.5	80.4
29.5	79.5
4.5	79.1
Downstream	
20.5	61.5
45.5	55.4
70.5	50.2
95.5	45.2
120.5	42.3
145.5	45.9

Photo 1. Flow conditions, type 1 design, discharge 2,640 cfs; pool el 80.8; tailwater el 66.0

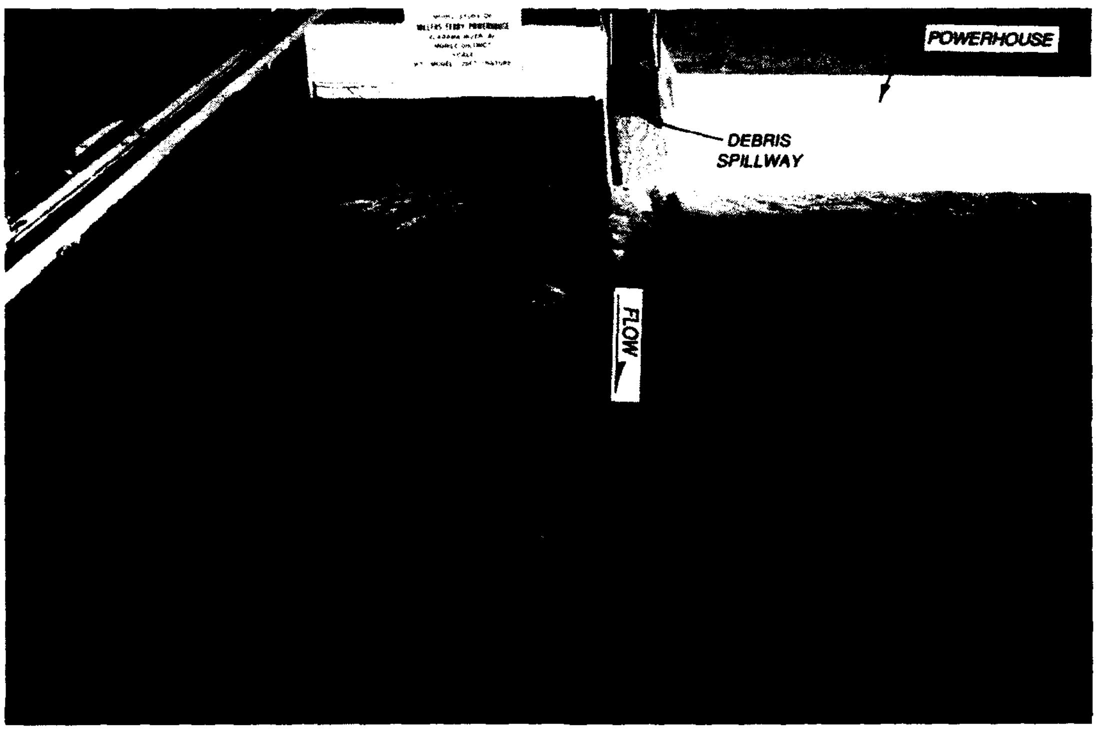


Photo 2. Flow conditions, type 1 design, discharge 2,640 cfs; pool el 80.8; tailwater el 50.0

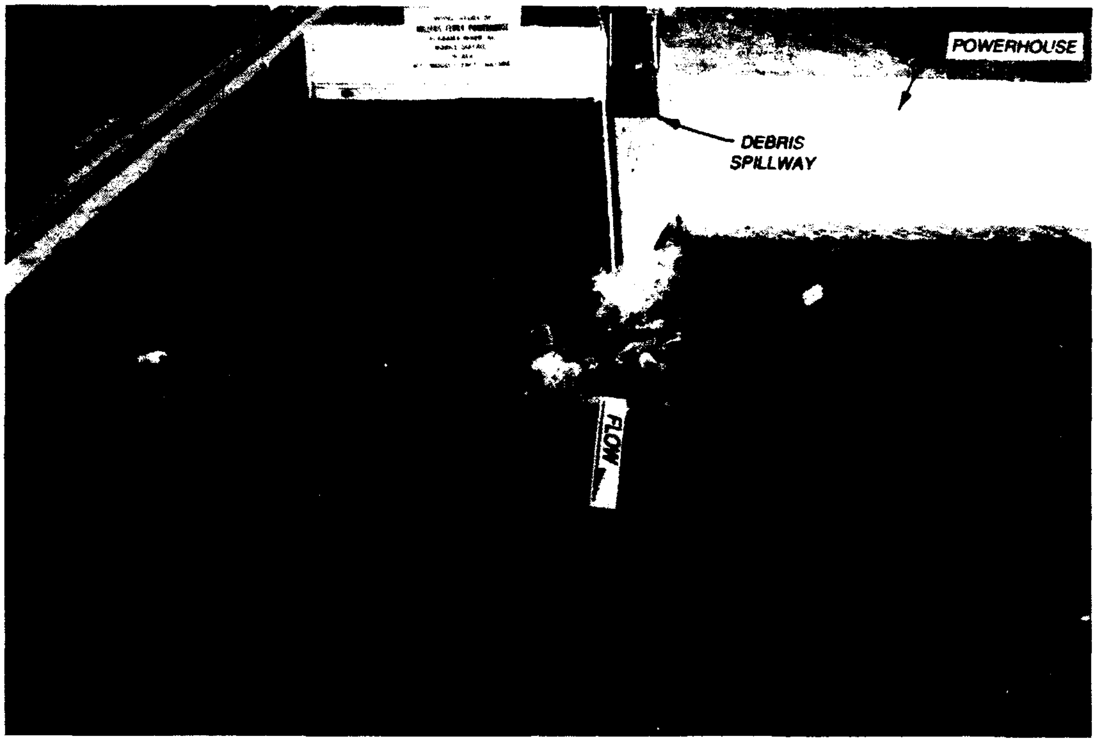


Photo 3. Flow conditions, type 1 design, discharge 2,640 cfs; pool el 80.8; tailwater el 40.0

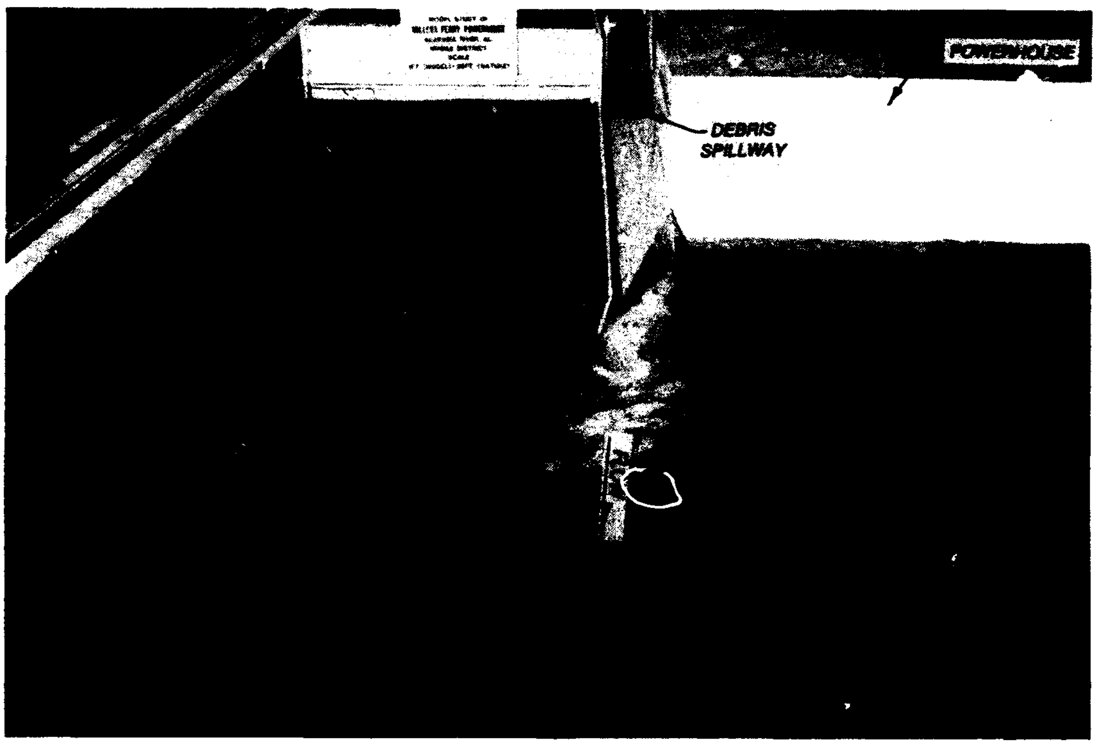


Photo 4. Flow conditions, type 1 design, discharge 2,640 cfs; pool el 80.8; tailwater el 32.0

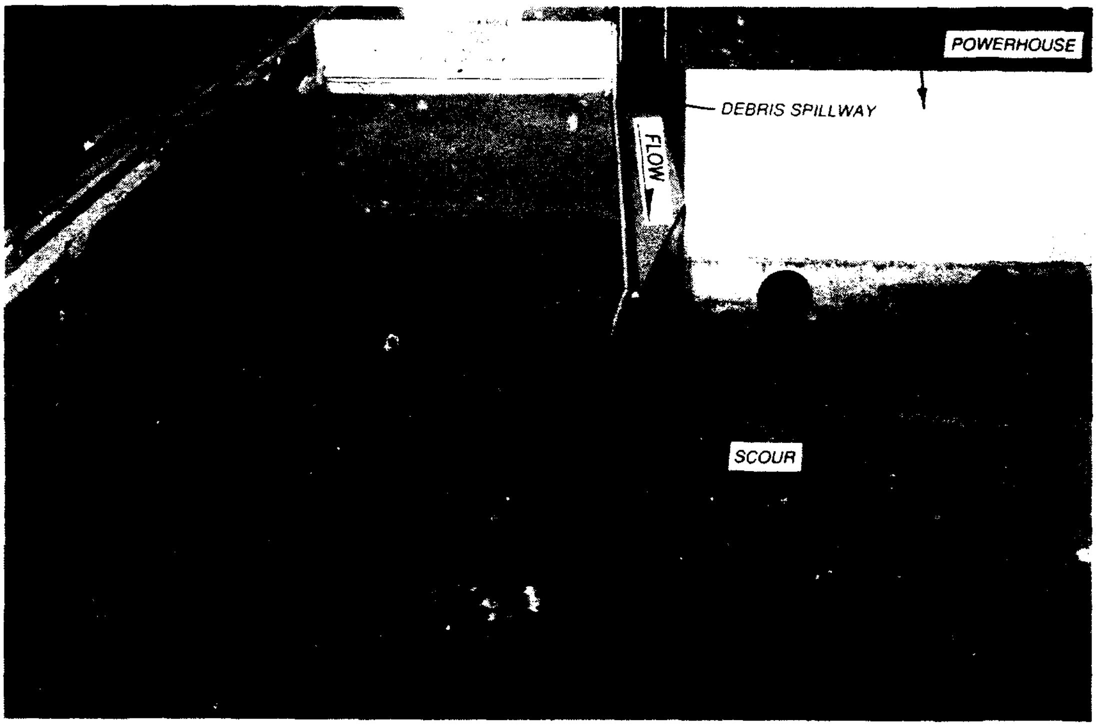


Photo 5. Scour downstream, type 1 design debris spillway; discharge 2,640 cfs, pool el 80.8; tailwater varied from el 66.0 to 32.0

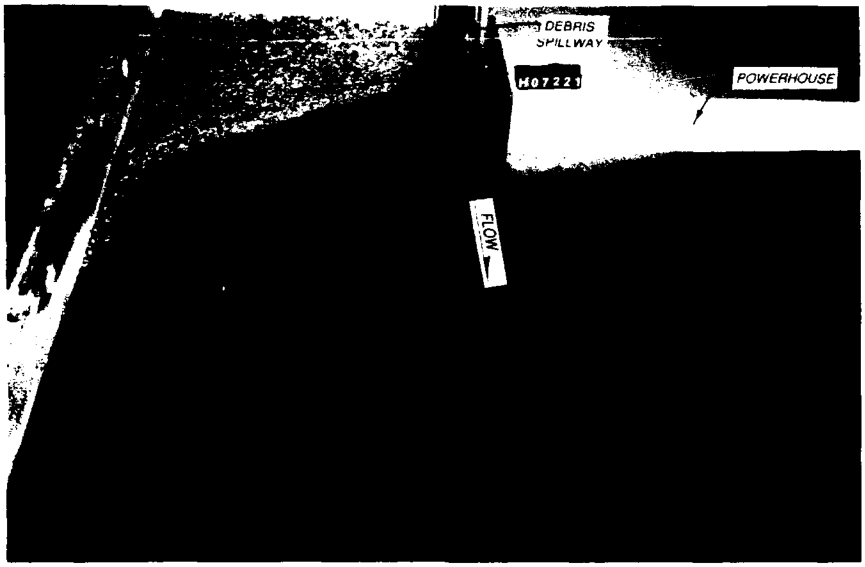


Photo 6. Flow conditions, type 3 design, discharge 2,640 cfs; pool el 80.8; tailwater el 66.0

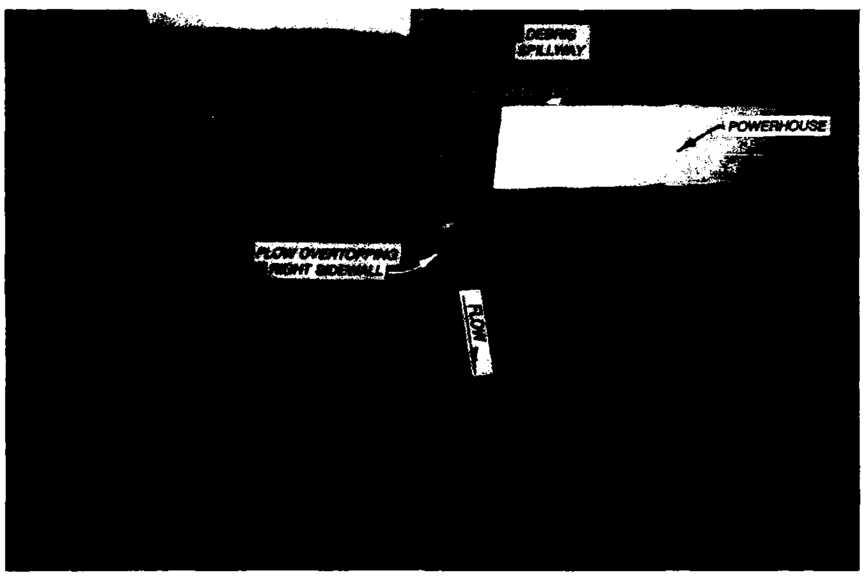
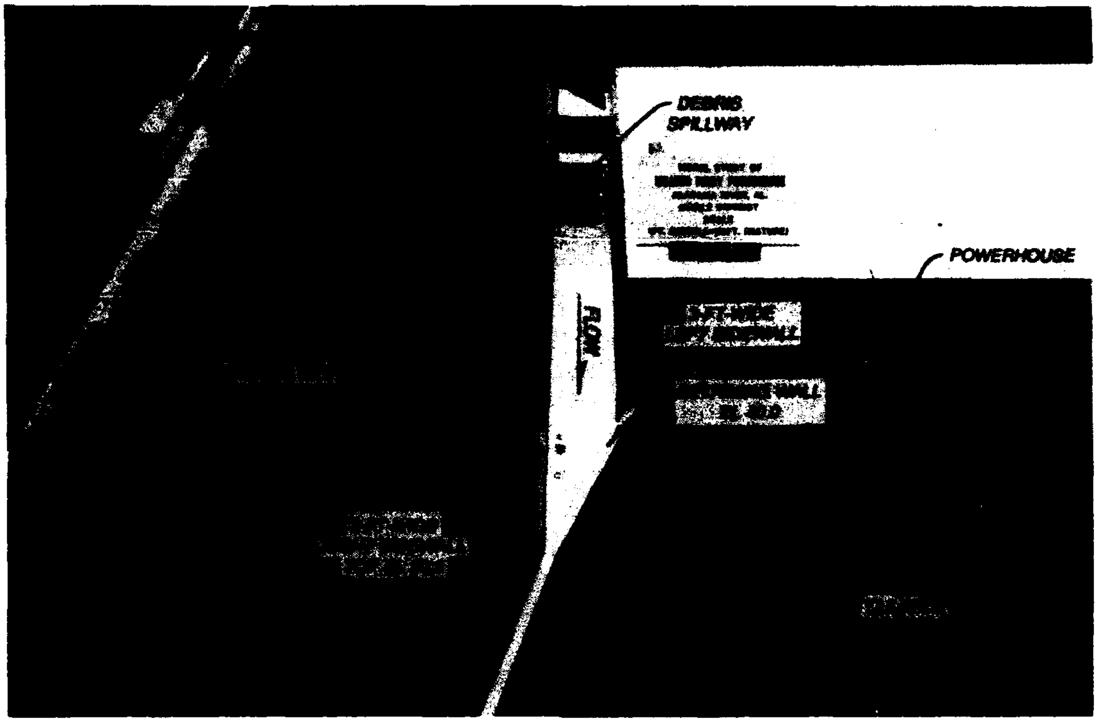


Photo 7. Flow conditions, type 3 design, discharge 2,640 cfs; pool el 80.8; tailwater el 50.0

Photo 8. Flow conditions, type 3 design, discharge 2,640 cfs; pool el 80.8; tailwater el 40.0

Photo 9. Flow conditions, type 3 design, discharge 2,640 cfs; pool el 80.8; tailwater el 32.0



a. Looking upstream at 0 hr

b. Scour after 30 min

Photo 10. Riprap damage on the berm at el 47.5, type 3 design, discharge 2,640 cfs; pool el 80.8; tailwater el 50.0

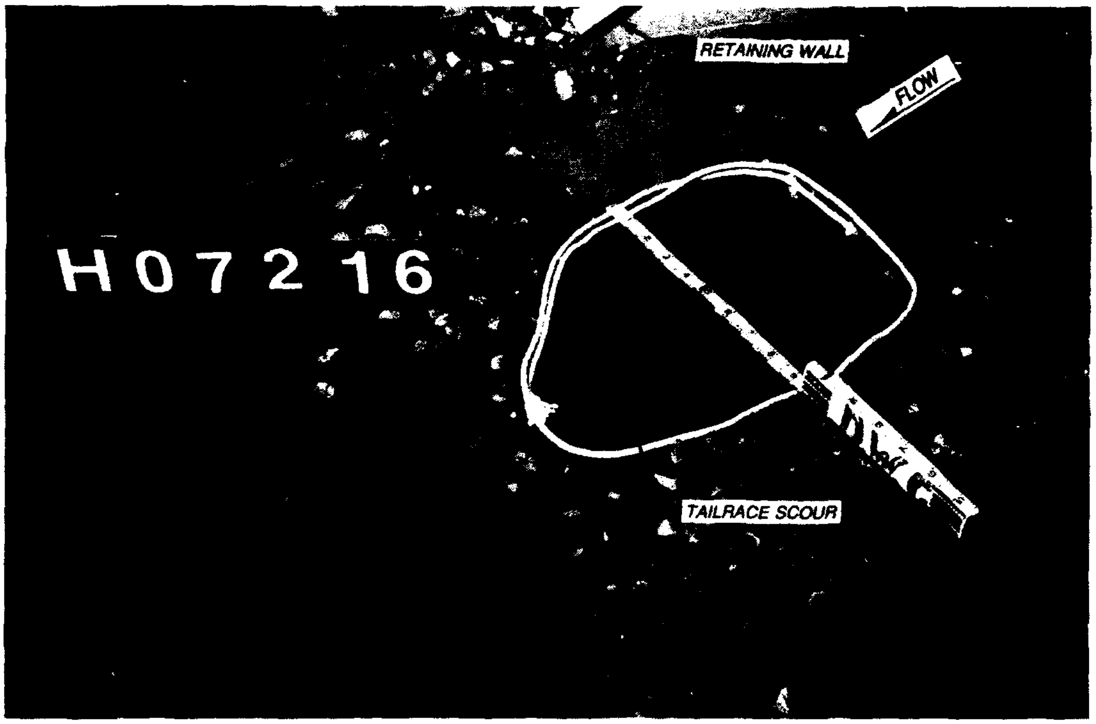


Photo 11. Scour in the tailrace riprap, type 3 design, discharge 2,640 cfs; pool el 80.8; tailwater el 32.0, 2.5 hr

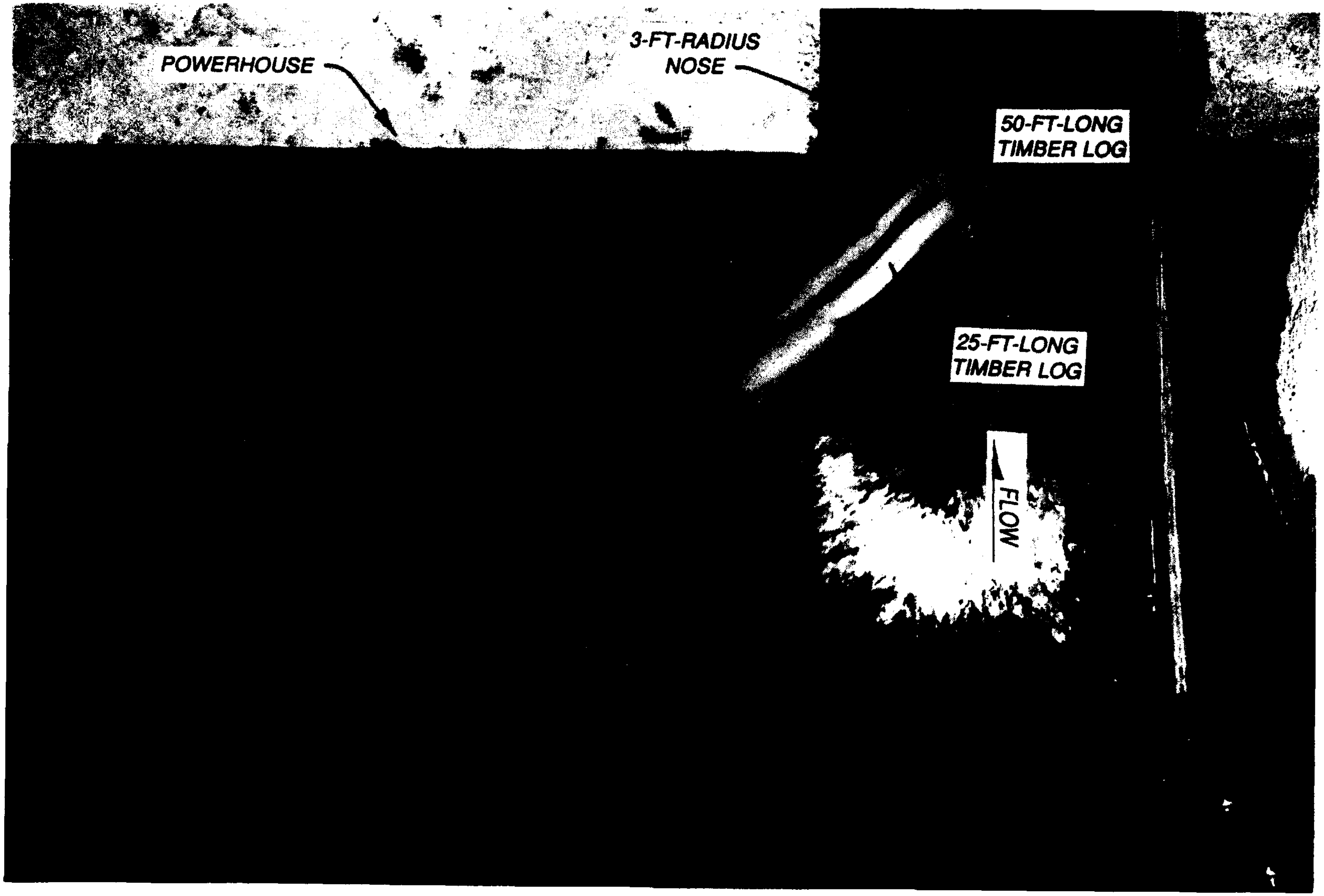


Photo 12. Type 17 design, timber logs entering debris spillway chute, discharge 2,640 cfs

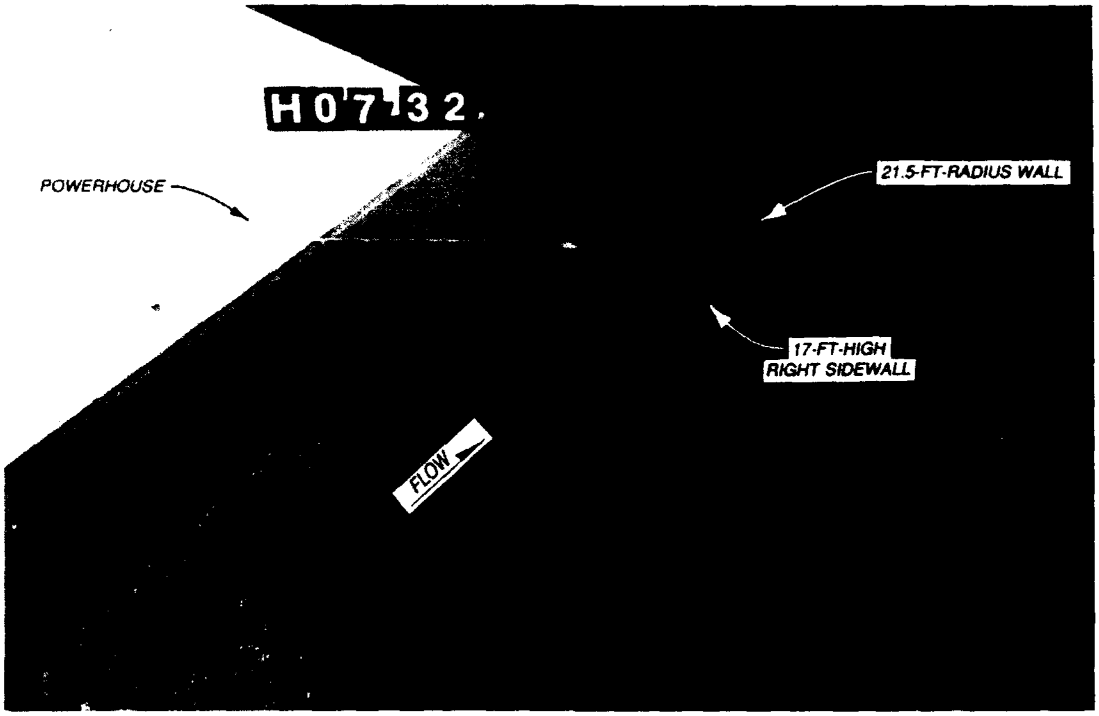


Photo 13. Flow conditions, type 17 design, chute flow; discharge 2,640 cfs; pool el 80.8; tailwater el 66.0

Photo 14. Flow conditions, type 17 design, chute flow; discharge 2,640 cfs; pool el 80.8; tailwater el 50.0

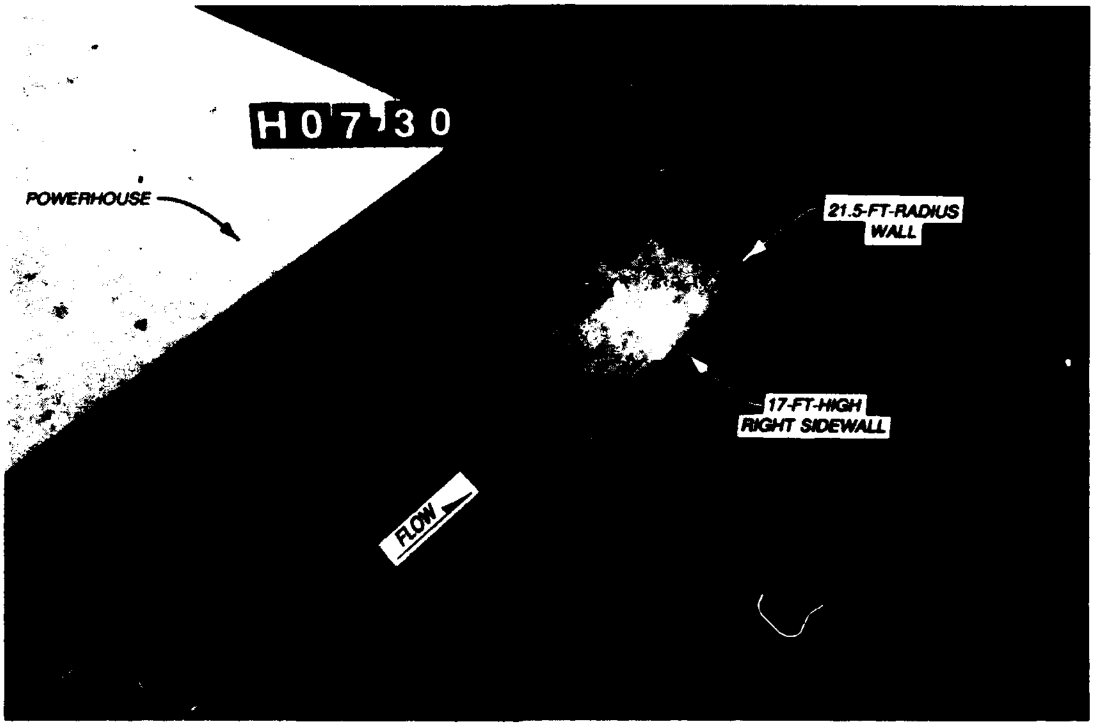


Photo 15. Flow conditions, type 17 design, chute flow; discharge 2,640 cfs; pool el 80.8; tailwater el 40.0

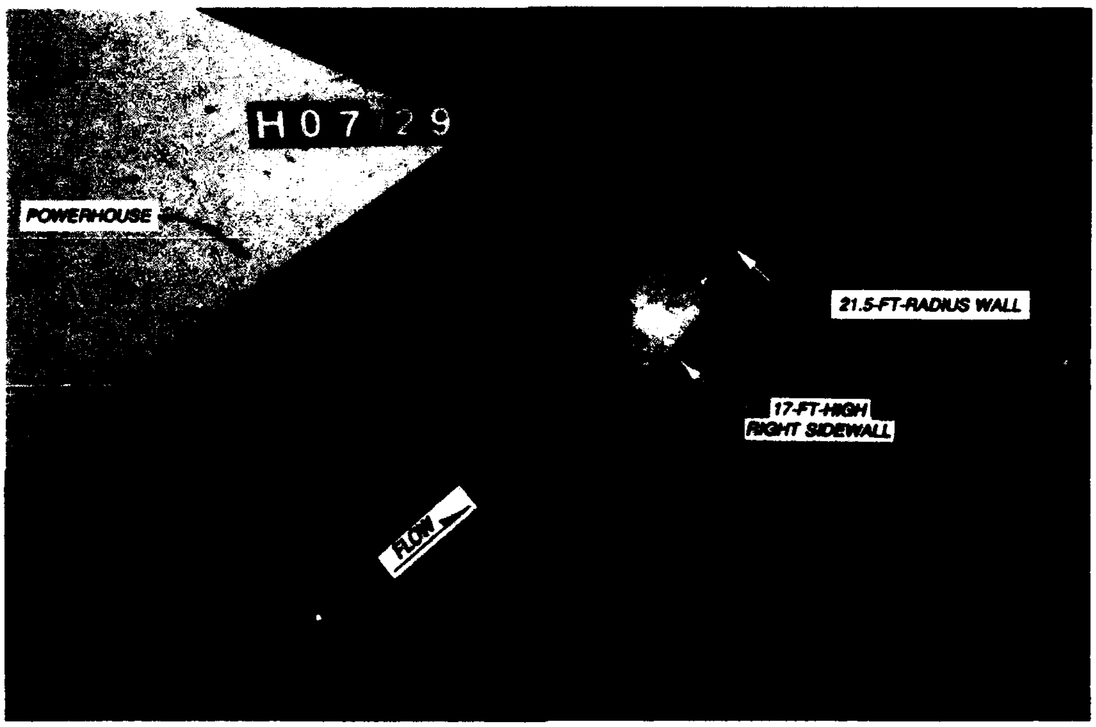


Photo 16. Flow conditions, type 17 design, chute flow; discharge 2,640 cfs; pool el 80.8; tailwater el 32.0

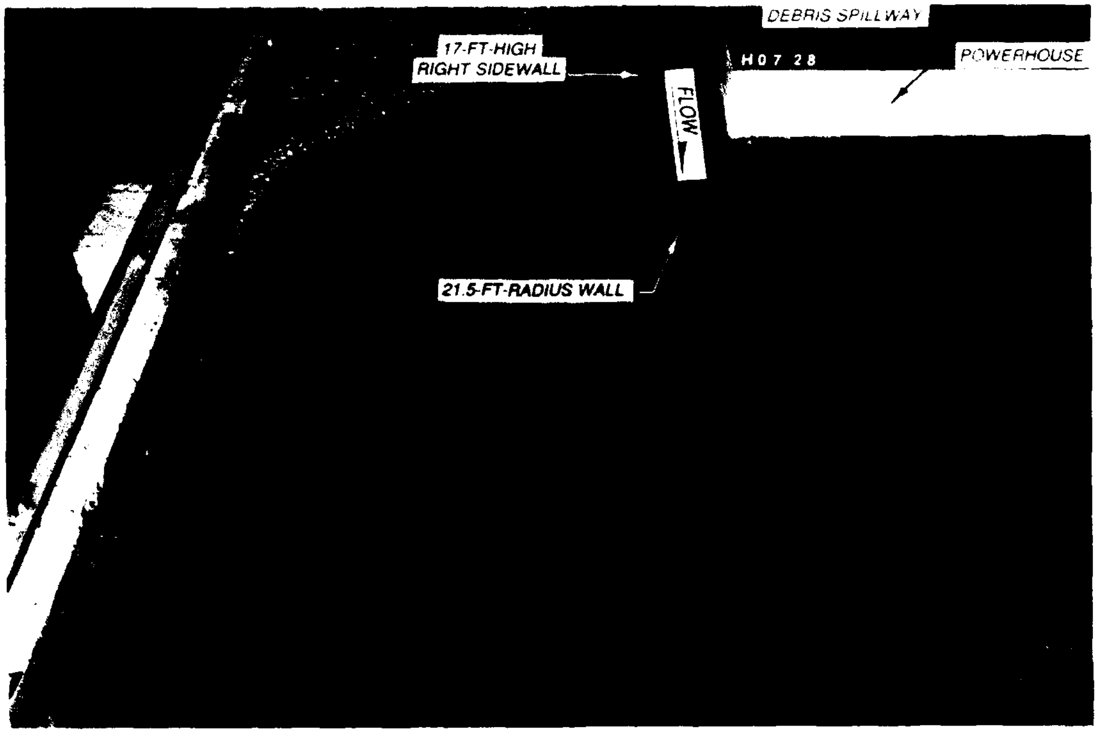


Photo 17. Flow conditions, type 17 design; discharge 2,640 cfs; pool el 80.8; tailwater el 66.0

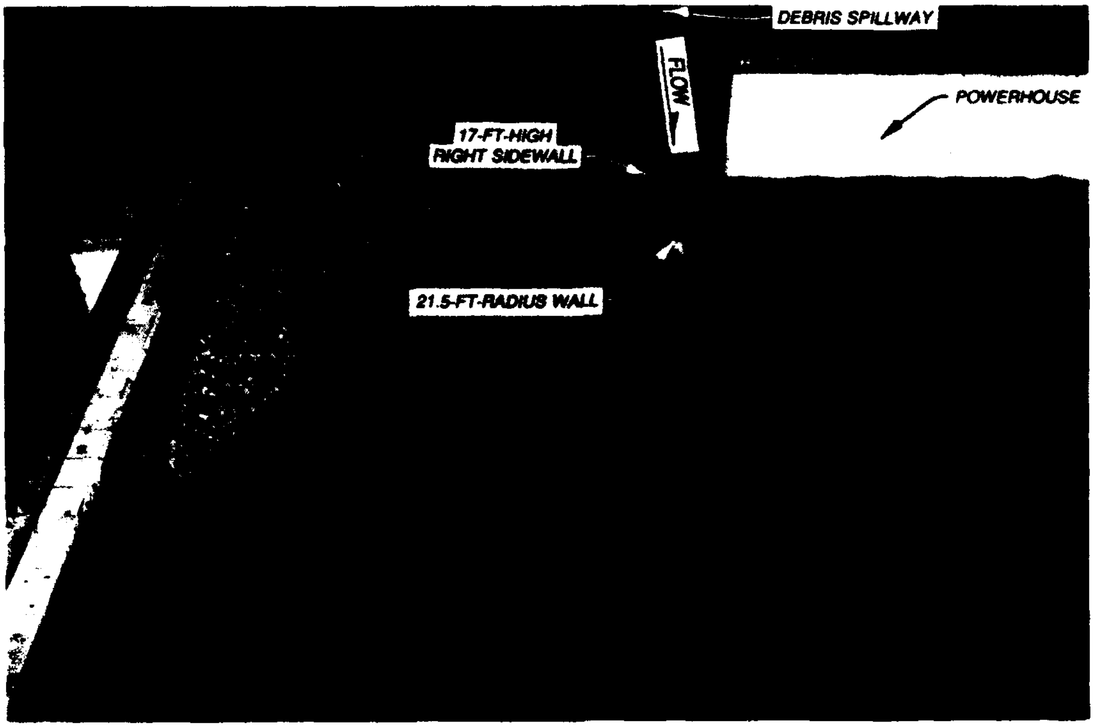


Photo 18. Flow conditions, type 17 design; discharge 2,640 cfs; pool el 80.8; tailwater el 50.0

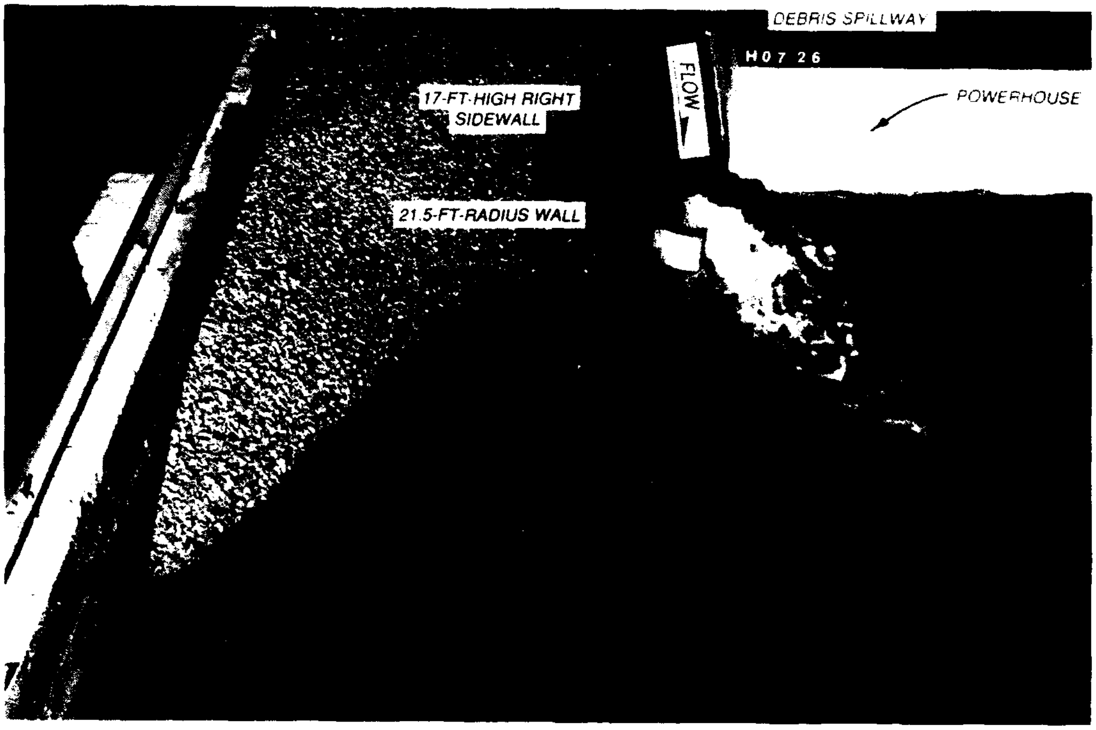


Photo 19. Flow conditions, type 17 design; discharge 2,640 cfs; pool el 80.8; tailwater el 40.0

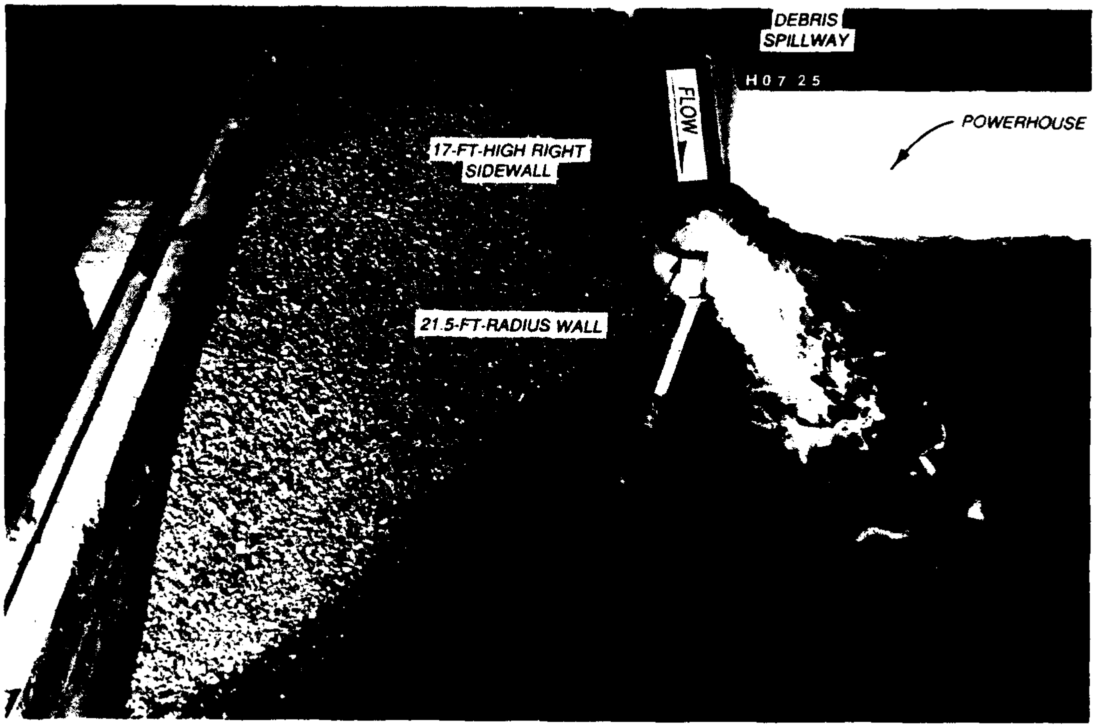
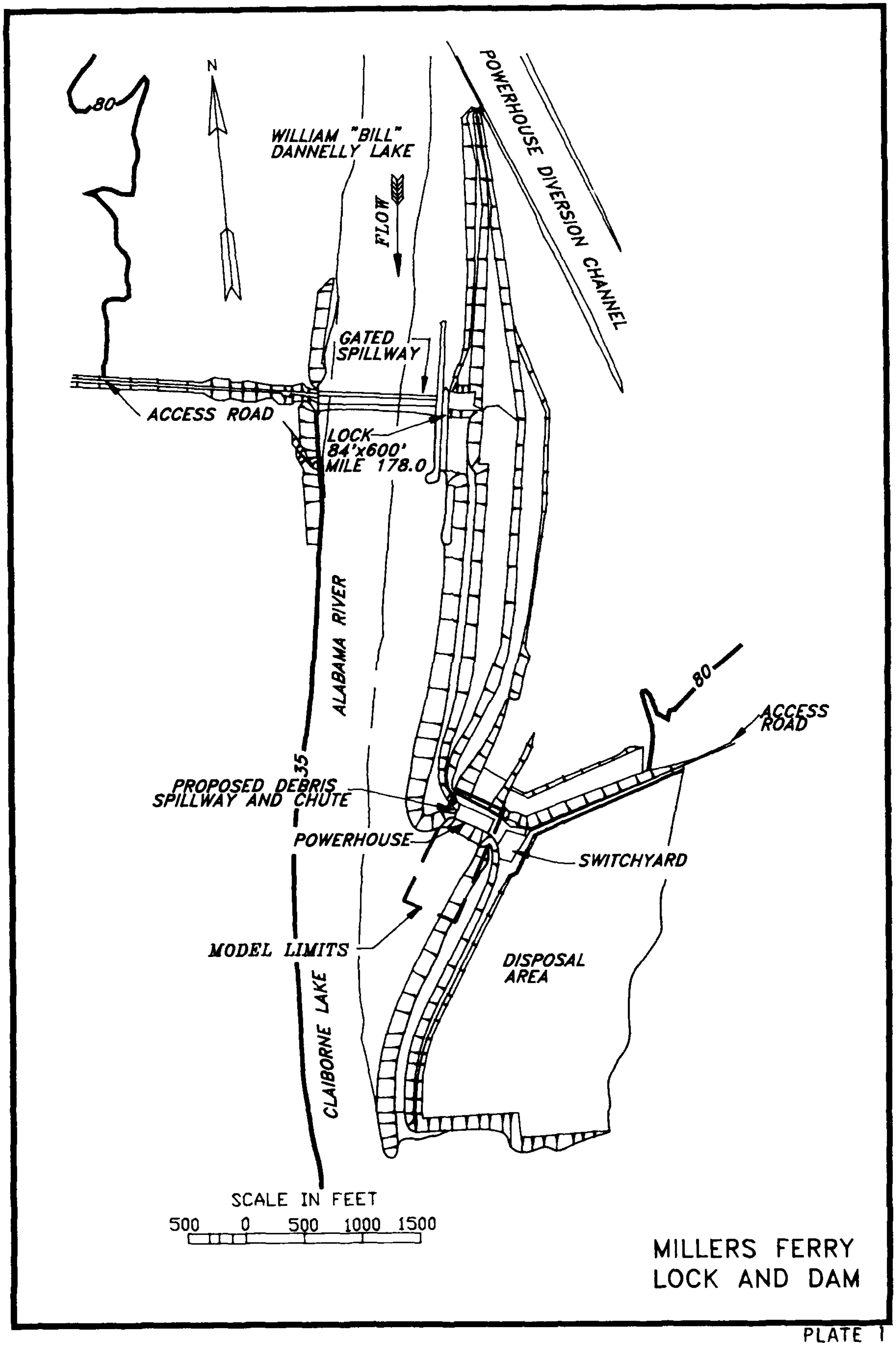
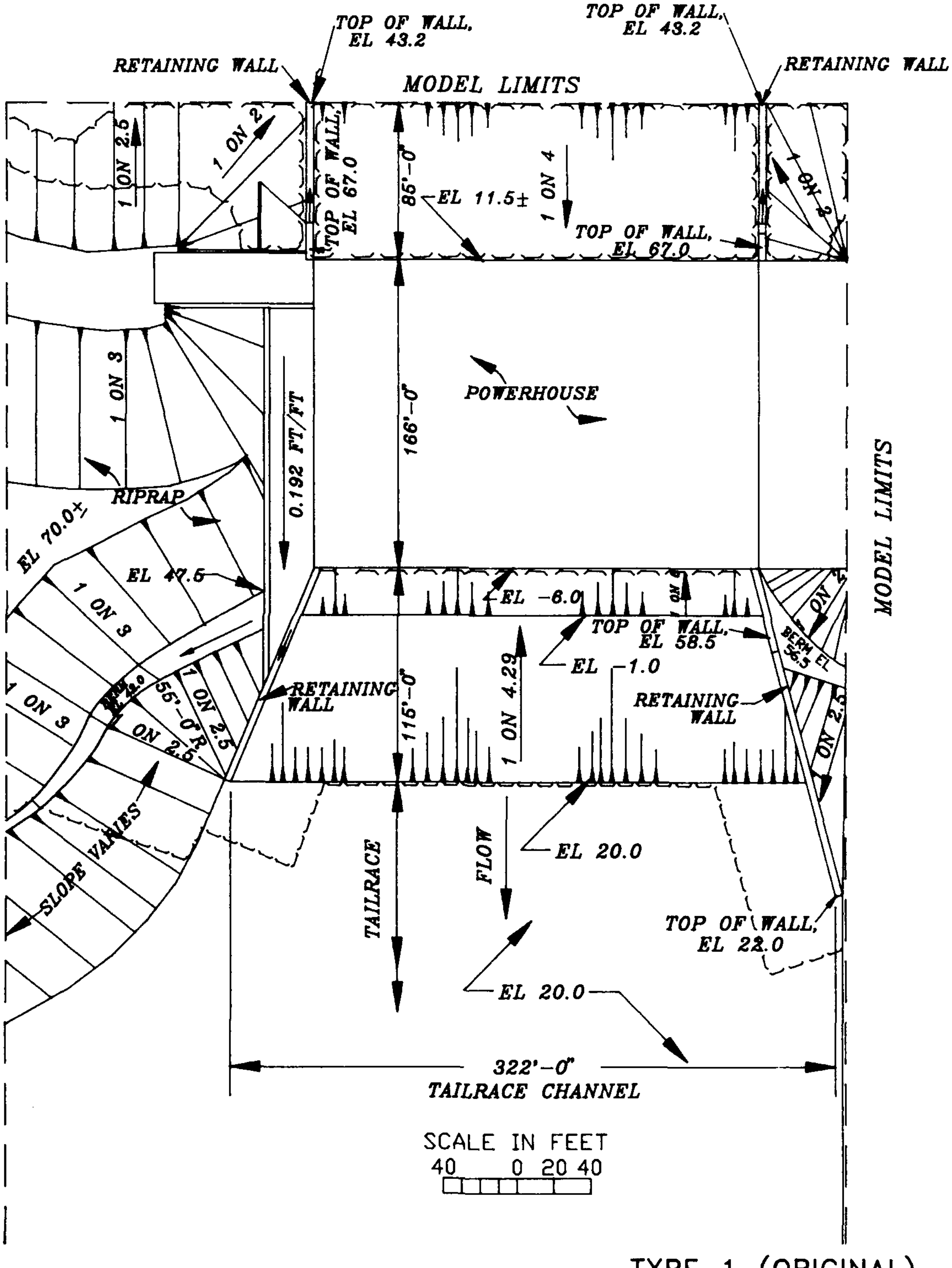


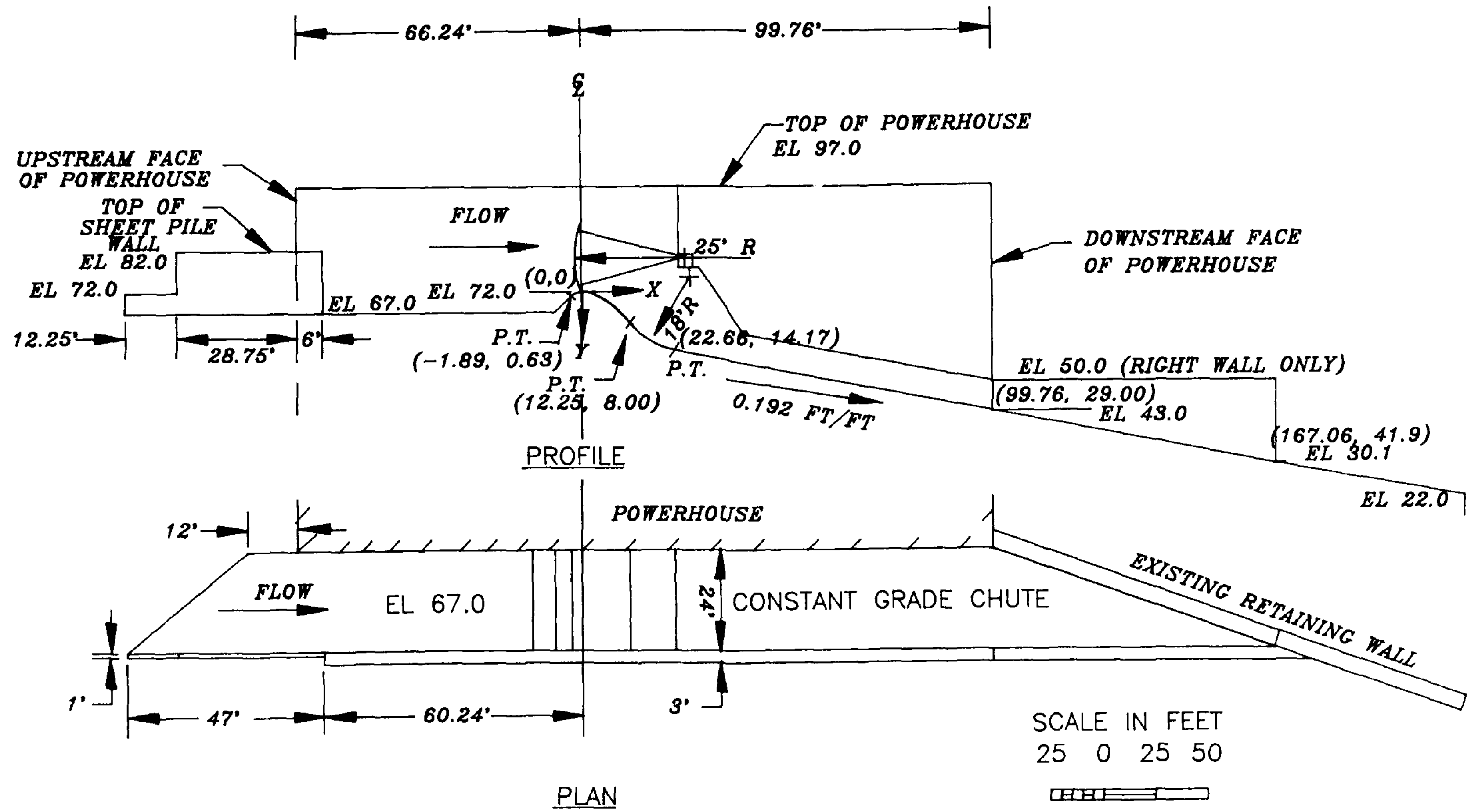
Photo 20. Flow conditions, type 17 design; discharge 2,640 cfs; pool el 80.8; tailwater el 32.0


N
80
WILLIAM "BILL" DANNELLY LAKE
FLOW
POWERHOUSE DIVERSION CHANNEL
GATED SPILLWAY
ACCESS ROAD
LOCK 84'x600' MILE 178.0
ALABAMA RIVER
80
ACCESS ROAD
35
PROPOSED DEBRIS SPILLWAY AND CHUTE
POWERHOUSE
SWITCHYARD
MODEL LIMITS
DISPOSAL AREA
CLAIBORNE LAKE
SCALE IN FEET
500 0 500 1000 1500
MILLERS FERRY LOCK AND DAM

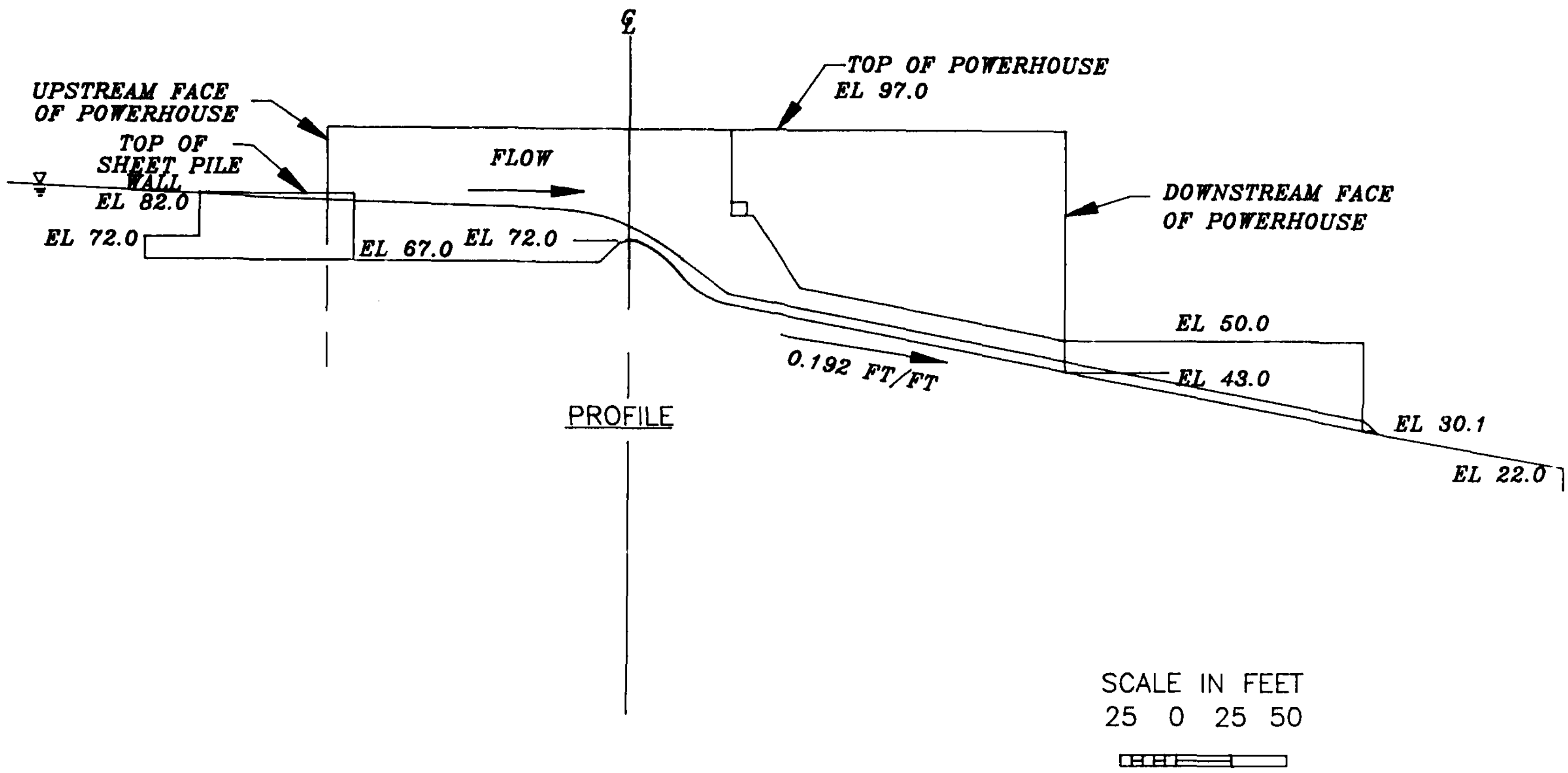


TYPE 1 (ORIGINAL) DESIGN

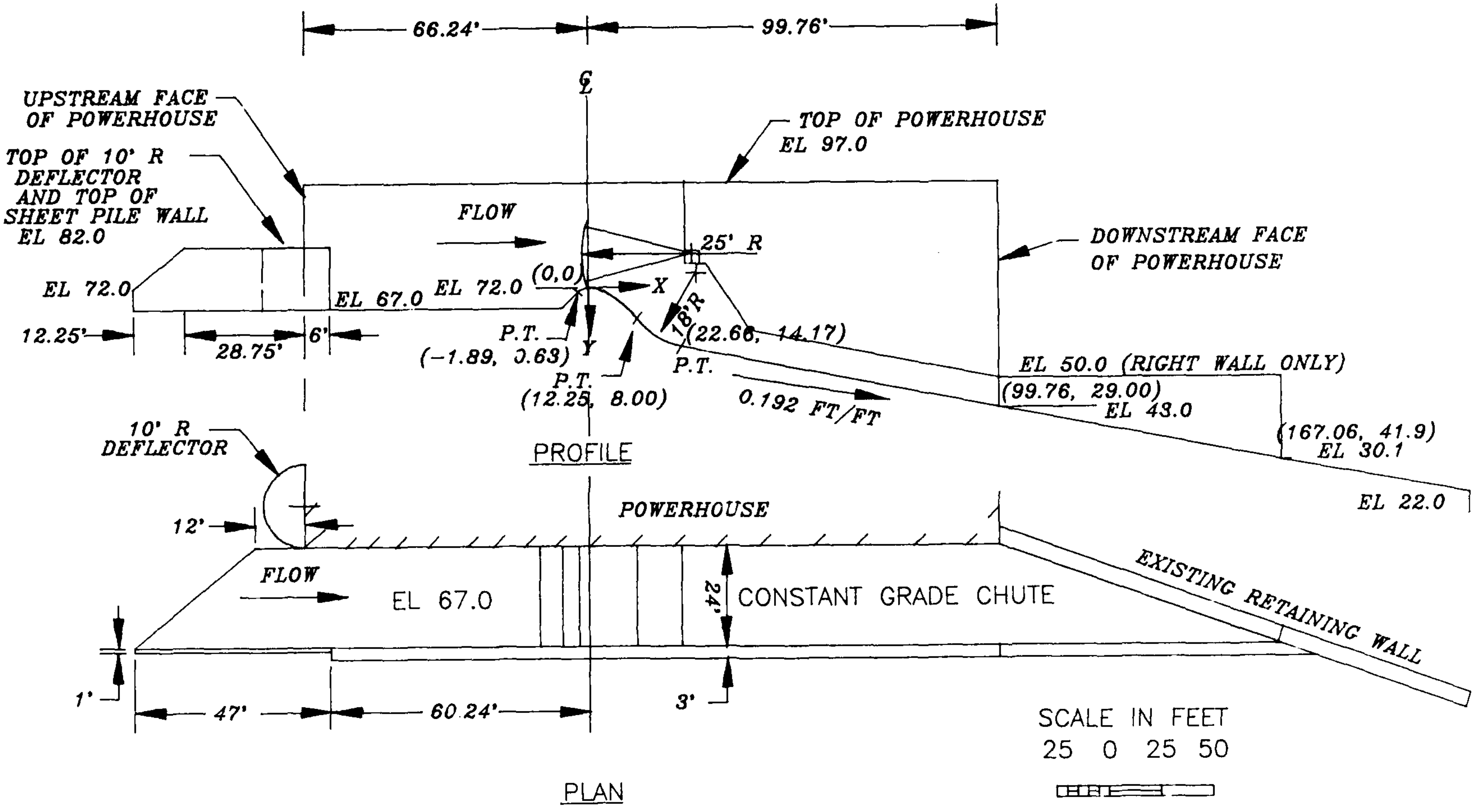
PLATE 2



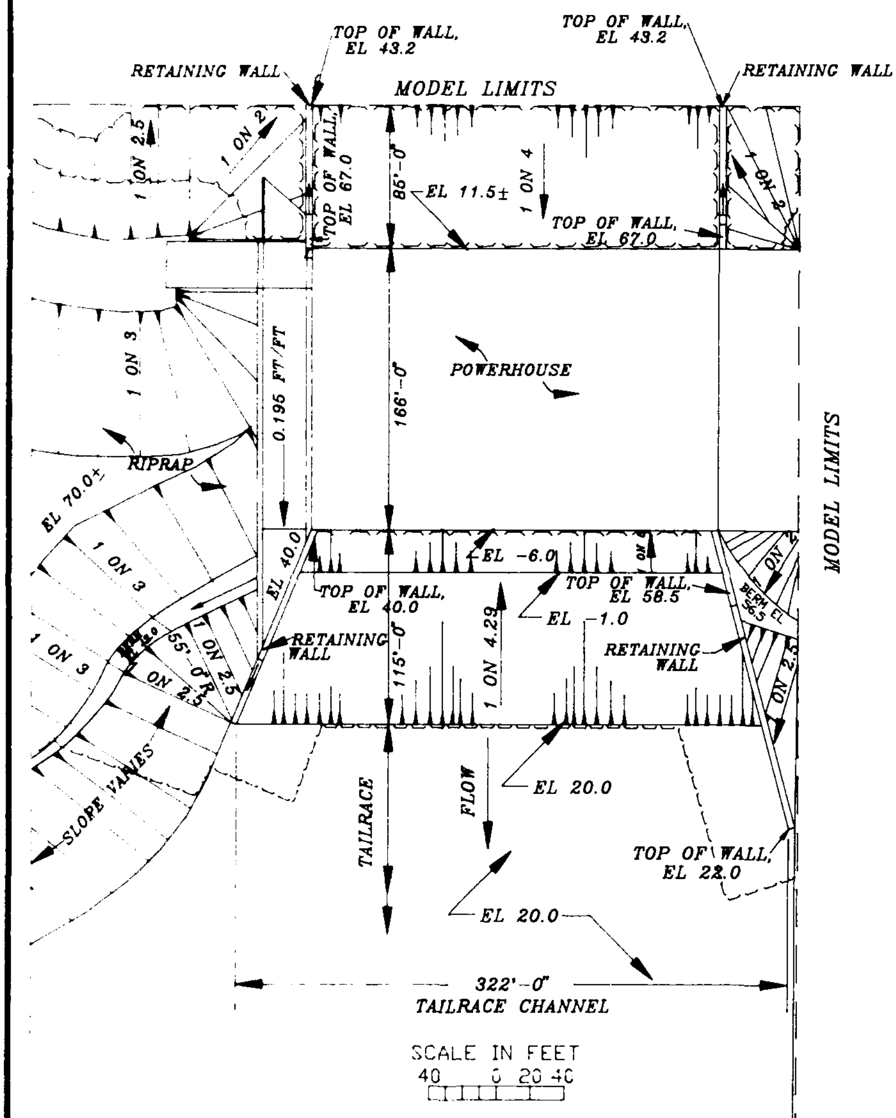
TYPE 1 (ORIGINAL) DESIGN
DEBRIS SPILLWAY



TYPE 1 (ORIGINAL) DESIGN
WATER-SURFACE PROFILE
Q = 2,640 CFS

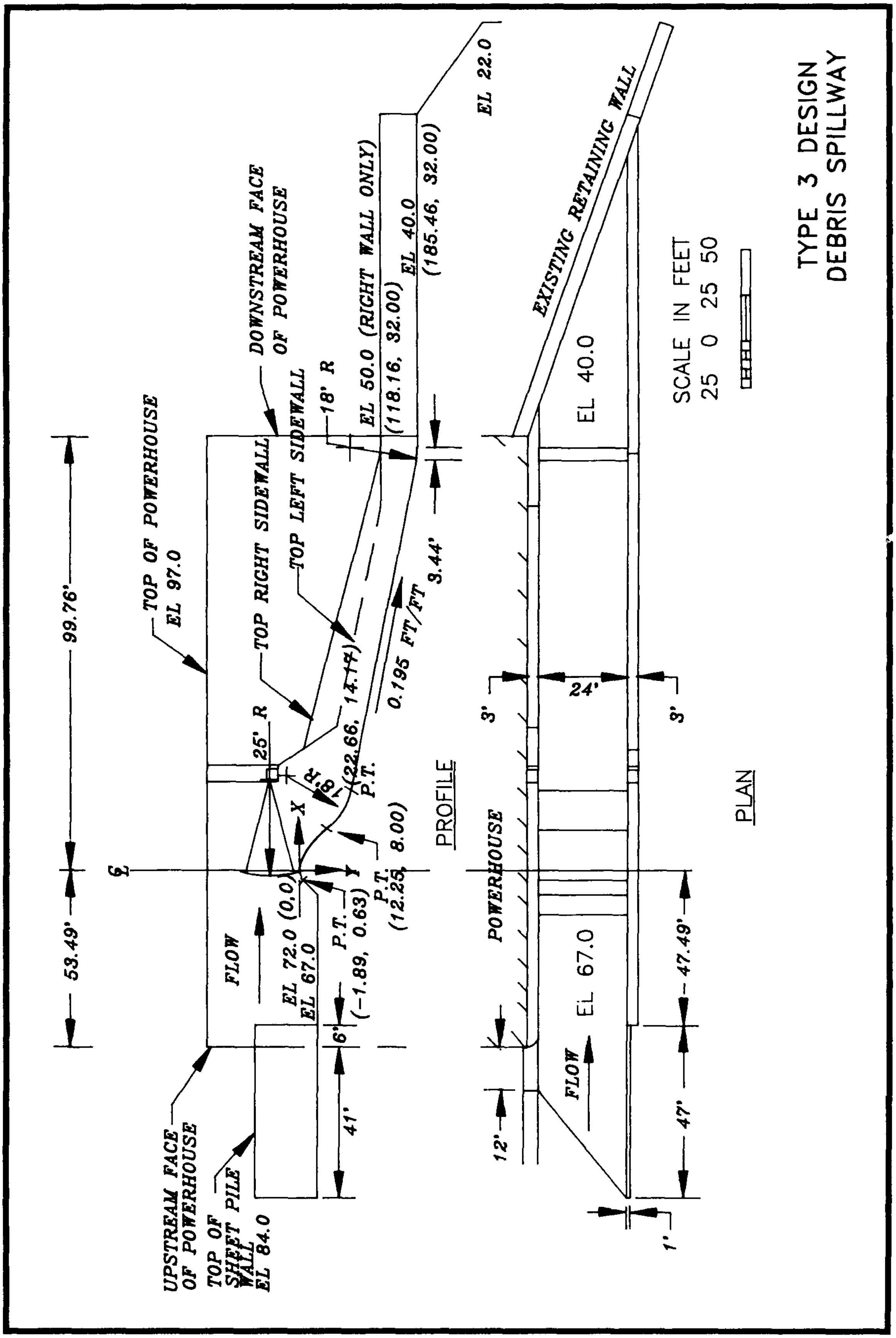


TYPE 2 DESIGN
DEBRIS SPILLWAY



TYPE 3 DESIGN

PLATE 6



TYPE 3 DESIGN
DEBRIS SPILLWAY
SCALE IN FEET
25 0 25 50
PROFILE
PLAN
UPSTREAM FACE OF POWERHOUSE
TOP OF SHEET PILE WALL EL 84.0
TOP OF POWERHOUSE EL 97.0
DOWNSTREAM FACE OF POWERHOUSE
TOP RIGHT SIDEWALL
TOP LEFT SIDEWALL
FLOW
EL 72.0 (0,0)
EL 67.0
P.T. (-1.89, 0.63)
P.T. (12.25, 8.00)
P.T. (22.66, 14.17)
25' R
18' R
X
Y
0.195 FT/FT
EL 50.0 (RIGHT WALL ONLY)
(118.16, 32.00)
EL 40.0
(185.46, 32.00)
EL 22.0
53.49'
99.76'
6'
41'
3.44'
EXISTING RETAINING WALL
POWERHOUSE
EL 67.0
EL 40.0
3'
24'
3'
12'
47'
47.49'
1'

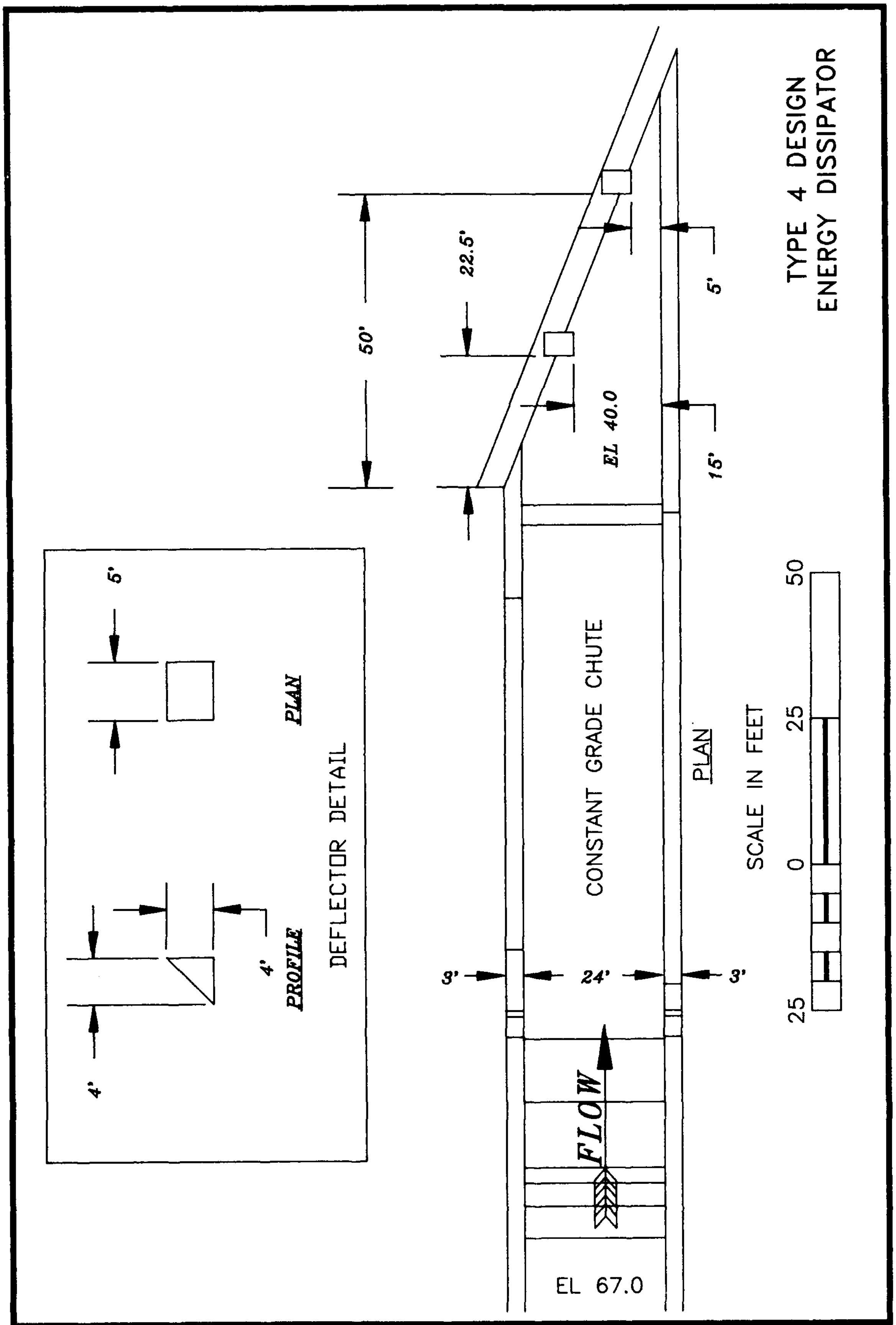
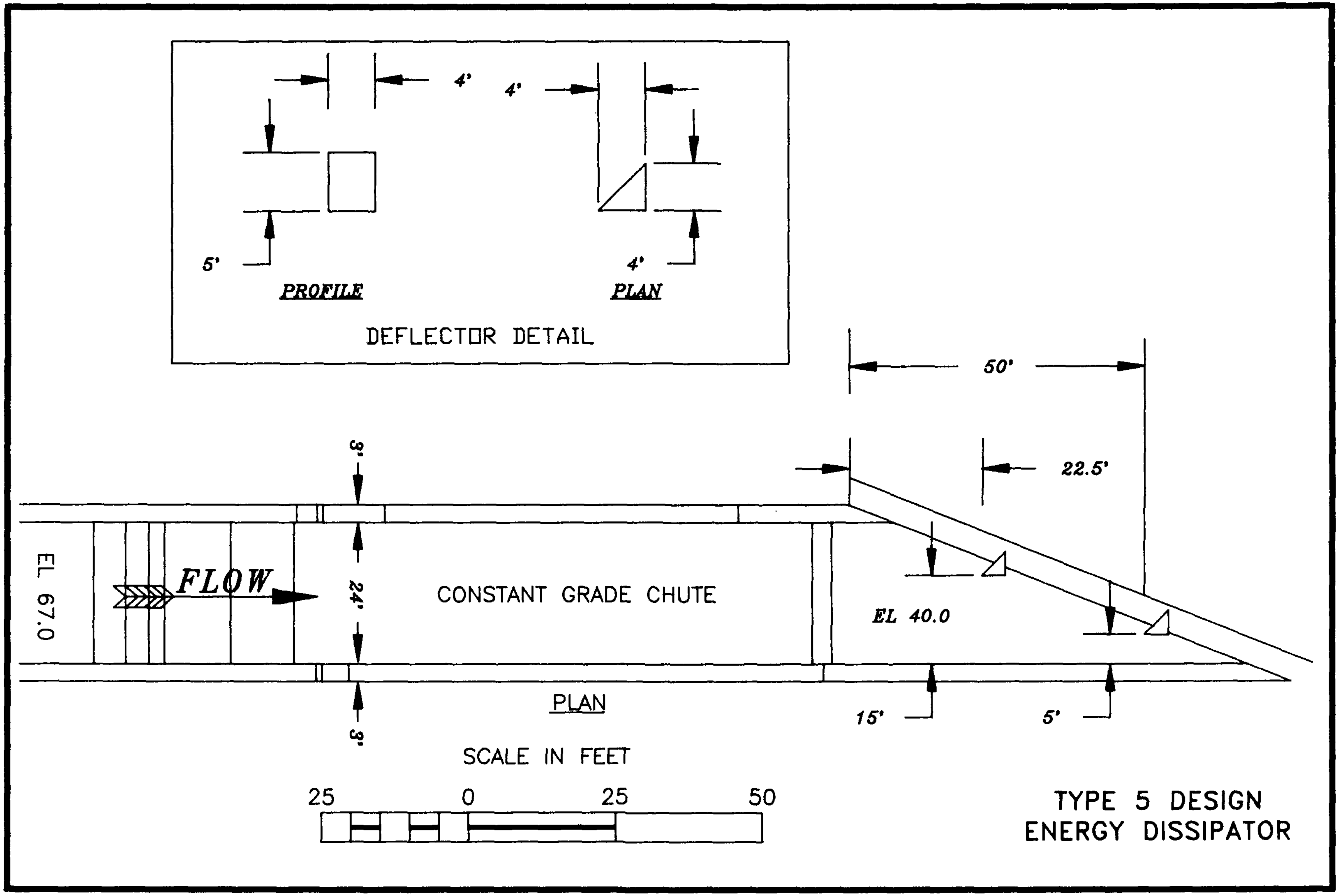
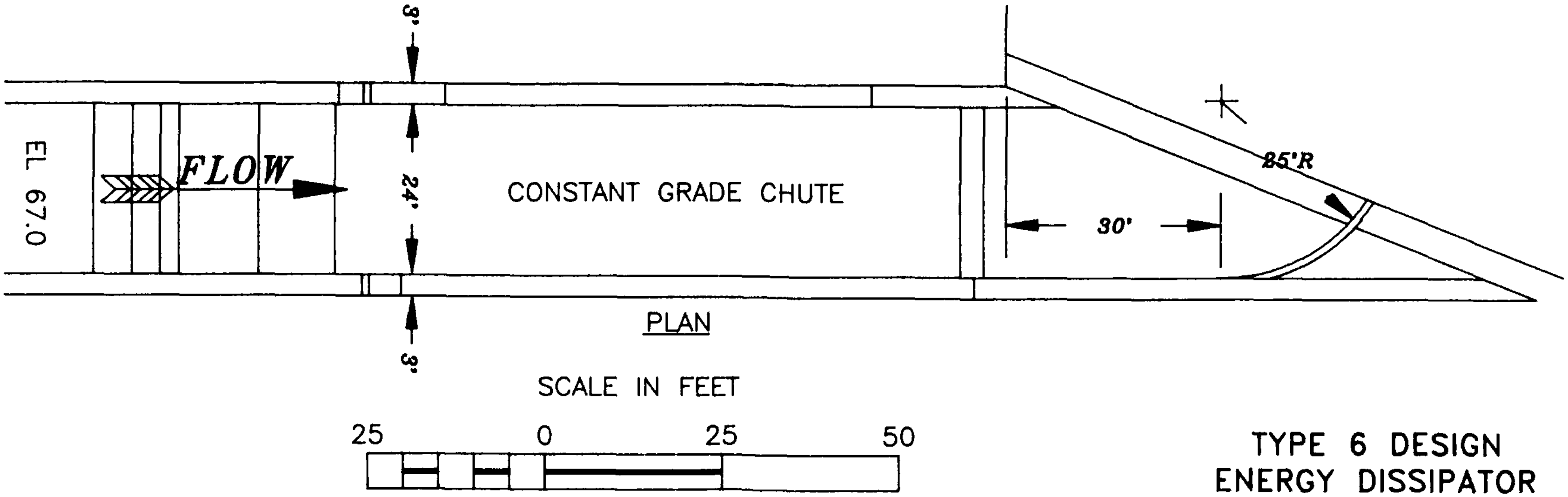

TYPE 4 DESIGN
ENERGY DISSIPATOR
50'
22.5'
5'
EL 40.0
15'
CONSTANT GRADE CHUTE
PLAN
SCALE IN FEET
25
0
25
50
3'
24'
3'
FLOW
EL 67.0
DEFLECTOR DETAIL
PROFILE
PLAN
4'
4'
5'

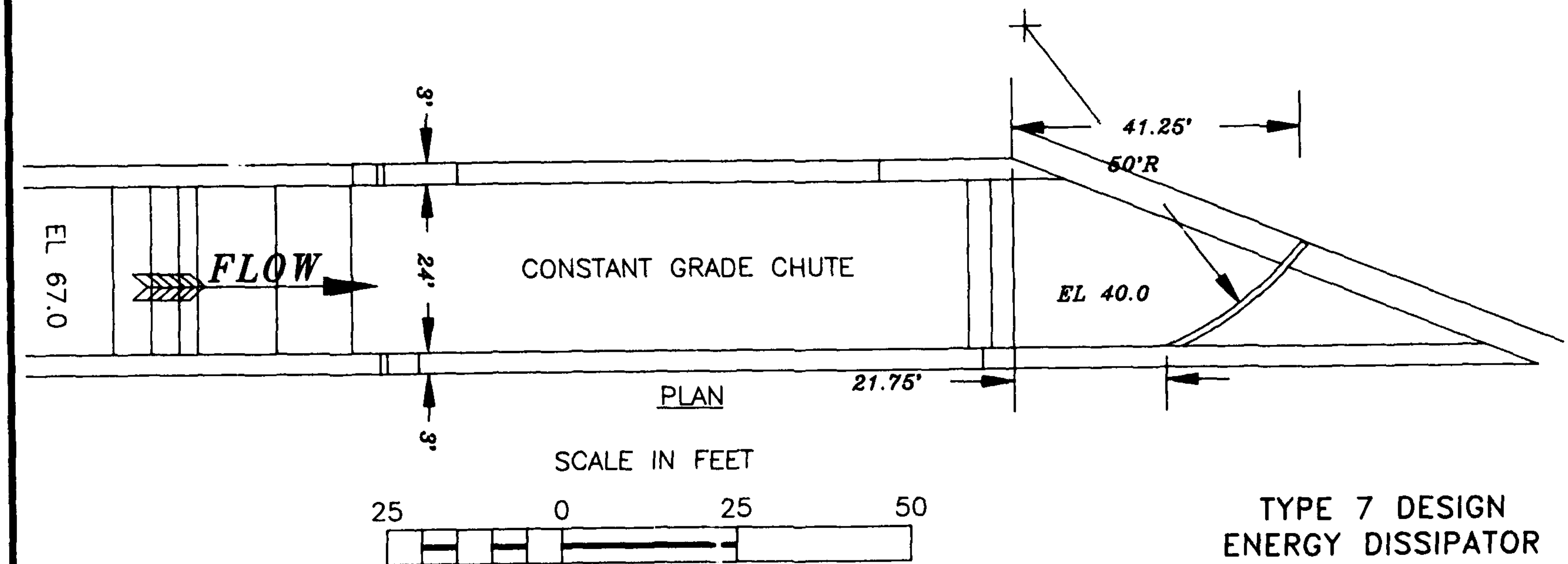
PLATE 8



TYPE 5 DESIGN
ENERGY DISSIPATOR



TYPE 6 DESIGN
ENERGY DISSIPATOR



TYPE 7 DESIGN
ENERGY DISSIPATOR

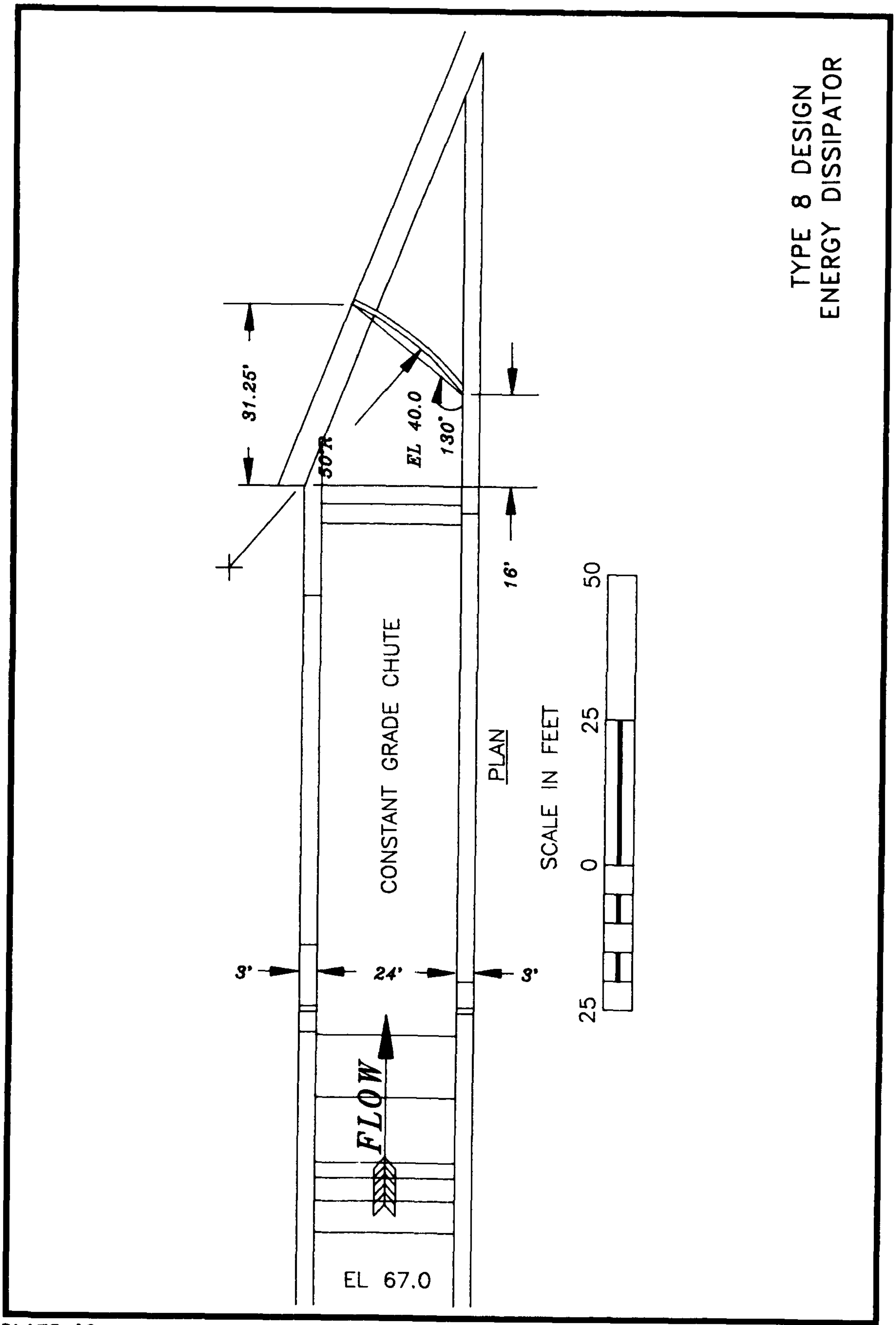
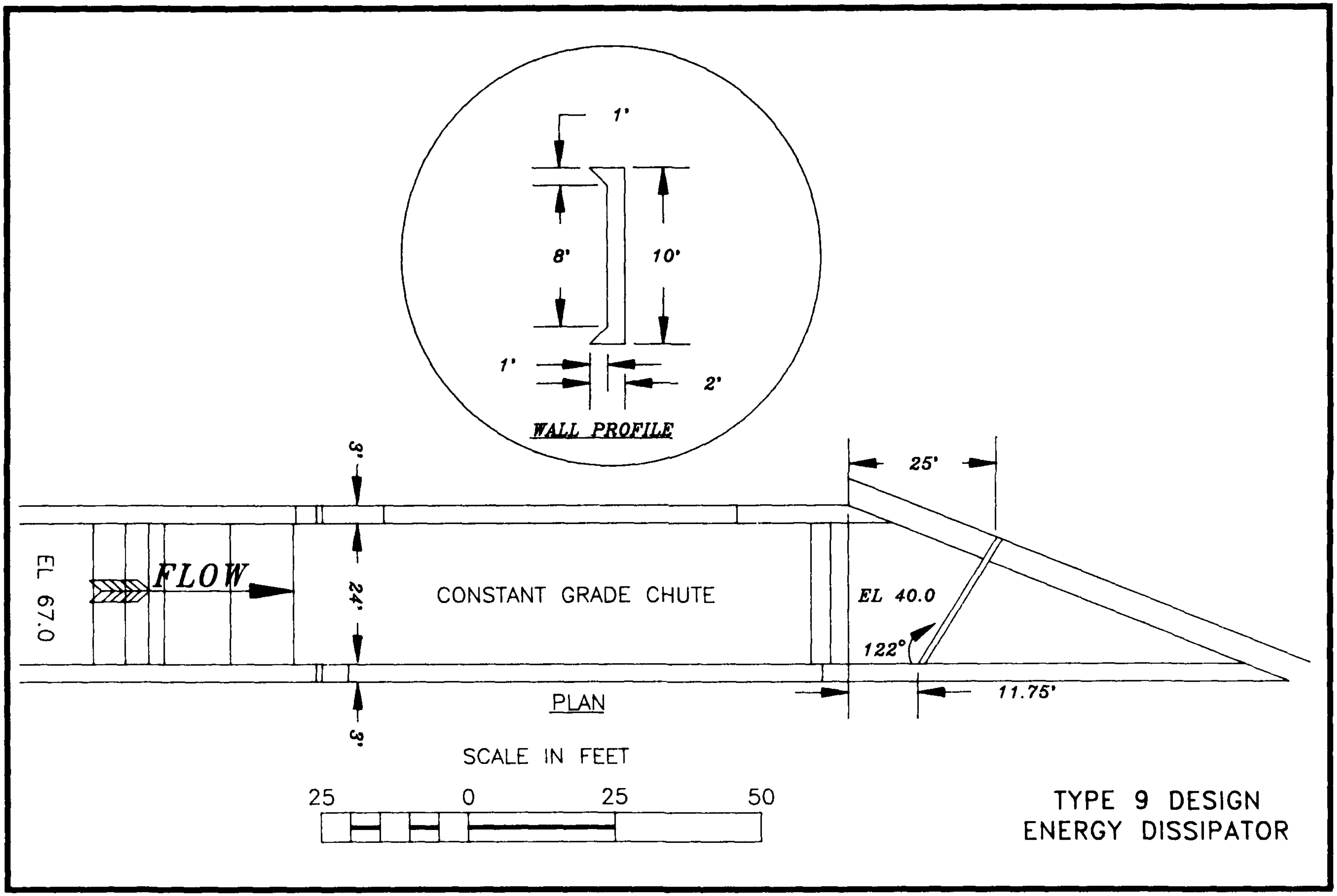
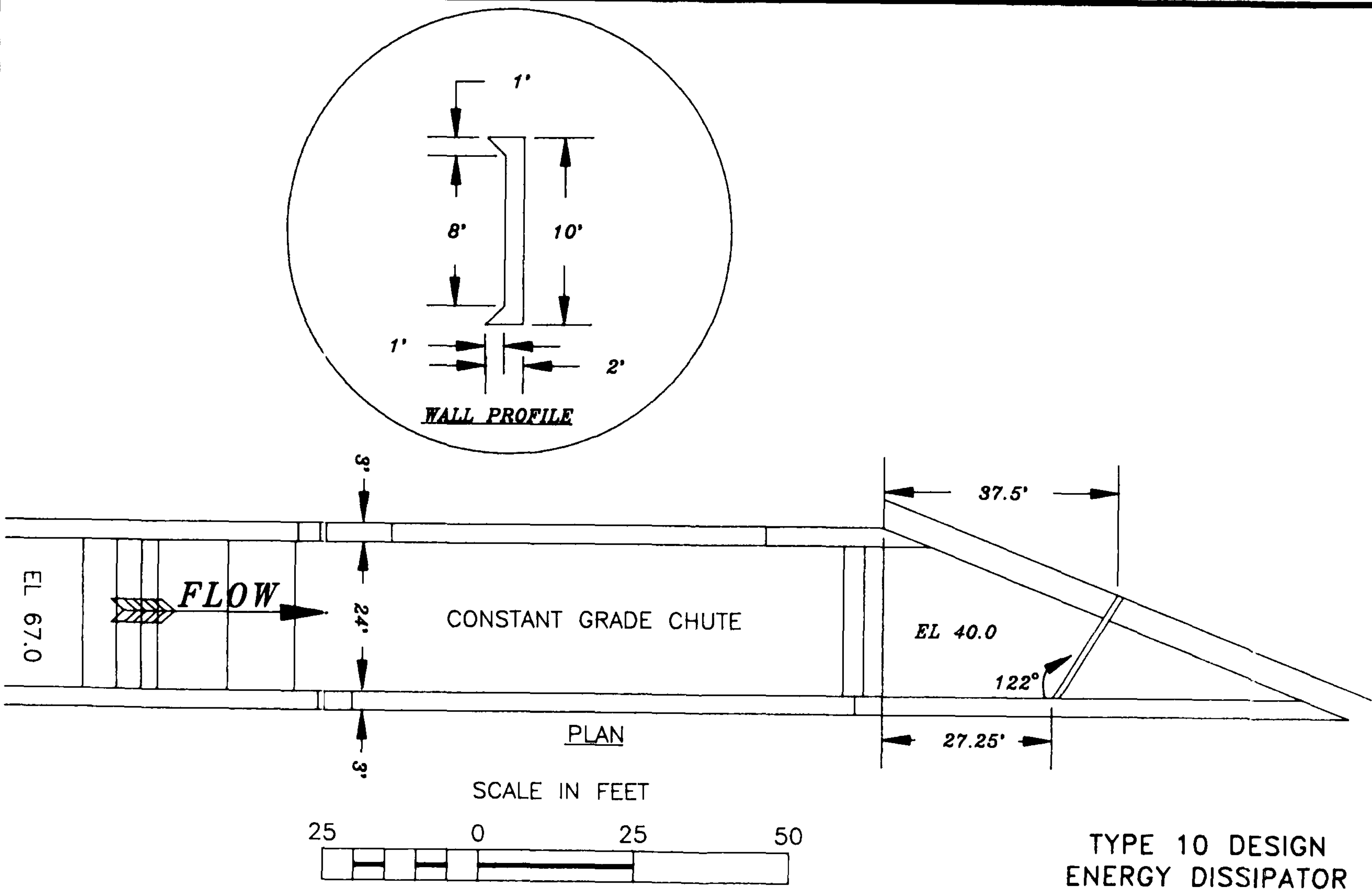

TYPE 8 DESIGN
ENERGY DISSIPATOR
31.25'
50'R
EL 40.0
130°
16'
CONSTANT GRADE CHUTE
PLAN
SCALE IN FEET
50
25
0
25
3'
24'
3'
FLOW
EL 67.0

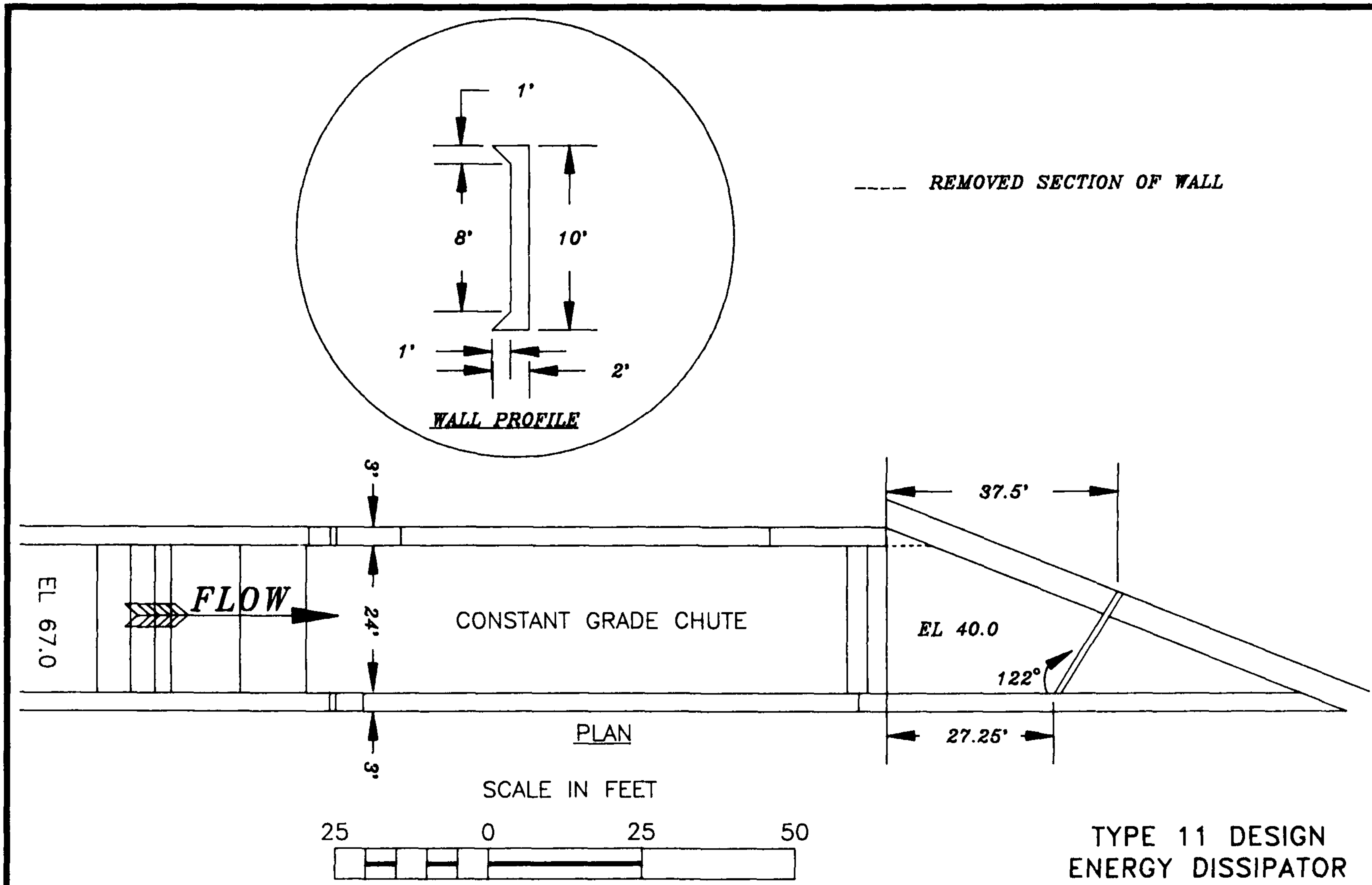
PLATE 12



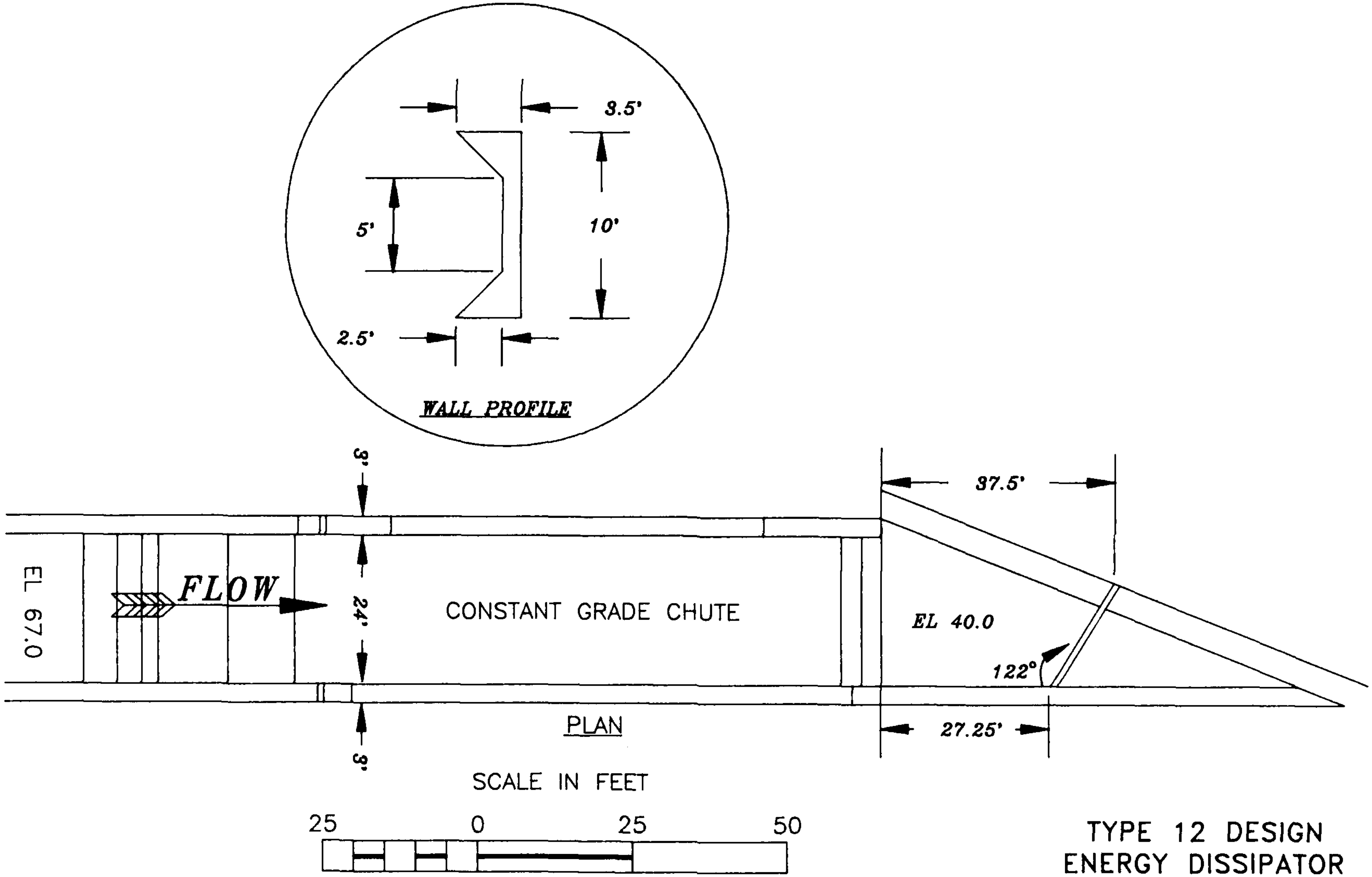
TYPE 9 DESIGN
ENERGY DISSIPATOR



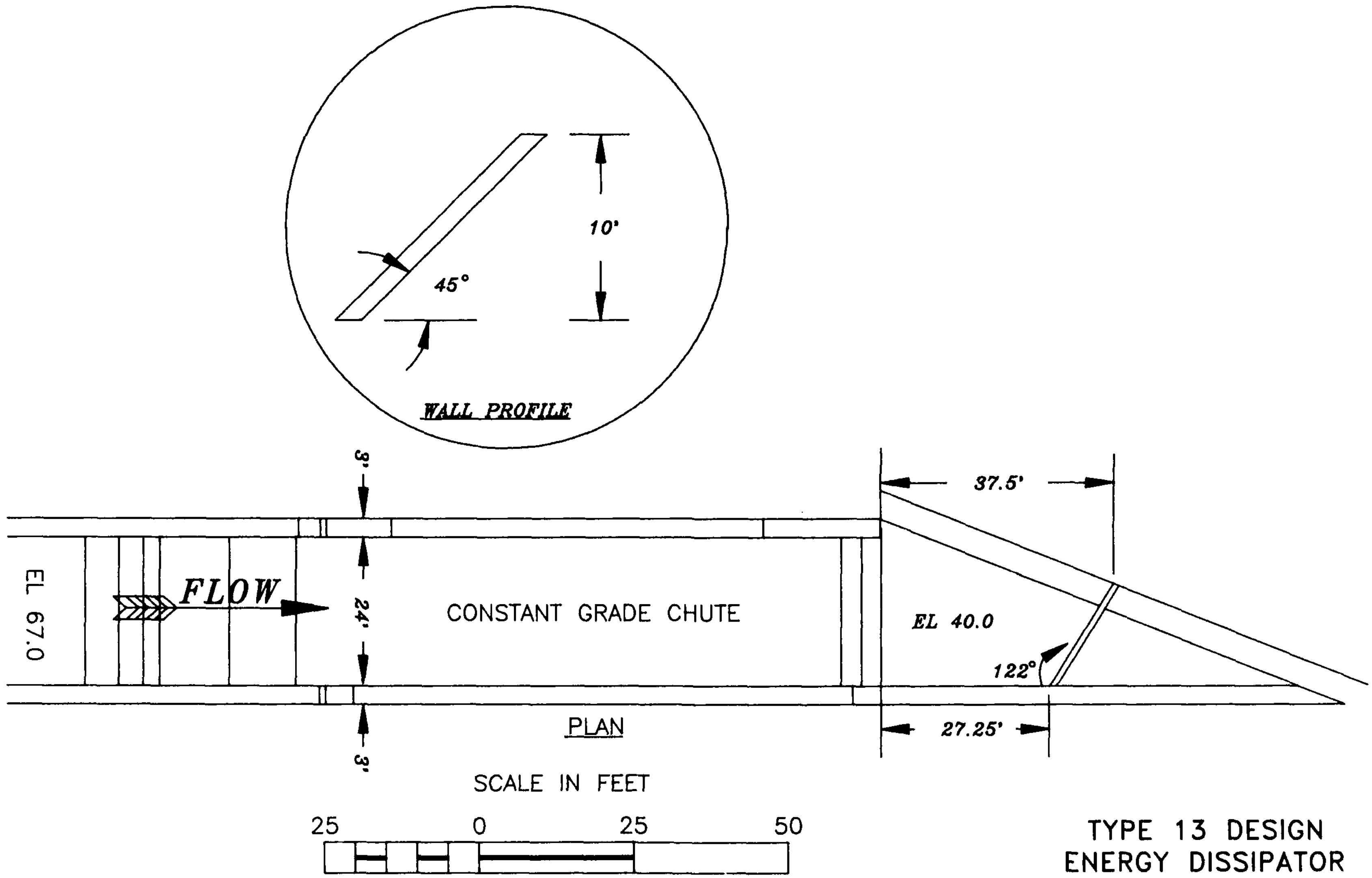
TYPE 10 DESIGN
ENERGY DISSIPATOR



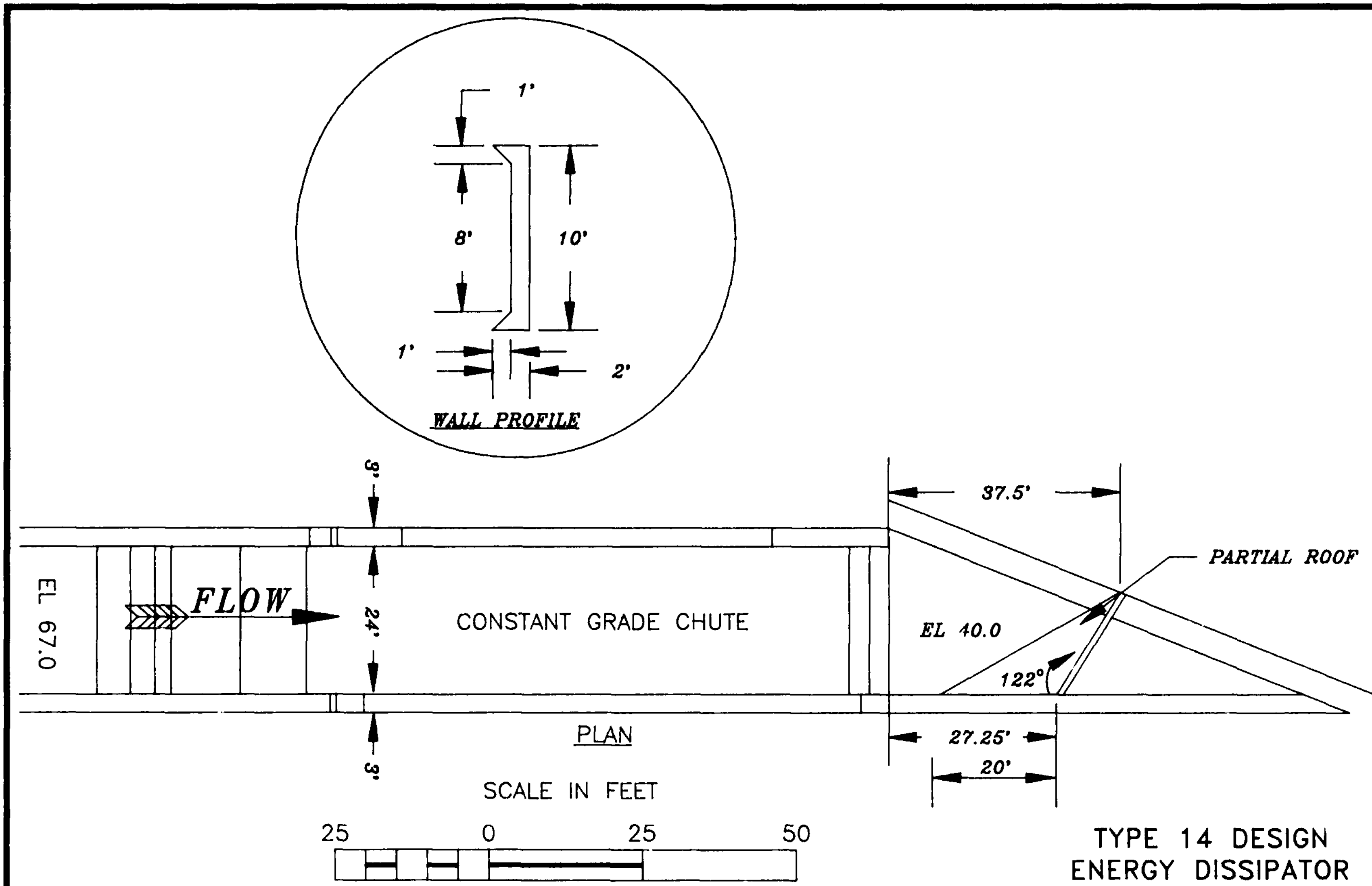
TYPE 11 DESIGN
ENERGY DISSIPATOR



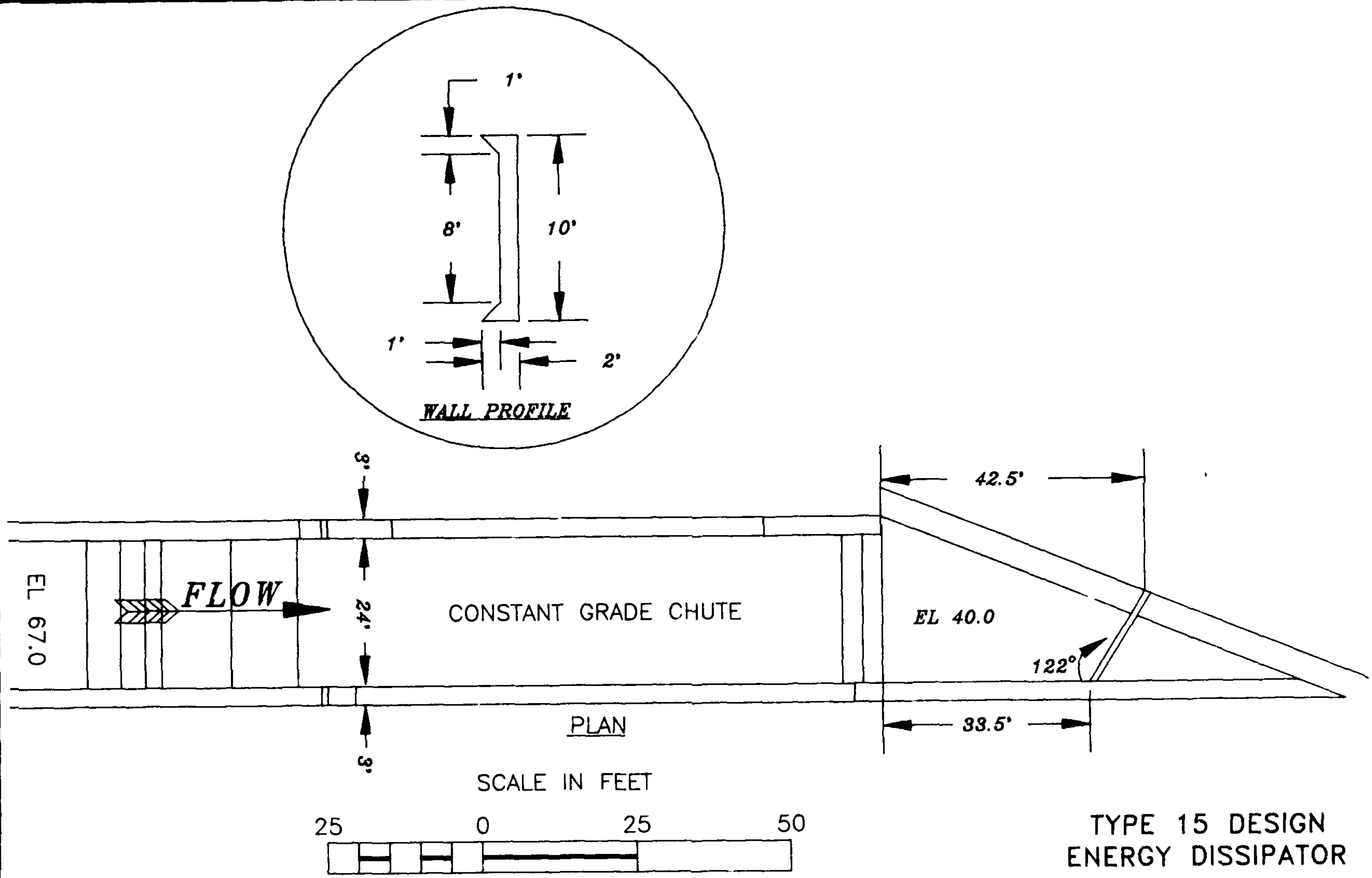
TYPE 12 DESIGN
ENERGY DISSIPATOR



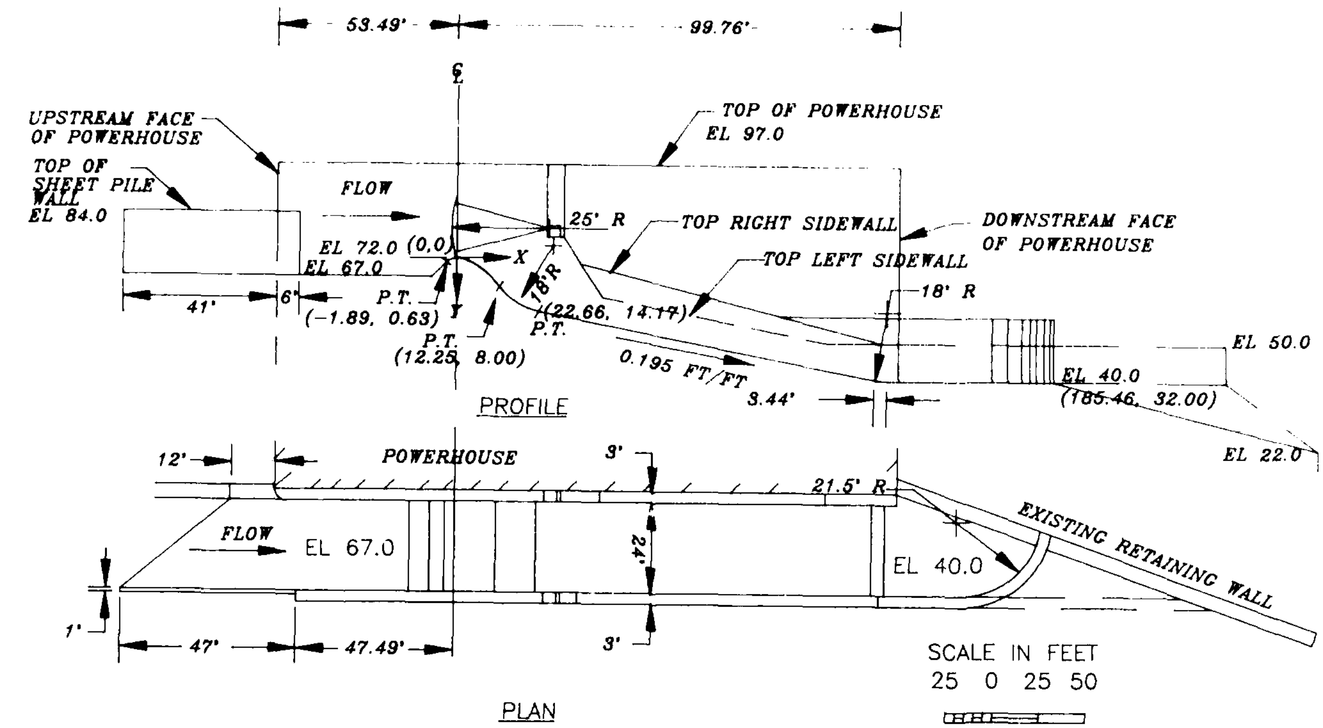
TYPE 13 DESIGN
ENERGY DISSIPATOR



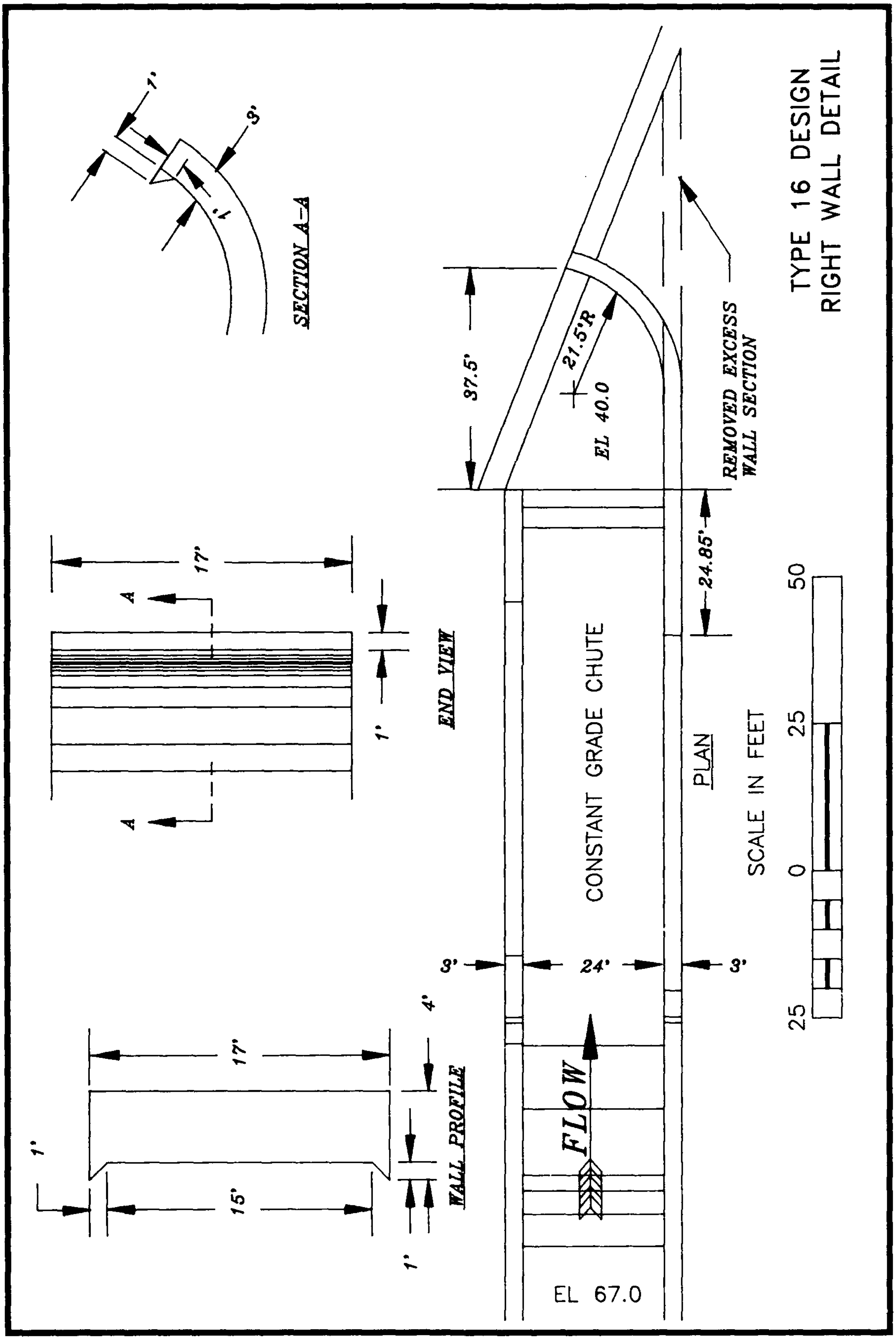
TYPE 14 DESIGN
ENERGY DISSIPATOR



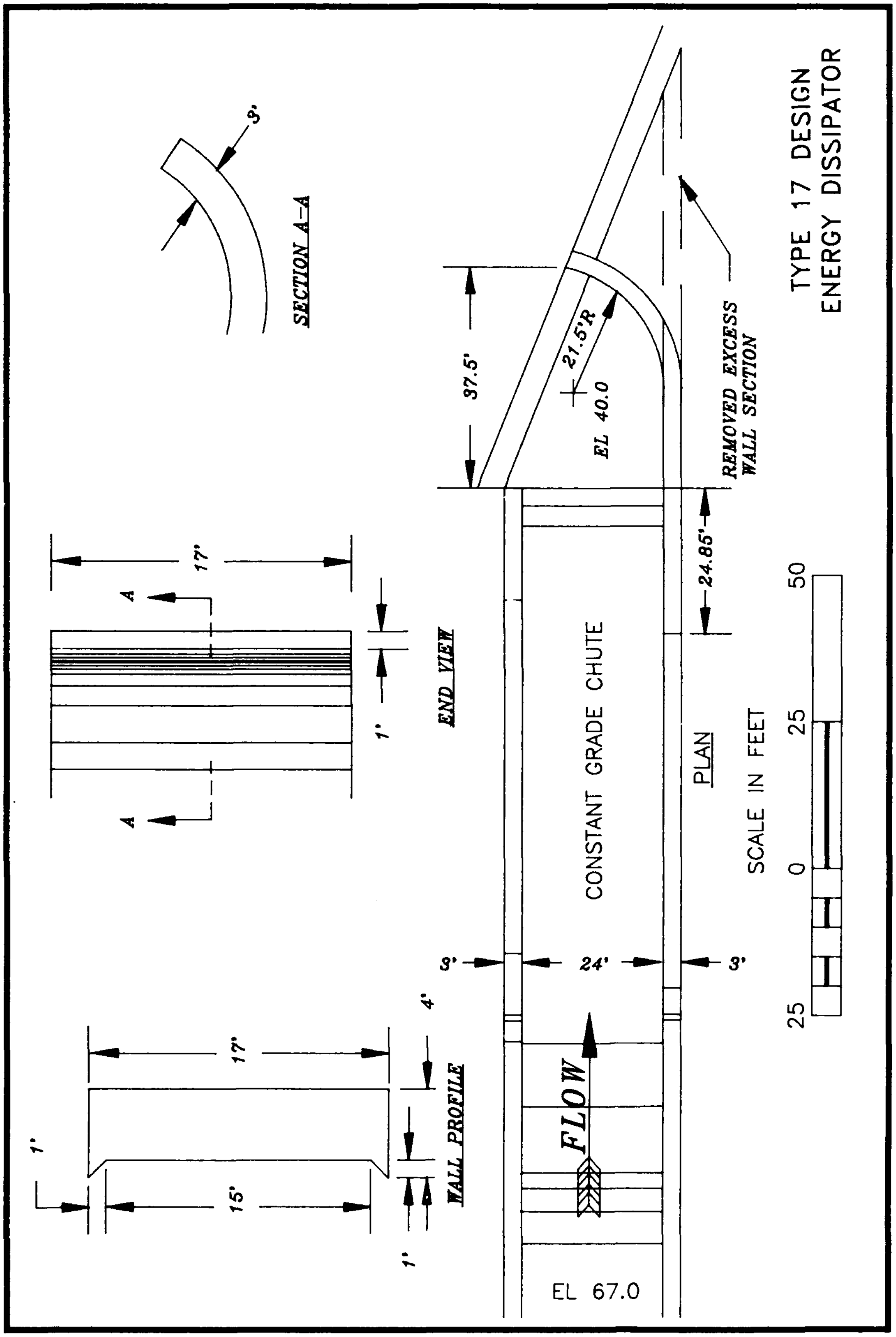
TYPE 15 DESIGN
ENERGY DISSIPATOR



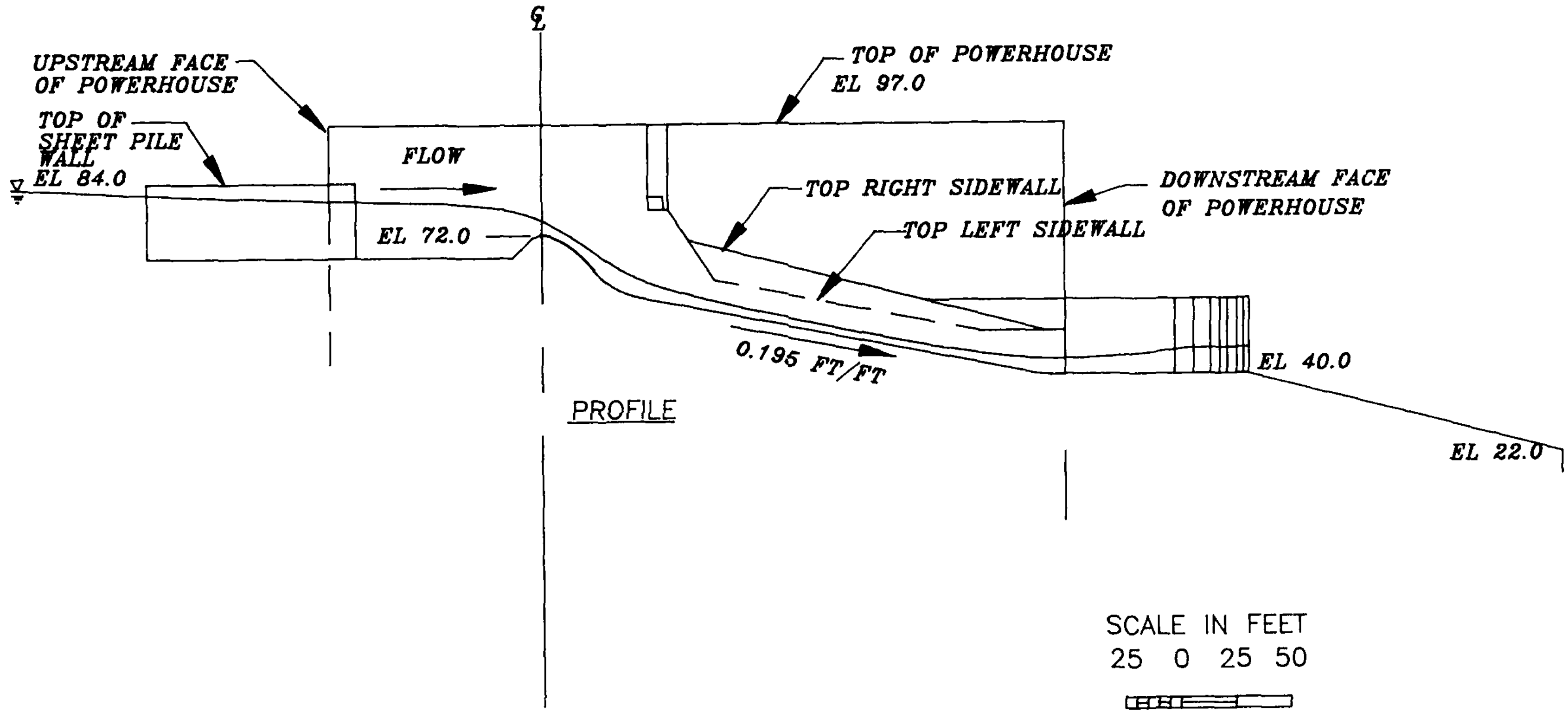
TYPE 16 DESIGN
ENERGY DISSIPATOR



TYPE 16 DESIGN
RIGHT WALL DETAIL
SECTION A–A
1'
3'
1'
37.5'
21.5'R
EL 40.0
REMOVED EXCESS
WALL SECTION
17'
A
A
END VIEW
1'
24.85'
CONSTANT GRADE CHUTE
PLAN
SCALE IN FEET
25
0
25
50
3'
24'
3'
17'
4'
1'
15'
1'
WALL PROFILE
FLOW
EL 67.0



TYPE 17 DESIGN
ENERGY DISSIPATOR
SECTION A-A
3'
37.5'
21.5'R
EL 40.0
REMOVED EXCESS
WALL SECTION
17'
A
A
1'
END VIEW
24.85'
CONSTANT GRADE CHUTE
PLAN
SCALE IN FEET
25
0
25
50
3'
24'
3'
4'
17'
1'
15'
1'
WALL PROFILE
FLOW
EL 67.0



TYPE 17 DESIGN
WATER–SURFACE PROFILE
Q = 2,640 CFS

Waterways Experiment Station Cataloging-in-Publication Data

Cooper, Deborah R.
Debris spillway and chute for Millers Ferry Powerhouse, Alabama River, Alabama / by Deborah R. Cooper ; prepared for US Army Engineer District, Mobile.
57 p. : ill. ; 28 cm. — (Technical report ; HL-92-16)
1. Spillways — Alabama — Alabama River — Models. 2. Hydraulic models. 3. Hydroelectric power plants — Alabama — Millers Ferry. I. United States. Army. Corps of Engineers. Mobile District. II. U.S. Army Engineer Waterways Experiment Station. III. Title. IV. Series: Technical report (U.S. Army Engineer Waterways Experiment Station) ; HL-92-16.
TA7 W34 no.HL-92-16