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STRUCTURAL DESIGN OF MONOLITHIC STRAIGHT DROP SPILLWAYS

PREFACE

This technical release continues the effort to develop design aids which can serve to improve the efficiency and quality of design work. The technical release deals with the structural design of monolithic straight drop spillways. The monolithic drop spillway is limited in the crest length that can be accommodated. Therefore, it is anticipated that a subsequent technical release will treat the structural design of articulated straight drop spillways. This material should be useful to both planning and design engineers. Either preliminary or detail designs may be obtained.

A draft of the subject technical release dated October 1976, was circulated through the Engineering Division and sent to the Engineering and Watershed Planning Unit Design Engineers for their review and comment.

This technical release was prepared by Mr. Edwin S. Alling, Head, Design Unit, Design Branch at Hyattsville, Maryland. He also wrote the computer program. Mr. Stanley E. Smith assisted in the preparation of the figures. Mrs. Dorothy A. Stewart typed the release.

TECHNICAL RELEASE
NUMBER 63

STRUCTURAL DESIGN OF MONOLITHIC STRAIGHT DROP SPILLWAYS

Contents

Figures

Tables

Referenced SCS Technical Materials

NEH- 6	Structural Design
NEH-11	Drop Spillways
TR -42	Single Cell Rectangular Conduits - Criteria and Procedures for Structural Design
TR -43	Single Cell Rectangular Conduits - Catalog of Standard Designs
TR -50	Design of Rectangular Structural Channels
TR -54	Structural Design of SAF Stilling Basins
SMN- 5	Flow Net Construction and Use

NOMENCLATURE

Not all nomenclature is listed. Hopefully the meaning of any unlisted nomenclature may be ascertained from that shown.

A ≡ required reinforcing steel area; height of headwall panel; bearing area
ABUTT ≡ required headwall buttress steel area at elevation of top of longitudinal sill
A(N) ≡ required steel area at location N
AHE ≡ steel area associated with MHE
AHW ≡ steel area associated with MHW
AHWA ≡ adjusted AHW
AP ≡ required steel area due to sidewall positive moment
ASW ≡ steel area associated with MSW
ASWA ≡ adjusted ASW
A_s ≡ steel area in reinforced concrete design

ATSNX ≡ required T and S steel area on the unexposed surface
ATSX ≡ required T and S steel area on the exposed surface
AV ≡ area of web steel, equals twice bar area of U stirrups
a ≡ width of one-way strip, used in apron stiffness analyses
B ≡ length of headwall panel
BACK ≡ distance used to define the wingwall footing extension back to the sidewall
BAT ≡ inside sidewall batter
BDN ≡ footing projection at downstream end of wingwall
BHE ≡ balance headwall extension
BL ≡ slope length of headwall buttress

BOTT ≡ depth of longitudinal sill below apron slab
BUP ≡ footing projection at upstream end of wingwall - section at articulation joint
BUTT ≡ depth of buttress at top of longitudinal sill
b ≡ longitudinal sill span; longitudinal apron span
CFSC ≡ coefficient of friction, soil to concrete
CFSS ≡ coefficient of friction, soil to soil
CHFT ≡ change in uplift head per foot of weighted creep distance
CLS ≡ coefficient for apron panel moments in long direction
CMHT ≡ horizontal moment coefficient for triangular panel loading
CMHU ≡ horizontal moment coefficient for uniform panel loading

CMV ≡ vertical moment coefficient
CREEPR ≡ weighted creep ratio
CSS ≡ coefficient for apron panel moments in short direction
CVHT ≡ horizontal shear coefficient for triangular panel loading
D ≡ effective depth of concrete section; diameter of reinforcing bar
DW ≡ specific energy head at the crest of the weir
DW2 ≡ DW for full flow loading
d_c ≡ critical depth
DWM*i* ≡ specific energy head for loading case *i*
E ≡ modulus of elasticity; eccentricity of VNET

EQ ≡ equivalent triangular loading
F ≡ drop through spillway = vertical distance from top of transverse sill to spillway crest
FFS ≡ reference distance for front face bending of sidewall
FLOATR ≡ safety factor against flotation
FNALL ≡ allowable normal compressive stress on horizontal cross section of headwall buttress
FPALL ≡ allowable compressive stress parallel to sloping face of headwall buttress
FPS ≡ F + S
FPSPH ≡ F + S + H
f_c ≡ compressive stress in concrete
f_c' ≡ compressive strength of concrete
f_s ≡ stress in reinforcing steel

GBF ≡ buoyant unit weight of foundation soil
GBH ≡ buoyant unit weight of earthfill, soil against headwall
GBW ≡ buoyant unit weight of earthfill, soil against sidewall and wingwall
GMF ≡ moist unit weight of foundation soil
GMH ≡ moist unit weight of earthfill, soil against headwall
GMW ≡ moist unit weight of earthfill, soil against sidewall and wingwall
GSF ≡ saturated unit weight of foundation soil
GSH ≡ saturated unit weight of earthfill, soil against headwall
GSW ≡ saturated unit weight of earthfill, soil against sidewall and wingwall
H ≡ depth of weir

HB ≡ earthfill height above top of apron at the junction of sidewall and wingwall
HBAT ≡ the height over which the inside surface of the sidewall may be battered
HBW ≡ working value of height of earthfill
HCUT ≡ depth of cutoff wall below top
HCUTN ≡ HCUT - TAP/12
HDIFF ≡ (HEAD - TAILPS) or (HBW - HWW)
HEAD ≡ upstream head acting against headwall, measured from top of apron slab
HEAD1 ≡ HEAD for no flow loading
HEAD2 ≡ HEAD for full flow loading
HEADM*i* ≡ upstream head on headwall for loading case *i*

HESTUB ≡ headwall extension stub
HESTUBN ≡ HESTUB - TSW/12
HNET ≡ resultant of the horizontal forces
HSIDE ≡ head on outside of sidewall at downstream side of headwall extension stub
HSW ≡ working value of height of section
HTOE ≡ depth of toewall below top of apron slab
HTOEN ≡ HTOE - TAP/12
HU*i* ≡ uplift head at point *i*
HWFTG ≡ headwall footing projection
HWFTGN ≡ headwall footing projection measured from the cutoff wall

HWING ≡ head on outside of sidewall at junction of sidewall and wingwall
HWW ≡ working value of water head
I ≡ moment of inertia
I_1 ≡ moment of inertia of strip
I_2 ≡ moment of inertia of elastic beam
J ≡ height of sidewall and wingwall at their junction; number of transverse apron spans
JDSN ≡ indicator, value > 0 means user specified J, value = 0 means J set from JMIN
JMIN ≡ value of J producing minimum concrete volume
j ≡ ratio used in reinforced concrete design

KHU ≡ coefficient defining distribution of horizontal shear in headwall panel due to uniform panel loading
KOF ≡ lateral earth pressure ratio, foundation soil
KOH ≡ lateral earth pressure ratio, soil against headwall
KOW ≡ lateral earth pressure ratio, soil against sidewall and wingwall
KPF ≡ passive earth pressure ratio, foundation soil
KPW ≡ passive earth pressure ratio, soil against sidewall and wingwall
k ≡ $(6/5)(EI_1/a\ell^3)$; $(3EI_1/ab^3)$; ratio used in reinforced concrete design
L ≡ length of weir = drop spillway width
LB ≡ length of drop spillway basin

LCCS ≡ transverse apron span center to center of sidewalls
LCUT ≡ out to out length of headwall plus headwall extension stubs

LEVEL ≡ distance used to locate the wingwall articulation joint with respect to the corner of the sidewall
LH ≡ distance from top of apron slab to horizontal sidewall strip under investigation
LHM ≡ distance from top of apron slab to horizontal sidewall strip with maximum steel requirement
LHMAX ≡ height at which ficticious 45° cut in sidewall intersects the top surface of the sidewall
LHS ≡ distance from top of apron slab to assumed limit of one-way horizontal bending in sidewall
LL ≡ longitudinal apron span
LS ≡ longitudinal sill span from toe of headwall buttress, if any, to upstream side of transverse sill; apron panel long span

LT ≡ transverse apron panel span
LTOT ≡ total longitudinal length of drop spillway
LV ≡ distance from face of headwall to location of assumed critical section for vertical sidewall bending
LWEB ≡ theoretical distance requiring web steel
LZERO ≡ distance to zero shear
ℓ ≡ transverse apron span; transverse sill span
M ≡ loading case under consideration; bending moment
MB ≡ moment in headwall buttress at elevation of top of longitudinal sill
MBAL ≡ balancing moment required for equilibrium at a joint

MHE ≡ headwall extension stub horizontal strip moment at junction of headwall, headwall extension stub, and sidewall
MHF ≡ longitudinal apron span fixed end moment at the headwall with assumed fixed support at the toewall
MHS ≡ longitudinal apron span fixed end moment at the headwall with assumed simple support at the toewall
MHT ≡ horizontal moment in headwall panel due to triangular panel loading
MHU ≡ horizontal moment in headwall panel due to uniform panel loading
MHW ≡ headwall horizontal strip moment at junction of headwall, headwall extension stub, and sidewall
MHWA ≡ adjusted MHW
ML ≡ longitudinal sill cross section dimension
ML1 ≡ apron panel one-way moment in the long direction
MLD ≡ apron panel design moment in the long direction

MP ≡ assumed maximum positive sidewall moment
MS ≡ statical sidewall moment; equivalent moment
MS1 ≡ apron panel one-way moment in the short direction

MSD ≡ apron panel design moment in the short direction
MSW ≡ sidewall horizontal strip moment at junction of headwall, headwall extension stub, and sidewall
MSWA ≡ adjusted MSW
MTF ≡ longitudinal apron span fixed end moment at the toewall
MV ≡ vertical moment in headwall panel
P ≡ required steel perimeter
PALLOW ≡ allowable bearing pressure

PAVER ≡ average bearing pressure
PBUTT ≡ required perimeter of headwall buttress steel at elevation of top of longitudinal sill
PDN ≡ bearing pressure at downstream limit of spillway
PL ≡ net pressure on upstream end of horizontal sidewall strip under investigation
PN ≡ net pressure on a transverse apron span section
P(N) ≡ required steel perimeter at location N
PNET ≡ net pressure on headwall extension stub strip
PT ≡ maximum value of a triangular loading, symbol used with respect to various structural elements
PU ≡ uniform loading symbol used with respect to various structural elements
PUP ≡ bearing pressure at upstream limit of spillway

p_t ≡ temperature and shrinkage steel ratio
QUANT ≡ volume of spillway or volume of associated wingwalls
q ≡ uniform loading on apron strips
q_o ≡ $(11/10)(q\ell/a)$; $(3/8)(qb/a)$
R ≡ strip reaction = sill loading per strip
R ≡ reaction; reduction factor
RHF ≡ longitudinal apron span reaction associated with MHF
RHS ≡ longitudinal apron span reaction associated with MHS
RI ≡ transverse apron span interior support reaction
R_o ≡ varying load on sill

RM ≡ ratio of moment with yielding elastic supports to moment with non-yielding supports
RS ≡ transverse apron span reaction at the sidewalls
RTF ≡ longitudinal apron span reaction associated with MTF
RTS ≡ longitudinal apron span reaction associated with MHS
S ≡ height of transverse sill above top of apron slab; spacing of reinforcing steel
SDOWN ≡ sum of all downward forces
SL ≡ longitudinal sill cross section dimension
SLIDER ≡ safety factor against sliding
S(N) ≡ maximum steel spacing at location N

SR ≡ stiffness ratio, used in apron analyses
SRMIN ≡ minimum acceptable value of SR
SS ≡ apron panel short span
ST ≡ transverse sill cross section dimension
SUM ≡ sum of required depth of cutoff wall and toewall, used in creep analyses
SUP ≡ sum of all upward forces
SWFTG ≡ sidewall footing projection
SWLDRN ≡ sidewall design switch indicating presence of sidewall drains
T ≡ required thickness
TAIL ≡ tailwater head, measured from top of sill

TAIL2 ≡ TAIL for full flow loading
TAILM*i* ≡ tailwater head for loading case *i*
TAILPS ≡ TAIL + S
TAP ≡ thickness of apron slab
TBLS ≡ thickness of headwall buttress and longitudinal sill
TCUT ≡ thickness of cutoff wall
THW ≡ thickness of headwall
THV ≡ thickness of headwall required by vertical bending, used to establish initial value of apron thickness
TL ≡ longitudinal sill cross section dimension
TSB ≡ thickness of sidewall at its bottom

TSV ≡ thickness of sidewall required by vertical bending, used to establish initial value of apron thickness
TSW ≡ thickness of sidewall at its top
TT ≡ transverse sill cross section dimension
TTOE ≡ thickness of toewall
TWCD ≡ total weighted creep distance
TWF ≡ thickness of wingwall footing
TWT ≡ thickness of wingwall toewall
TWW ≡ thickness of wingwall
t ≡ thickness of strip = thickness of apron slab
t_m ≡ minimum tailwater by NEH-11

U ≡ allowable bond stress
UL ≡ longitudinal sill cross section dimension
UT ≡ transverse sill cross section dimension
u ≡ flexural bond stress in concrete
V ≡ shear
VB ≡ shear in headwall buttress at elevation of top of longitudinal sill

VC ≡ shear concrete section can resist without web steel
VFTG ≡ spillway sidewall footing adjustment volume

VHT ≡ horizontal shear in headwall panel due to triangular panel loading
VHU ≡ horizontal shear in headwall panel due to uniform panel loading
VL ≡ longitudinal sill cross section dimension
VNET ≡ resultant of the vertical forces
VT ≡ transverse sill cross section dimension
VV ≡ vertical shear in headwall panel
VWING ≡ volume of wingwalls esclusive of VFTG
v ≡ shearing stress in concrete
W ≡ load brought to transverse sill by the longitudinal sills
WDES ≡ perpendicular distance from the sidewall to the point where the outside edge of the wingwall footing intersects the plane of the downstream end section

WEXT ≡ perdendicular distance from sidewall to the point on the outside edge of the wingwall footing that is in the plane of the articulation joint
WPROJ ≡ wingwall projection, the perpendicular distance from the sidewall to the farthest point on the outside edge of the wingwall footing
WTWT ≡ perpendicular distance from the sidewall to the point of intersection of wingwall toewall and plane of downstream end section
WWLB ≡ perpendicular distance from the plane of the downstream end section to the point where the wingwall footing extended backward would intersect the outer edge of the sidewall
w ≡ uniform loading on apron
X ≡ height of headwall panel strip for shear computations; distance from articulation joint to section of wingwall under investigation; reference distance
XIN ≡ distance against the sidewall intersects the top of the sidewall
XL ≡ longitudinal sill cross section dimension
XT ≡ transverse sill cross section dimension

x ≡ distance from origin
Y ≡ vertical reference distance
YDN ≡ vertical distance from earth surface on downstream side of headwall extension stub to section at YHES
YHES ≡ vertical distance from top of headwall extension stub to assumed critical section for horizontal bending of headwall extension stub
YL ≡ longitudinal sill cross section dimension
YT ≡ transverse sill cross section dimension
YUP ≡ vertical distance from assumed earth surface on upstream side of headwall to section at YHES
y ≡ vertical displacement
Z ≡ distance from moment center to VNET

ZNS ≡ slope parameter for the earthfill adjacent to the sidewall in the direction normal to the wingwall

ZNW ≡ slope parameter for the earthfill adjacent to the wingwall in the direction normal to the wingwall

ZPS ≡ slope parameter for the earthfill adjacent to the sidewall in the direction parallel to the sidewall

ZPSE ≡ effective slope parameter for the earthfill adjacent to the sidewall

ZTOP ≡ slope parameter for the top surface of the sidewall

β ≡ $(k/4EI_2)^{1/4}$

δ ≡ vertical displacement at longitudinal sill locations

TECHNICAL RELEASE
NUMBER 63

STRUCTURAL DESIGN OF MONOLITHIC STRAIGHT DROP SPILLWAYS

Introduction

This technical release is concerned with the structural design of monolithic straight drop spillways. The work is based on the hydraulic criteria and expands on the structural procedures of National Engineering Handbook, Section 11, "Drop Spillways." The material presented herein treats the structural design of drop spillways having the general layout indicated by Engineering Standard Drawing ES-111, sheet 2, contained in NEH-11. The material herein does not include hydraulic proportioning which must preceed structural design. It is assumed these structural designs will be obtained from computers although the basic approach is independent of computer usage.

A computer program was written in FORTRAN for IBM equipment to perform these straight drop spillway designs. The program operates in two modes. It will execute preliminary designs to aid the designer in planning or in selecting the spillway he desires to use in final design. The program will also execute detail designs of specified spillways. Concrete thicknesses and distances are determined and steel requirements, in terms of minimum area and maximum spacing, are evaluated at various locations. These locations, together with associated discussions, should be sufficiently numerous to adequately determine steel requirements throughout the structure. In several instances beam web steel requirements are indicated. Actual steel sizes and layouts are not selected, these are the perogative of the designer.

This technical release documents the criteria and procedures used in the computer program, explains the input data required to obtain a design, and illustrates computer output for preliminary and detail designs. Assumptions used in the various analyses are stated throughout the technical release in appropriate locations. If the assumptions differ significantly from actual field conditions at a particular site — the designer should either not use the program, make suitable modifications in the affected elements, or accept only those results that are unaffected by the unsatisfied assumptions.

At the present time designs may be obtained by requests to the

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Input information which must be provided for each design run, is discussed under the section, "Computer Designs, Input."

Type of Straight Drop Spillway

Monolithic straight drop spillways are the only structures dealt with herein. The monolithic drop spillway is limited in the crest length and/or basin length, that can be accommodated. These limits may be due to structural requirements, problems associated with expansion or contraction from temperature changes and shrinkage, or the probability of significant deviation from assumed uniform foundation conditions. As these limits usually involve engineering judgement, no arbitrary maximum crest length or basin length is imposed in the program. A subsequent technical release and computer program will treat the structural design of articulated straight drop spillways. Articulated drop spillways will permit essentially unlimited crest length by constructing adjacent components that are basically structurally independent elements.

Spillways are assumed symmetrical about their longitudinal centerline in both construction and loading. Each spillway is designed for the loading conditions described in the next section and each must satisfy flotation (uplift) requirements and sliding requirements. Figure 1 is a definition sketch for these spillways and presents basic nomenclature. The sketch shows a longitudinal sill with a thickness TBLS. The sill shown consists of three parts external to the apron slab. These are a raised portion of height S above the apron, a dropped portion of depth BOTT below the apron, and a headwall buttress of length BUTT from the headwall. A given longitudinal sill may not have a dropped portion and it may not have a buttress. Longitudinal sill configuration depends on headwall and/or apron stiffness requirements as discussed subsequently. Further, depending on either designer preference or program selection for minimum concrete, a particular spillway design may have no longitudinal sill or it may have one or two longitudinal sills.

Figure 1 illustrates an idealized spillway and does not show floor blocks, fillets on toewall and cutoff wall, headwall extensions beyond the headwall extension stub HESTUB, or wingwalls. The headwall extensions are articulated from the spillway proper at the distance HESTUB from the inside face of the sidewall. Headwall extensions beyond HESTUB are therefore structurally independent of the rest of the spillway and act as water resistant diaphragms. The designer must provide headwall extensions of adequate length beyond the headwall extension stubs and must design them for controlling conditions.

Wingwalls are omitted from Figure 1 for clarity and because the wingwalls and spillway proper are designed to act essentially independently of each other. The wingwall is articulated from the spillway sidewall. Hence each wall acts as a vertical cantilever at the junction of sidewall and wingwall. The wingwalls with their footings are not included in the stability analyses of the basin proper. Figure 2 gives the wingwall layout. The junction of sidewall and wingwall is level for a distance of 6 inches parallel to the sidewall and a distance shown as LEVEL parallel to the wingwall. The distance LEVEL locates the articulation joint and depends on wingwall and sidewall thicknesses. This distance is discussed later.

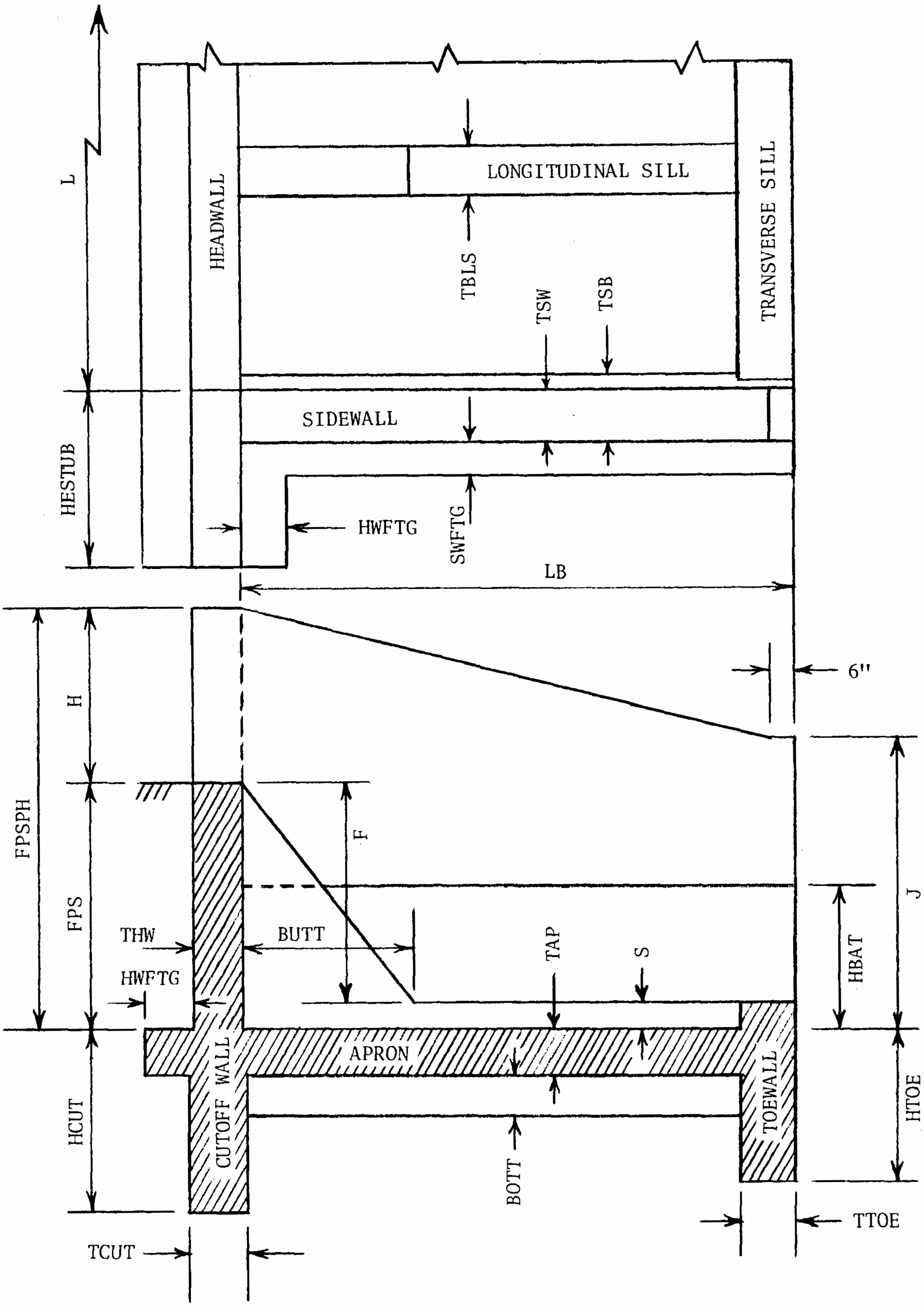


Figure 1. Straight drop spillway definition sketch.

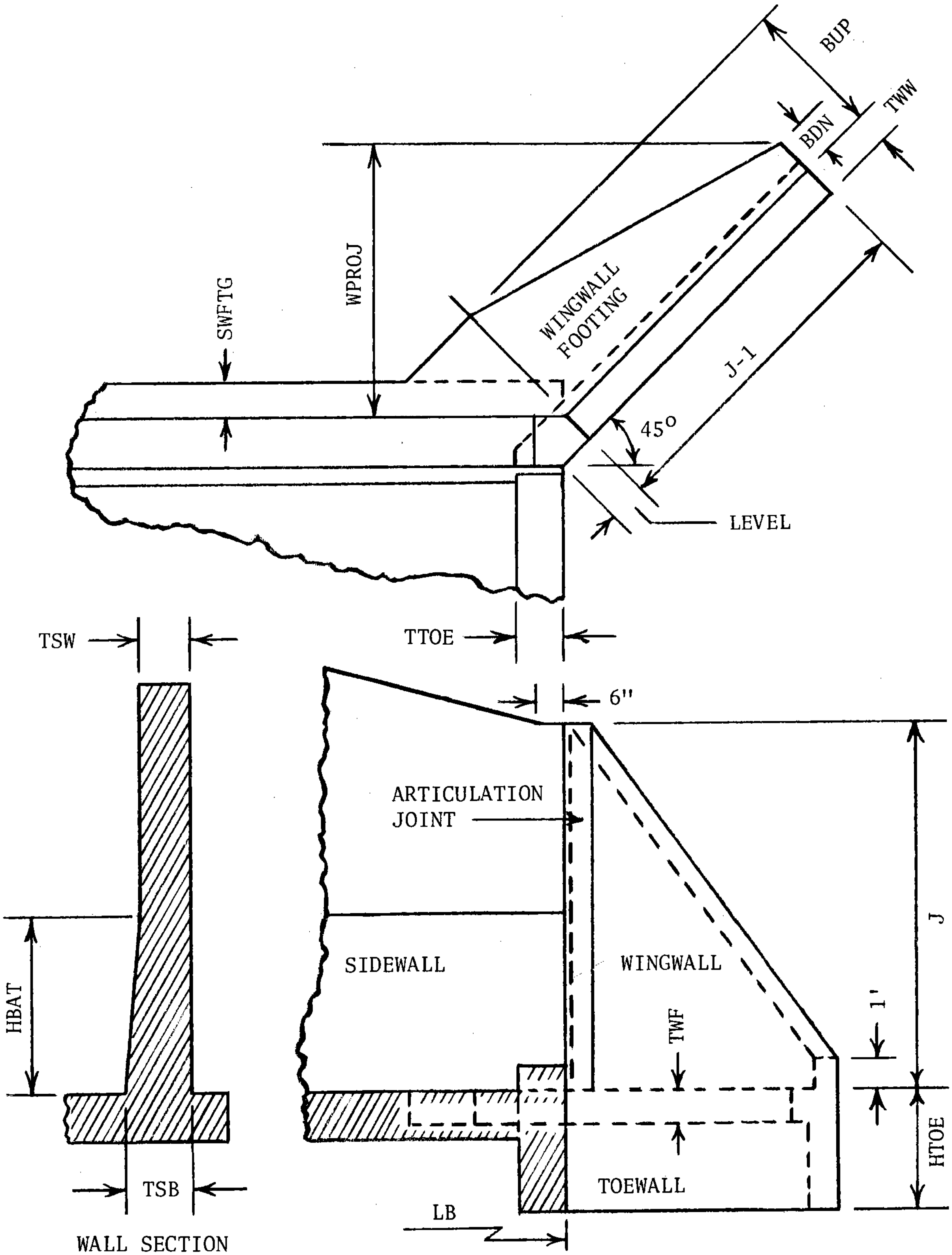


Figure 2. Wingwall layout.

Loading Conditions

Drop spillway design requires the investigation of a number of loading cases. Figure 3 shows a typical loading case. The surface of the earthfill against the sidewall varies linearly as a function of the slope parameter, ZPS. The height of earthfill at the junction of sidewall and wingwall is HB. The surface of the earthfill against the headwall is at crest elevation - with one loading exception noted later. The downstream channel surface is taken at the elevation of the bottom of the apron slab.

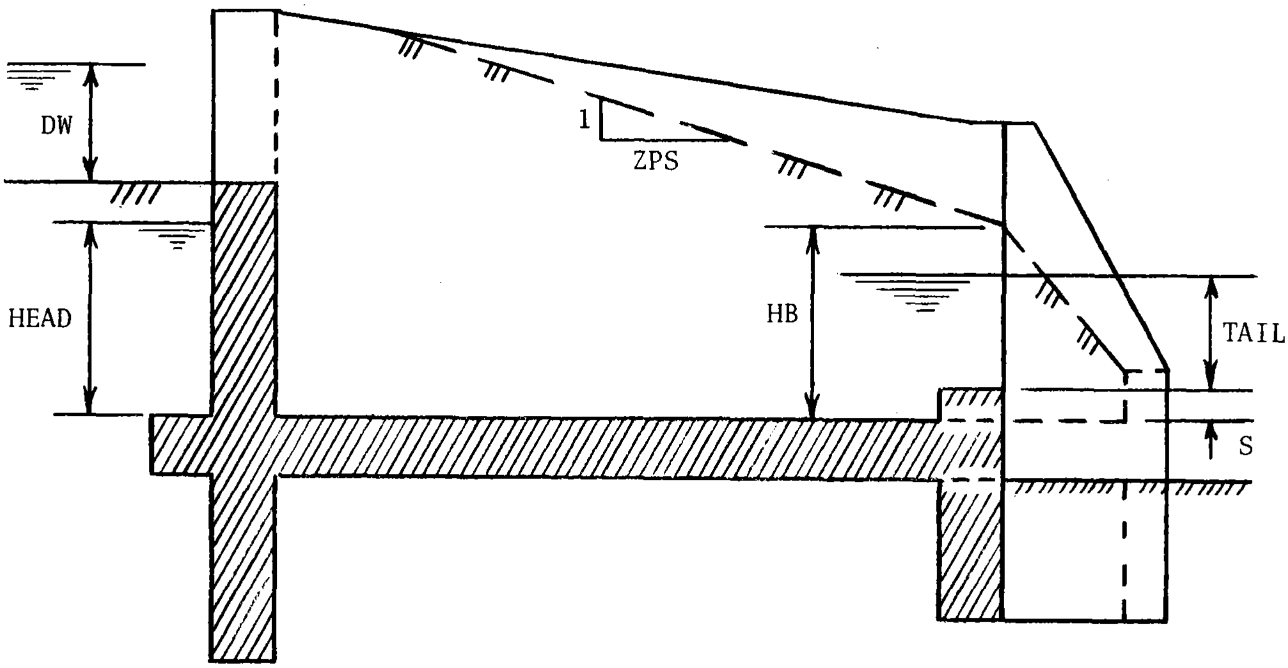


Figure 3. Typical loading case and variation of earthfill.

The water parameters for this representative loading are

DW ≡ specific energy head at the crest of the weir, in ft

HEAD ≡ upstream head acting against headwall, measured from top of apron slab, in ft

TAIL ≡ tailwater head, measured from top of sill, in ft

Structurally, it is convenient to reference the tailwater from the top of the apron. Thus let

TAILPS = TAIL + S.

If HEAD is less than TAILPS, then HEAD is reset to

HEAD = TAILPS

to prevent reverse flow situations. If HEAD exceeds the headwall height, FPS, (see Figure 1) then HEAD is reset to

HEAD = DW + FPS.

Submerged flow, that is

TAILPS > FPS

is a permissible loading case.

Water and Earthfill Loadings, Cases Considered

In all, eight loading cases are included in the design of these spillways. Not all loadings are considered for all functions. Figure 4 indicates the full range of loading cases involved. The various loadings are described below in order of investigation.

M = 1, Load Condition No. 2. This is the full flow loading, see Figure 4. The water parameters are DW2, HEAD2, and TAIL2. They are ordinarily the maximum values treated.

M = 2 through 5, Intermediate Load Conditions. These four loadings represent intermediate flows between the two basic flow cases of full flow and no flow. Any of these four loadings may be critical for a particular function depending on given values of DW, HEAD, and TAIL.

M = 6, Load Condition No. 1. This is the no flow loading, see Figure 4. There is no flow over the weir. The head on the headwall is HEAD1. The tailwater surface is taken at the elevation of the bottom of the apron slab. Prior to design, the apron slab is assumed 12 inches thick for this loading and also for M = 7 and 8 below. Thus TAILPS = -1.0.

M = 7, Construction Condition. This construction condition, with ground water surface taken at the elevation of the bottom of the apron slab, may be critical for some functions, see Figure 4. Thus HEAD = TAILPS = -1.0.

M = 8, No Backfill Condition. This is a construction condition prior to placing any backfill. The structure simply rests on the foundation. This loading may control some transverse steel requirements in the apron slab.

Flotation Requirements

The total weight of the drop spillway plus all downward forces acting on it must exceed the uplift forces by a suitable safety factor under all conditions of loading. The flotation safety factor, FLOATR, is selected by the user. Headwall footing projections are increased, and/or sidewall footing projections are provided, when required to develop necessary additional downward forces.

Sliding Requirements

The horizontal resisting forces that can be mobilized must exceed the horizontal driving forces acting on the spillway in a downstream direction by a suitable safety factor under all conditions of loading. The sliding safety factor, SLIDER, is selected by the user.

The forces resisting sliding are the frictional resistance between the spillway and the foundation, the frictional resistance between the sidewalls and the earthfill, the passive resistance of the effective earthfill downstream of the headwall extension stub, the passive resistance of the channel material downstream of the toewall, and the resisting hydrostatic pressure of any tailwater. Frictional forces against the sidewalls (or against planes parallel to the sidewalls) are neglected as being extremely unreliable. The passive resistance of the channel material downstream of the toewall is neglected since much of it may be scoured away. The force tending to induce sliding is caused by lateral earth and hydrostatic water pressures acting against the upstream side of the spillway.

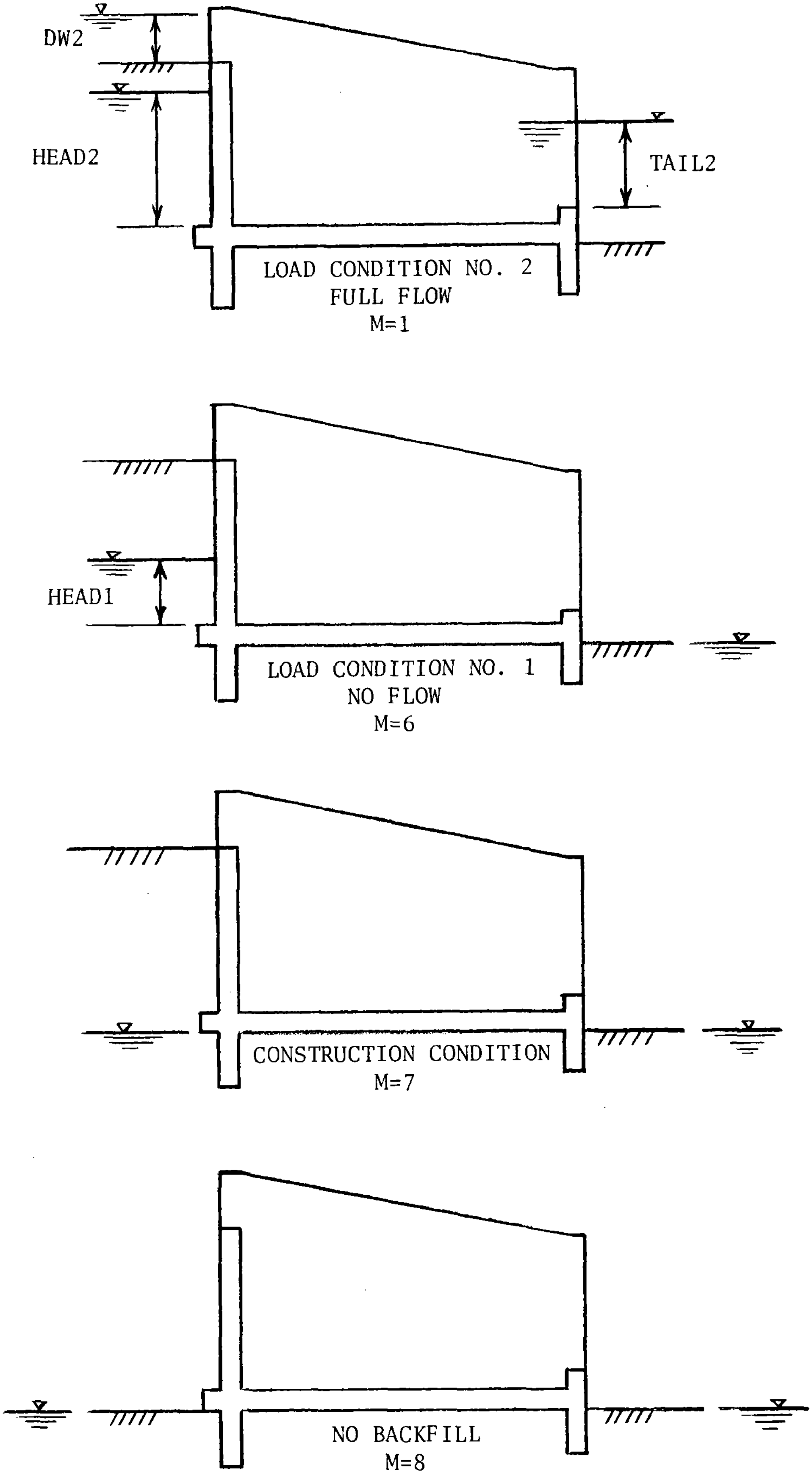


Figure 4. Loading cases considered.

Design Parameters

There are some twenty-eight independent parameters, exclusive of water parameters, involved in the structural design of these straight drop spillways. All parameters are classified as either primary parameters or secondary parameters. Values for primary parameters must be supplied by the user for each design run. Secondary parameters will be assigned default values if values are not supplied by the user. The methods of supplying parameter values are discussed under the section, "Computer Designs, Input."

Primary Parameters

H $\equiv$ depth of weir, in ft
F $\equiv$ drop from crest of weir to top of transverse sill, in ft
S $\equiv$ height of transverse sill above top of apron, in ft
J $\equiv$ height of sidewall and wingwall at their junction, in ft
L $\equiv$ crest length = stilling basin width, in ft
LB $\equiv$ length of basin, in ft

Secondary Parameters

Default values have been assigned the secondary parameters to aid the designer in obtaining sound trial designs. Use of default values may result in an overly conservative (or unconservative) design. The designer is urged to evaluate the secondary parameter values he wishes to use.

It is convenient to separate secondary parameters into two groups, water parameters and other parameters. The other parameters and their default values are listed in Table 1. Usage of these parameters is explained where first encountered. Water parameters are presented below.

Water Parameters

Three optional methods of specifying water parameter values are available to the user. These range from values completely defined by default to values completely user supplied (except that DW = 0.0 and TAIL = -(S + 1.0) for M = 6 as shown in Figure 4). Table 2 shows the three options.

In Option 1, all values are established by default. The basic default values are taken as

DW2 = H

TAIL2 = 1.17H   from $t_{min} = 7/4(d_c)$

HEAD1 = (S + 0.25F)   from NEH-11, Table 4.1

HEAD2 = HEAD1 + TAIL2

The remaining default values are built from the relation that head varies as the two-thirds power of discharge and the assumption that DW, HEAD, and TAIL all vary similarly.

In Option 2, the user supplies values for DW2, HEAD2, TAIL2, and HEAD1. The remaining values are constructed as in the first option.

In Option 3, the user supplies all values except DW and TAIL for M = 6. With this option, the user has essentially complete control of the loading cases (M = 1 through 6) he wishes to investigate. As an example, for submerged flow cases, the user may set DW values in correspondence with the HDIFF = HEAD - TAILPS values he believes applicable to his design situation. Thus he can effectively control values of HEAD used in design.

Table 1. Secondary parameters and default values

Parameter		Default Value
CREEPR	≡ weighted creep ratio	5.0
FLOATR	≡ safety factor against flotation	1.5
SLIDER	≡ safety factor against sliding	1.0
BAT	≡ inside sidewall batter, in inches per ft of height	0.0
SWLDRN	≡ sidewall design switch indicates no/yes presence of sidewall drains	0.
HB	≡ earthfill height above top of apron at downstream end of basin, in ft	*
ZPS	≡ slope parameter for earthfill adjacent to the sidewall in the direction parallel to the sidewall	2.0
HTOE	≡ depth of toewall below top of apron, in ft	4.0
TTOE	≡ thickness of toewall, in inches	10.0
CFSS	≡ coefficient of friction, soil to soil	0.55
CFSC	≡ coefficient of friction, soil to concrete	0.35
KOH	≡ lateral earth pressure ratio, soil against headwall	0.80
GMH	≡ moist unit weight of earthfill, soil against headwall, in pcf	120.0
GSH	≡ saturated unit weight of earthfill, soil against headwall, in pcf	140.0
KOF	≡ lateral earth pressure ratio, foundation soil	KOH
GMF	≡ moist unit weight of foundation soil, in pcf	GMH
GSF	≡ saturated unit weight of foundation soil, in pcf	GSH
KPF	≡ passive earth pressure ratio, foundation soil	2.0
KOW	≡ lateral earth pressure ratio, soil against sidewall and wingwall	KOH
GMW	≡ moist unit weight of earthfill, soil against sidewall and wingwall, in pcf	GMH
GSW	≡ saturated unit weight of earthfill, soil against sidewall and wingwall, in pcf	GSH
KPW	≡ passive earth pressure ratio, soil against sidewall and wingwall	KPF

*HB = 0.707(J - 1.0)/ZPS + 1.0

Table 2. Water parameters, options, and default values.

M	DW	HEAD	TAIL	ASSUMED QM/QLC2
		OPTION 1 - COMPLETE DEFAULT		
1	DW2 = H	HEAD2 = HEAD1 + TAIL2	TAIL2 = 1.17H	1.0
2	0.86 DW2	HEAD1 + 0.86 TAIL2	0.86 TAIL2	0.8
3	0.71 DW2	HEAD1 + 0.71 TAIL2	0.71 TAIL2	0.6
4	0.54 DW2	HEAD1 + 0.54 TAIL2	0.54 TAIL2	0.4
5	0.34 DW2	HEAD1 + 0.34 TAIL2	0.34 TAIL2	0.2
6	0.0	HEAD1 = (S + .25F)	- (S + 1.0)	0.0
		OPTION 2 - PARTIAL DEFAULT		
1	$\boxed{\text{DW2}}$	$\boxed{\text{HEAD2}}$	$\boxed{\text{TAIL2}}$	1.0
2	0.86 DW2	HEAD1 + .86(HEAD2 - HEAD1)	0.86 TAIL2	0.8
3	0.71 DW2	HEAD1 + .71(HEAD2 - HEAD1)	0.71 TAIL2	0.6
4	0.54 DW2	HEAD1 + .54(HEAD2 - HEAD1)	0.54 TAIL2	0.4
5	0.34 DW2	HEAD1 + .34(HEAD2 - HEAD1)	0.34 TAIL2	0.2
6	0.0	$\boxed{\text{HEAD1}}$	-(S + 1.0)	0.0
		OPTION 3 - NO DEFAULT		
1	$\boxed{\text{DW2}}$	$\boxed{\text{HEAD2}}$	$\boxed{\text{TAIL2}}$	-
2	$\boxed{\text{DWM2}}$	$\boxed{\text{HEADM2}}$	$\boxed{\text{TAILM2}}$	-
3	$\boxed{\text{DWM3}}$	$\boxed{\text{HEADM3}}$	$\boxed{\text{TAILM3}}$	-
4	$\boxed{\text{DWM4}}$	$\boxed{\text{HEADM4}}$	$\boxed{\text{TAILM4}}$	-
5	$\boxed{\text{DWM5}}$	$\boxed{\text{HEADM5}}$	$\boxed{\text{TAILM5}}$	-
6	0.0	$\boxed{\text{HEAD1}}$	-(S + 1.0)	-

$\boxed{}$ ≡ user supplied values

Design Criteria

Materials
Class 4000 concrete and intermediate grade steel are assumed.

Working Stress Design
Design of sections is in accordance with working stress methods. The allowable stresses in psi are

Extreme fiber stress in flexure	$f_c = 1600$
Shear, V/bD*	$v = 70$
Flexural Bond	
tension top bars	$u = 3.4\sqrt{f_c'}/D$
other tension bars	$u = 4.8\sqrt{f_c'}/D$
Steel	
in tension	$f_s = 20{,}000$
in compression, axially loaded	$f_s = 16{,}000$

Minimum Slab Thicknesses

Walls	10 inches
Bottom slabs	11 inches

Temperature and Shrinkage Steel
The minimum steel ratios are

for unexposed faces	$p_t = 0.001$
for exposed faces	$p_t = 0.002$

Slabs more than 32 inches thick are taken as 32 inches. Where spans are long, it may be desirable to increase temperature and shrinkage steel areas over those provided herein. See NEH-6, article 4. Reinforced Concrete.

Web Reinforcement
The necessity of providing some type of stirrup or tie in a slab because of bending action is avoided by

(1) limiting the shear stress, as a measure of diagonal tension, so that web steel is not required, and

(2) providing sufficient effective depth of sections so that compression steel is not required for bending.

Web steel may be required in the headwall buttress, longitudinal sill, or transverse sill. Web steel is required when the shear stress v exceeds 70 psi. Indices are provided from which area and spacing of web steel may be determined.

*Shear sometimes critical at D from face, sometimes at face, see page 17 of TR-42.

Cover for Reinforcement

Steel cover is everywhere 2 inches except that the cover is 3 inches for bottom steel in slabs and other members deposited on earth.

Steel Required by Combined Bending Moment and Direct Force

Required area determined as explained on pages 31-34 of TR-42, "Single Cell Rectangular Conduits - Criteria and Procedures for Structural Design."

Spacing Required by Flexural Bond

Spacing determined as explained on page 47 of TR-42.

Spacing of Reinforcement

The maximum permissible spacing of any reinforcement is 18 inches.

Multiple Steel Layers

The required steel area and spacing at any location is computed assuming a single layer of steel at that location. If at a particular location, the tabulated required steel area is too large, or the spacing is too small to accommodate the required reinforcement in a single layer, then multiple steel layers may be used if the steel requirements are modified accordingly. TR-43, "Single Cell Rectangular Conduits - Catalog of Standard Designs," pages 9 and 10 contains an approach to this situation.

Preliminary Design

Trial concrete thicknesses are determined and preliminary concrete volumes are computed during the preliminary design phase of the structural design of monolithic straight drop spillways. The number of transverse apron slabs is selected either on the basis of minimum concrete volume, or in accordance with user preference. Quantities may be increased during detail design if computations for required steel areas indicate thicknesses are inadequate.

Assumptions, criteria, and procedures for the various elements of preliminary design are discussed below. Figure 5 presents a simplified flow chart covering major elements. Figure 5 is provided to enhance understanding of the sequence of design and the recycling process that is often necessary. In computations that follow, distances are in feet and thicknesses are in inches except as noted.

Weighted Creep Analyses

Weighted creep theory is used to determine the required depth of cutoff wall, HCUT, below the top of the apron slab. The weighted creep ratio, CREEPR, is selected by the user. The analysis assumes all loss in head occurs from (i) to (a), see Figure 6. A flow net would indicate some loss also occurs between (ℓ) and (i), but this would be complicated by the presence, location, type, and size of the headwall drainage system. Refer to Soil Mechanics Note No. 5, example 4, page 25 and NEH-11 pages 4.14-4.19.

Determine HDIFF as the maximum difference in head between HEAD and TAILPS for loadings M = 1 through 6.

Then

CREEPR x HDIFF = LTOT/3. + 2. x HCUTN + 2. x HTOEN

or

HCUTN + HTOEN = (CREEPR x HDIFF - LTOT/3.)/2

but

HCUTN + HTOEN = HCUT + HTOE - 2. x TAP/12.

so

HCUT + HTOE = (CREEPR x HDIFF - LTOT/3.)/2 + 2. x TAP/12.

Let

SUM = (CREEPR x HDIFF - LTOT/3.)/2 + 2. x TAP/12.

Then

HCUT = SUM - HTOE

Prior to design, HWFTG and THW are unknown, so LTOT is conservatively taken, in initial computations, as

LTOT = LB + 1.833.

The apron slab thickness, TAP, is also unknown. It is assumed as 12. inches in initial computations.

The initial value of HCUT is subject to change when the required value of TAP is known. The value of HCUT may also be changed if it is subsequently determined that HTOE must be increased because of transverse sill stiffness considerations.

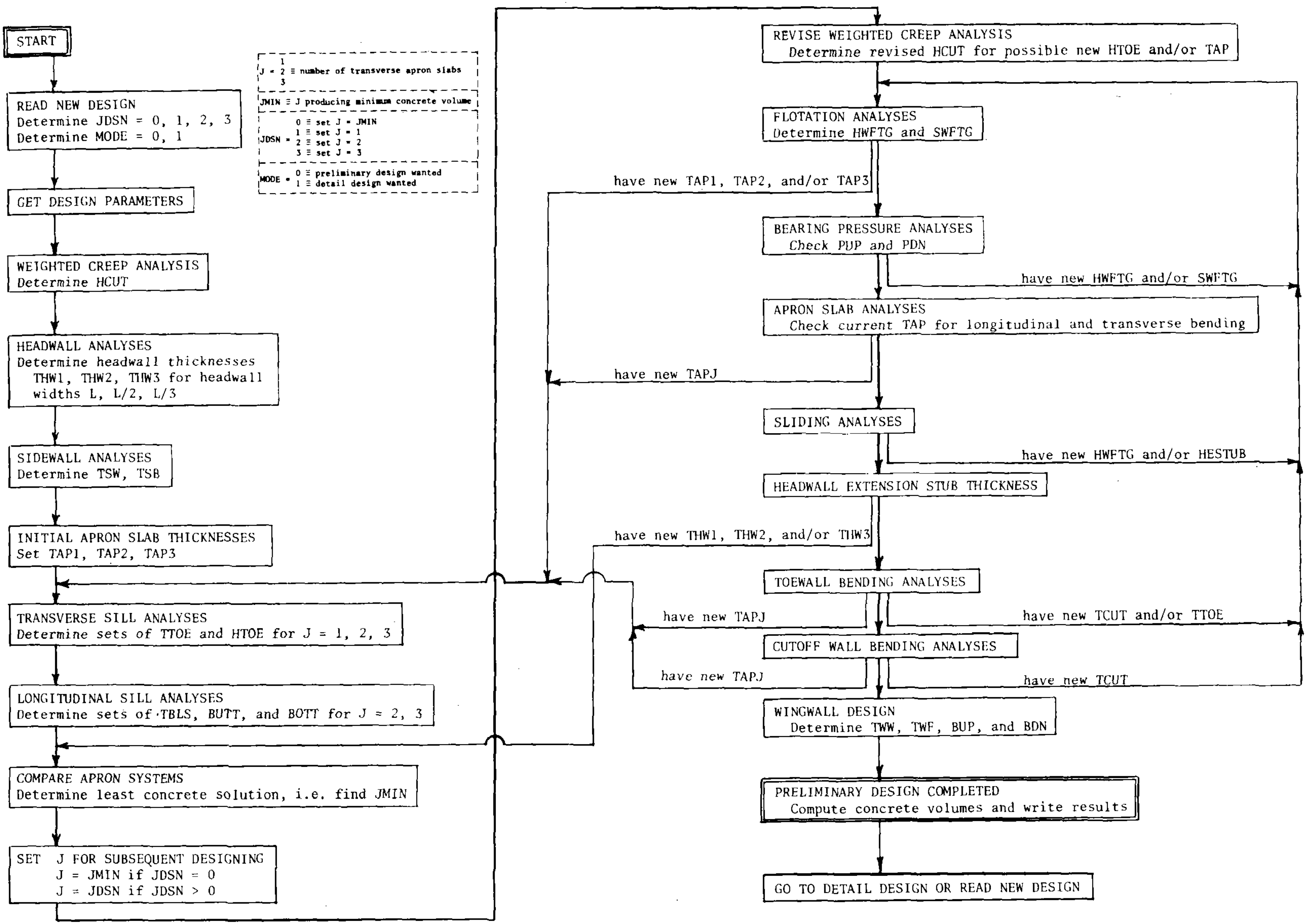


Figure 5. Simplified flow chart for preliminary design.

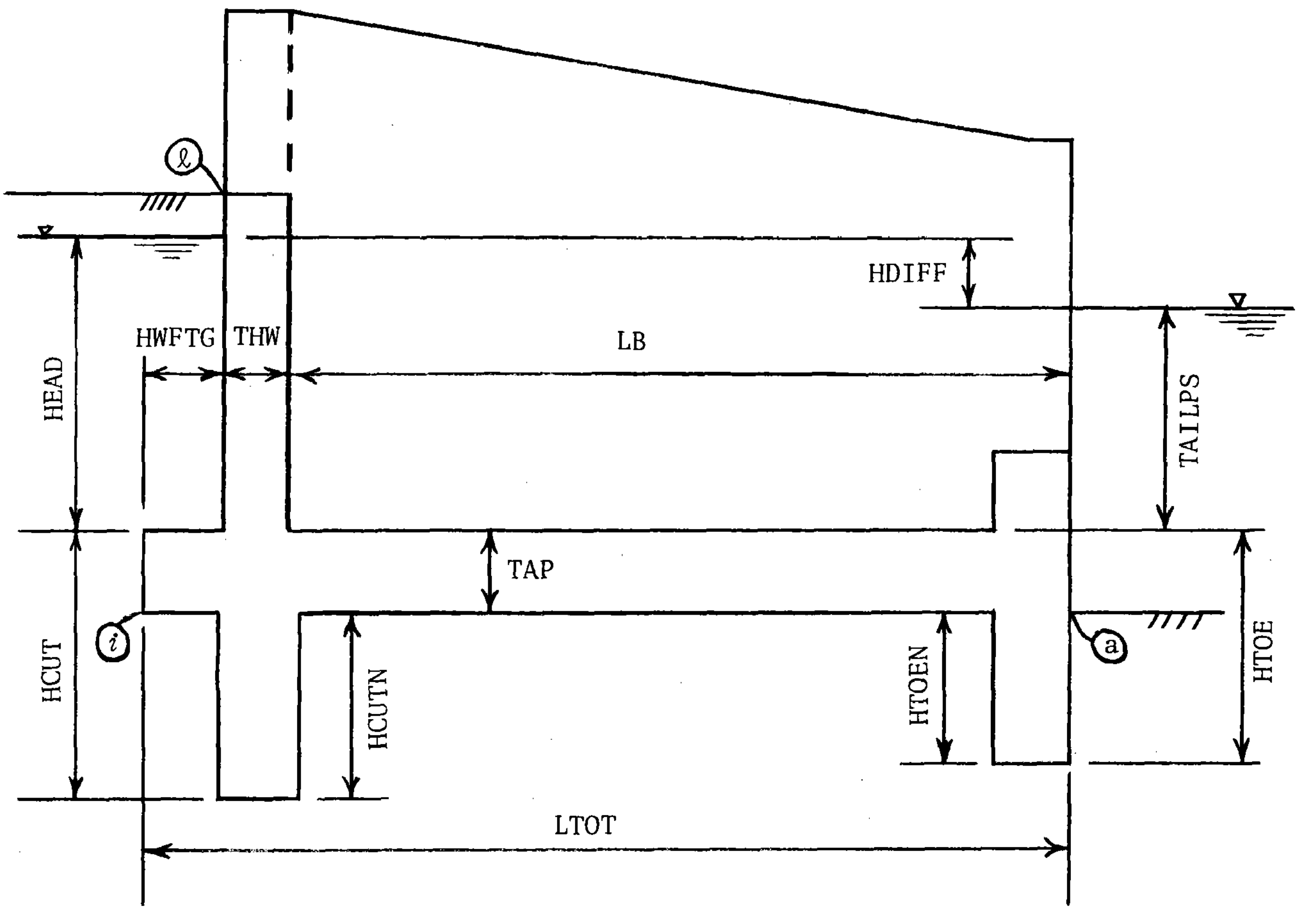


Figure 6. Determination of HCUT.

Several arbitrary restrictions are placed on the value of HCUT determined from the preceding relations. In initial computations:

(1) HCUT may not be less than HTOE

(2) HCUT may not exceed 10. ft. If the computed HCUT exceeds 10., it is made 10. and HTOE is found from

$$HTOE = SUM - 10.$$

If HTOE now exceeds 10. ft, the design is canceled.

In subsequent computations, with a new TAP or HTOE:

(1) HCUT may not be less than HTOE

(2) If the computed HCUT exceeds 10., it is again made 10. and a new HTOE is found as above. If HTOE thus exceeds 10., both HCUT and HTOE are made equal to 1/2 SUM and the design proceeds.

Headwall Analyses

For purposes of determining headwall slab thicknesses, headwall panels are treated as two-way slabs free at the top and fixed along the other three edges. They have a height, A, equal to the sum of F plus S, that is, FPS. They have a width, B, equal to L, L/2, or L/3 depending on the number of headwall buttresses.

Treatment of upstream headwall pressures. When HEAD is less than FPS, the headwater above the crest is treated as surcharge loading and pressure. When HEAD exceeds FPS, the headwater above the crest contributes to the hydrostatic loading and pressure. Figure 7 shows the various components of horizontal loading. These relations apply to all analyses.

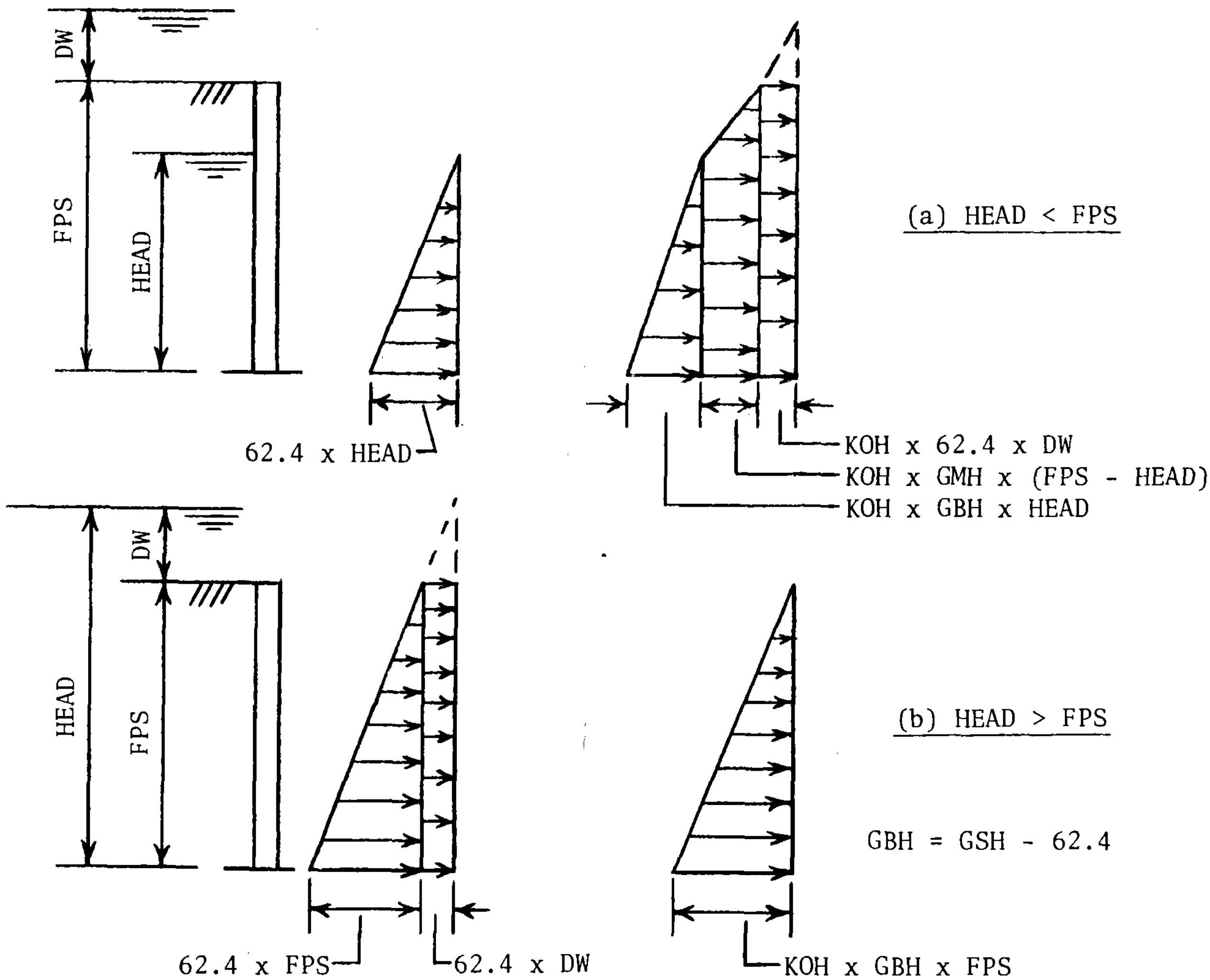


Figure 7. Treatment of upstream headwall pressures.

<u>Conversion of headwall loadings for headwall analyses</u>. To determine headwall panel shears and moments in a reasonably accurate yet simple fashion, headwall loadings are converted to a combination of triangular and uniform loadings. The resulting triangular and uniform loading pressures at the bottom of the headwall panel are PT and PU psf respectively. They are evaluated as shown in Figure 8 for the three possible combinations.

The downstream TAILPS hydrostatic loading, when TAILPS < FPS, is resolved to an approximately equivalent full height triangular loading by equating vertical cantilever moments.

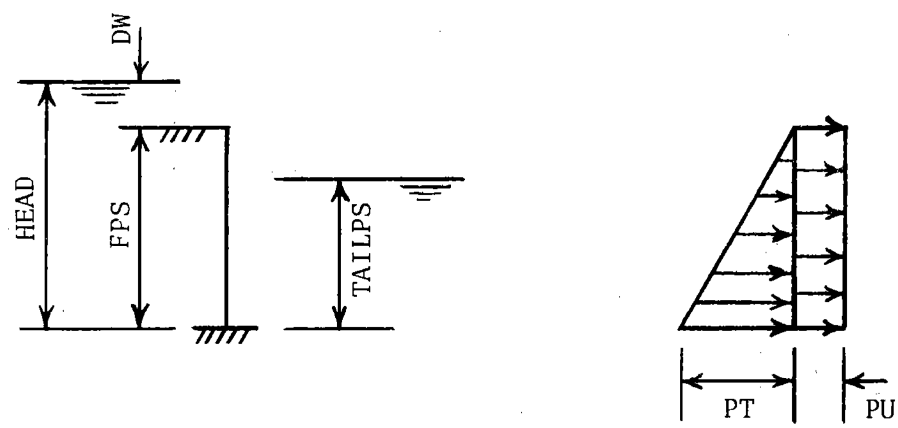
$$M = \frac{1}{6} \times 62.4 \times \overline{TAILPS}^3 = \frac{1}{6} \times EQ \times \overline{FPS}^3$$

$$\therefore \quad EQ = 62.4 \times (TAILPS/FPS)^3$$

This produces a smaller effect than would be obtained by equating vertical shear expressions.

An upstream "broken back" loading occurs when HEAD < FPS, refer to Figure 7(a). This is resolved to an approximately equivalent full height triangular plus uniform loading by taking the triangular and uniform base pressures at their full values.

Both the above approximations are conservative for intermediate values of TAILPS and HEAD, and are exact for limiting values, i.e., for TAILPS = 0, TAILPS = FPS, HEAD = 0, and HEAD = FPS.



(a)	HEAD ≧ FPS TAILPS > FPS	PU = 62.4 x (HEAD - TAILPS) PT = KOH x GBH x FPS
(b)	HEAD ≧ FPS TAILPS ≦ FPS	PU = 62.4 x DW PT ≃ KOH x GBH x FPS + 62.4 x FPS x (1 - $(TAILPS/FPS)^3$)
(c)	HEAD < FPS TAILPS < FPS	PU = KOH x 62.4 x DW PT ≃ KOH x (GBH x HEAD + GMH x (FPS - HEAD)) + 62.4 x (HEAD - FPS x $(TAILPS/FPS)^3$)

Figure 8. Headwall loading combinations.

Panel moments and shears. The approach used to obtain vertical and horizontal moments and shears in headwall panels is to apply a coefficient times the corresponding one-way moment or shear. The coefficients are functions of the B/A ratio for the panel. The values were selected after a consideration of ES-104 in NEH-6 and various slab bending solutions by Timoshenko. Figure 9 shows the headwall

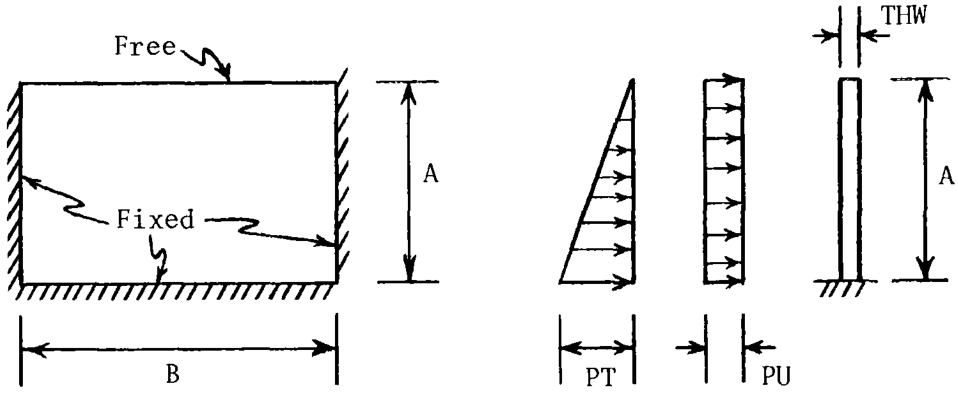


Figure 9. Headwall panel and loading.

panel and the triangular and uniform loadings.

Vertical moments. -- The reference one-way moments are computed as the vertical cantilever moments due to both triangular and uniform loadings. Thus at the middle of the bottom edge of the panel, in ft lbs per ft

$$MV = CMV \times (PT/6 + PU/2) \times A^2$$

where for B/A < 4

$$CMV = (B/A)/4$$

for B/A ≧ 4

$$CMV = 1.0$$

The conservative assumption is made that vertical moments vary linearly from MV/3 at the sides of the panel to MV at a point that is the smaller of B/4 or A from the sides of the panel and then remain constant between the points.

Horizontal moments due to triangular loading. -- The reference one-way moments are computed as the horizontal, fixed-end beam moments at the bottom of the panel. Thus the end moments at the top of the side supports are, in ft lbs per ft

$$MHT1 = CMHT1 \times PT \times B^2/12.$$

and the end moments two-thirds down from the top of the side supports are

$$MHT2 = CMHT2 \times PT \times B^2/12.$$

where for $B/A \leq 4$

$$CMHT2 = 1/(3B/A) \text{ but not more than } 2/3$$

$$CMHT1 = (B/A)/4 \text{ but not more than } CMHT2$$

for $B/A > 4$, use $B = 4A$ and

$$CMHT2 = CMHT1 = 0.0833$$

The horizontal moments are assumed to vary linearly from MHT1 at the top of the panel to MHT2 at two-thirds down from the top to zero at the bottom of the panel.

Horizontal moments due to uniform loading. -- The reference one-way moments are computed as the horizontal, fixed-end beam moments. Thus the end moments at the top of the side supports are, in ft lbs per ft

$$MHU = CMHU \times PU \times B^2/12.$$

where for $B/A \leq 1/2$

$$CMHU = 1.2$$

for $1/2 < B/A \leq 4$

$$CMHU = (47 - 10 \times B/A)/35$$

for $B/A > 4$, use $B = 4A$ and

$$CMHU = 0.2$$

The horizontal moments are assumed to remain constant at MHU from the top of the panel down to two-thirds down from the top and then to vary linearly to zero at the bottom of the panel.

Vertical shear. -- The load carried by a unit width vertical strip in the middle of the panel equals or exceeds the load carried by any other vertical strip. The height of the strip is taken as the smaller of B/2 or A. The shear at the bottom of the strip due to both triangular and uniform loading is taken at full one-way cantilever value and thus is, in lbs per ft

$$VV = (PU + PT(1 - X/(2A))) \times X$$

where X is the height of the strip. The shear along the bottom edge of the panel is assumed to vary in the same manner as described for vertical moments.

Horizontal shear due to triangular loading. -- The reference one-way shear is computed as the horizontal, fixed-end beam shear at the bottom of the panel. Thus the shears at the top of the side supports are, in lbs per ft

$$VHT1 = CVHT1 \times PT \times B/2$$

and the shears two-thirds down from the top of the side supports are

$$VHT2 = CVHT2 \times PT \times B/2$$

where for $B/A < 4$

$$CVHT1 = 0.2$$

$$CVHT2 = 1/(1 + B/A)$$

for $B/A \geqq 4$ use $B = 4A$ and

$$CVHT1 = 0.2$$

$$CVHT2 = 0.2$$

The shear along the side supports of the panel is assumed to vary in the same manner as described for horizontal moments due to triangular load.

Horizontal shear due to uniform loading. -- The shears at the top of the side supports are taken at the full one-way value and thus are, in lbs per ft

$$VHU = PU \times B/2$$

except that when $B/A > 4$, use $B = 4A$. The shears are assumed to remain constant at VHU from the top of the panel down to a distance KHU x A from the top and then to vary linearly to zero at the bottom of the panel, where

$$KHU = 0.4/(B/A)$$

Required headwall thicknesses. The headwall may be used with no headwall buttress, or it may be used with one or two buttresses. Therefore the headwall thickness required for each panel width: $B = L$, $L/2$, and $L/3$ must be determined. Further, any of the loadings, $M = 1$ through 6, may produce maximum required thickness, hence each must be investigated. Thus the controlling thickness must be found for each loading for each panel width. The controlling thickness for each case is found as the maximum of the thicknesses required for: vertical moment at the middle of the bottom edge of the panel; horizontal moment at the side supports, two-thirds down from the top of the panel; vertical shear in the middle strip of the panel, effective depth distance, D, from the bottom of the panel; and horizontal shear at effective depth distance, D, from the side supports of the panel. In preliminary design it is assumed that maximum horizontal shears due to triangular loading and due to uniform loading, occur at the same location.

For vertical moment and shear, the effective depth is, in inches

$$D = THW - 2.5$$

and for horizontal moment and shear at the side supports, the effective depth is

$$D = THW - 3.0.$$

For thickness required by vertical moment at the bottom of the panel, the direct force is the weight of a unit width strip above the section. For horizontal moment, the direct force is taken as zero since its value is uncertain.

The maximum required headwall thicknesses, called THW1, THW2 and THW3 for B = L, L/2, and L/3, determined as indicated above, are saved for subsequent comparisons and computations. Likewise the maximum headwall thicknesses required by vertical moment, called THV1, THV2, and THV3 are also saved since they may determine initial values of the apron slab thickness.

Sidewall Analyses

Sidewall analyses follow the basic concept of a ficticious 45^{o} cut through the wall, starting from the lower upstream corner of the sidewall. The sidewall is assumed to receive its principal support from the headwall and the apron slab. However, in recognition that headwall support does not reach full sidewall height, further assumptions are made concerning the location and magnitude of critical vertical sidewall bending. Wall design by the 45^{o} cut approach, although fundamentally conservative, completely neglects positive moments, i.e., tension on the front surface of the wall, that actually exist in the region of the assumed cut. These positive moments are considered in detail design since they are not of sufficient magnitude to influence the required thickness of the sidewall.

Sidewall water heads. The water heads acting on the outside of the sidewall are required for sidewall and wingwall designs, and for computing sidewall moments in apron slab design. Weighted creep theory is used to obtain the heads, HSIDE and HWING shown in Figure 10. The line of creep is assumed to extend from point (j), around the cutoff wall, and along the backside of the sidewall and wingwall as shown. Thus for any loading, in ft of water

$$\text{HSIDE} = \text{HEAD} - (\text{HEAD} - \text{TAILPS}) \times \left(\frac{2 \times \text{HCUT}}{2 \times \text{HCUT} + \text{LB}/3 + (\text{J} - 1)/3}\right)$$

and

$$\text{HWING} = \text{HEAD} - (\text{HEAD} - \text{TAILPS}) \times \left(\frac{2 \times \text{HCUT} + \text{LB}/3}{2 \times \text{HCUT} + \text{LB}/3 + (\text{J} - 1)/3}\right)$$

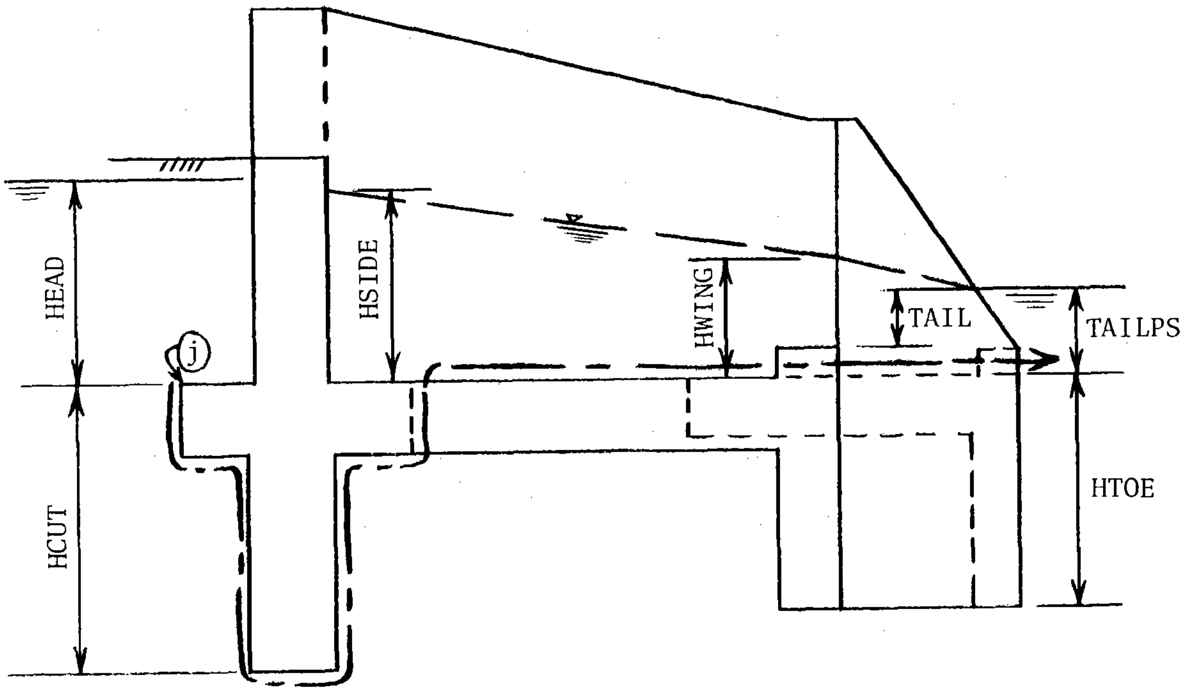


Figure 10. Sidewall water heads for given loading.

The designer is provided an option. He may have HSIDE and HWING computed as above. Or he may have HSIDE and HWING set equal to TAILPS on the assumption that drains through the sidewall and wingwall effectively reduce these heads to TAILPS. The designer exercises his option by the sidewall design switch, SWLDRN. SWLDRN = 0. means there are no drains, whereas SWLDRN = 1. means there are such drains.

Vertical bending. To account for the headwall height, FPS, not reaching full sidewall height, FPSPH, the critical section for vertical sidewall bending is assumed located a distance, LV, from the face of the headwall.

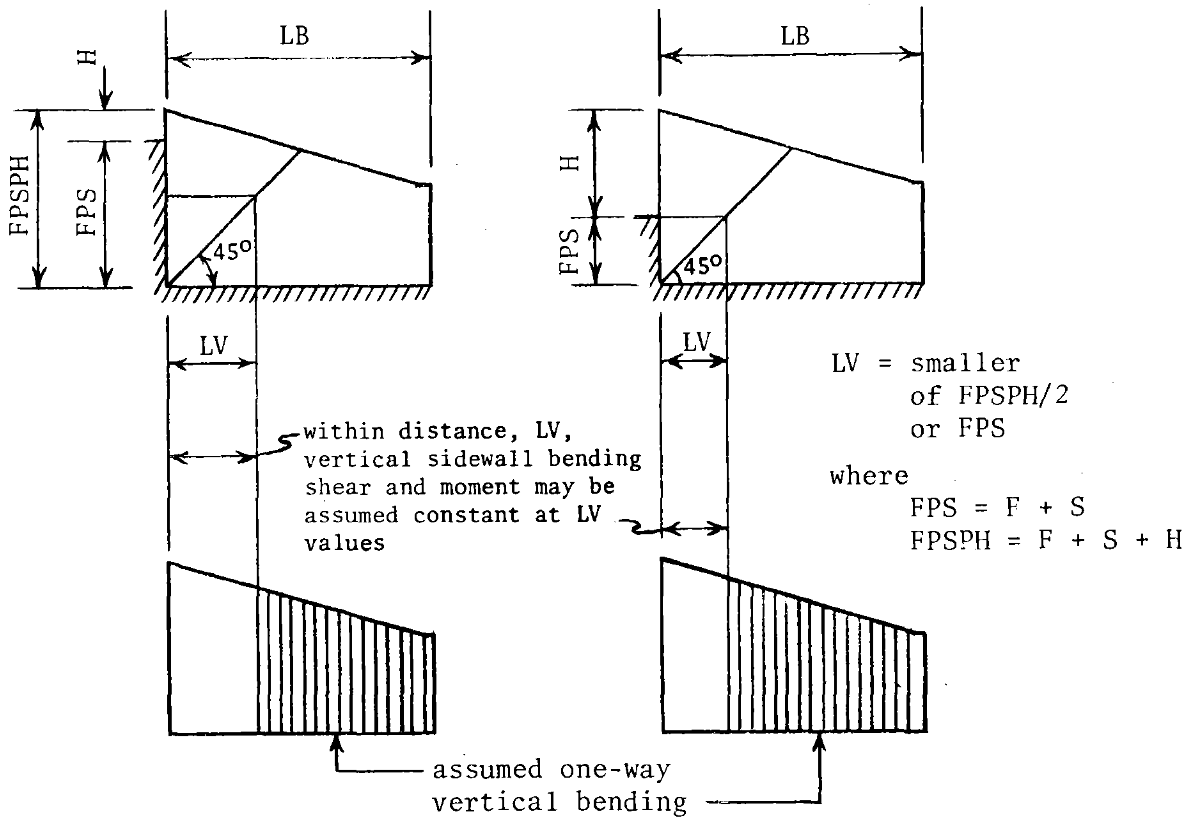


Figure 11. Assumptions for vertical sidewall bending.

LV is a variable distance, the smaller of FPSPH/2 or FPS, which reflects the amount of support provided the sidewall by the headwall. For FPS ≧ FPSPH/2, LV remains constant at LV = FPSPH/2. For FPS < FPSPH/2, LV decreases with FPS. Thus as FPS approaches zero, the sidewall is assumed to approach a pure cantilever wall with critical section at the upstream end. This assumption is conservative in that the headwall extension stub, above headwall elevation, may offer restraint which reduces pure cantilever bending. Figure 11 shows the assumed critical section and indicates additional assumptions regarding vertical sidewall bending which are important in detail design.

Figure 12 indicates conditions for a typical loading case. The critical section is at LV from the face of the headwall. The height of the sidewall at this section is

$$HSW = J + (FPSPH - J) \times (LB - 6/12 - LV)/(LB - 6/12)$$

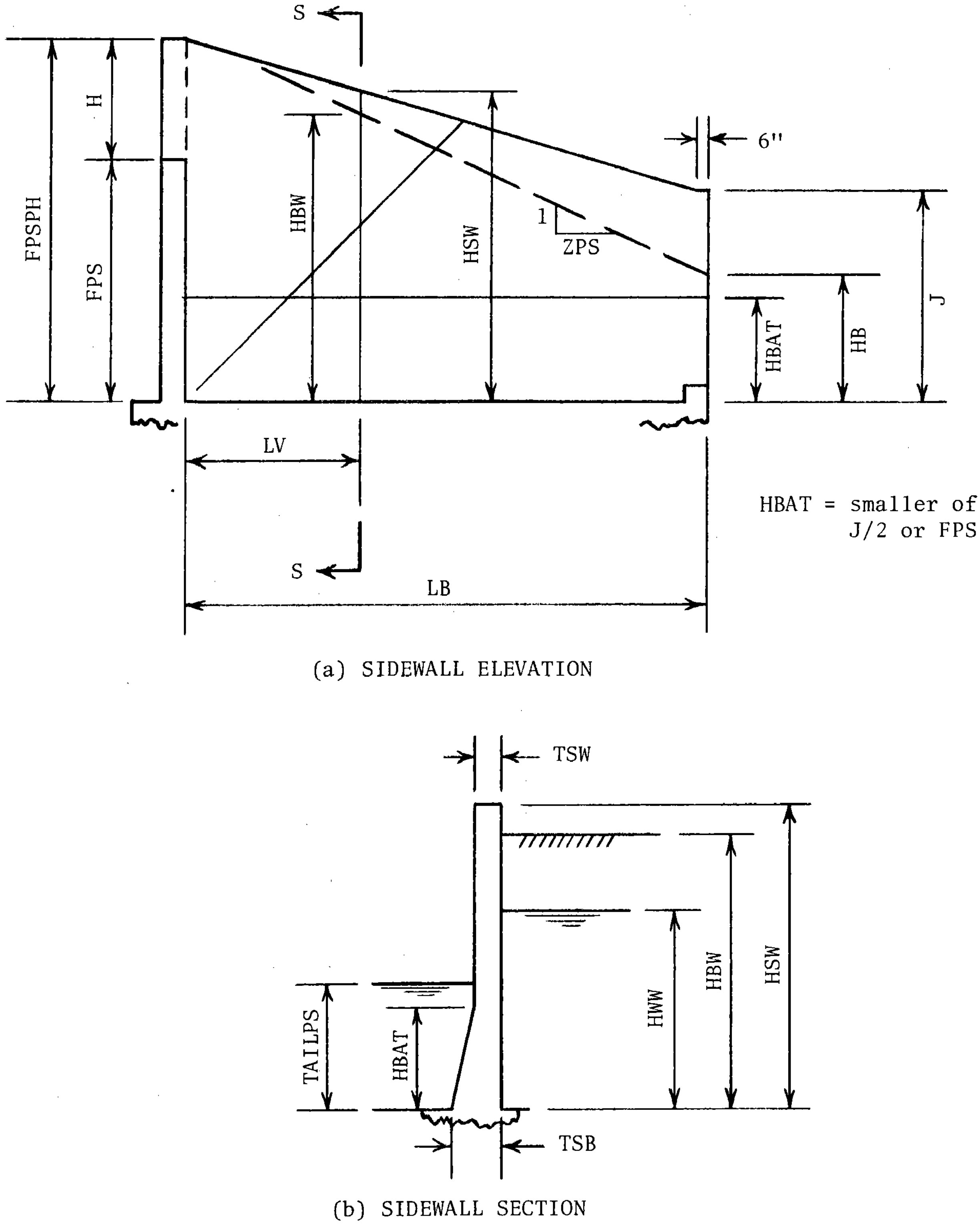


Figure 12. Vertical bending of sidewall.

The height of backfill at this section is

HBW = HB + (LB - LV)/ZPS

but not more than HSW. The water heads, HSIDE and HWING, are computed for the loading, then the water head on the outside of the wall at this section is

HWW = HWING + (HSIDE - HWING) x (LB - LV)/LB

With HSW, HBW, HWW, and TAILPS known for a particular loading, the thickness, TSB, required for full height, vertical cantilever bending may be found. It is determined as the larger thickness required for either shear at D above the bottom of the wall or vertical moment at the bottom of the wall. The procedures for determining this thickness are similar to those shown on pages 9-10 of TR-50, "Design of Rectangular Structural Channels", and on pages 21-23 of TR-54, "Structural Design of SAF Stilling Basins" except for modifications due to the vertical plane surface on the back of the sidewall and due to possible batter of the lower front surface of the sidewall. The effective depth for vertical bending is, in inches

D = TSB - 2.5.

Any of the loadings M = 1 through 6, may produce the maximum required TSB for vertical bending, hence each is investigated. This maximum required TSB is saved for comparison with that required for horizontal bending. The maximum required thickness for vertical moment, called TSV, is also saved since it may determine the initial value of the apron slab thickness.

Horizontal bending. Horizontal bending will seldom govern the thickness of the sidewall. It is presented here for the sake of completeness and because it is important in detail design. As previously noted, the sidewall is designed for sufficient strength on the basis of a combination of vertical support along the headwall and horizontal support along the apron. Nevertheless, horizontal bending is also influenced by whatever support the headwall extension stub above the headwall can provide. Therefore, horizontal cantilever bending of the sidewall is conservatively determined without regard to the makeup of the vertical support. One-way horizontal bending is assumed at and below a distance, LHS, above the bottom of the sidewall. Horizontal cantilevers extend from the ficticious 45^{o} cut to the vertical support. LHS is taken equal to the larger of FPS or 2/3 of FPSPH except that it may not exceed LHMAX. Figure 13 shows LHS and LHMAX and separates the sidewall above the assumed cut into two areas: one the area below LHS, subjected to assumed one-way bending, and the other the area above LHS. Loads on the area above LHS, are not of immediate concern, they are carried by a combination of indeterminate processes including:

(a) horizontal bending resisted by the headwall extension stub,

(b) additional horizontal bending below LHS,

(c) vertical bending, and

(d) torsion in the "corner column" made up of the intersection of headwall, headwall extension stub, and sidewall.

$$\text{LHMAX} = \frac{\text{FPSPH}}{1 + \dfrac{(\text{FPSPH} - \text{J})}{(\text{LB} - 6/12)}} \quad \text{but not more than (LB - 6/12)}$$

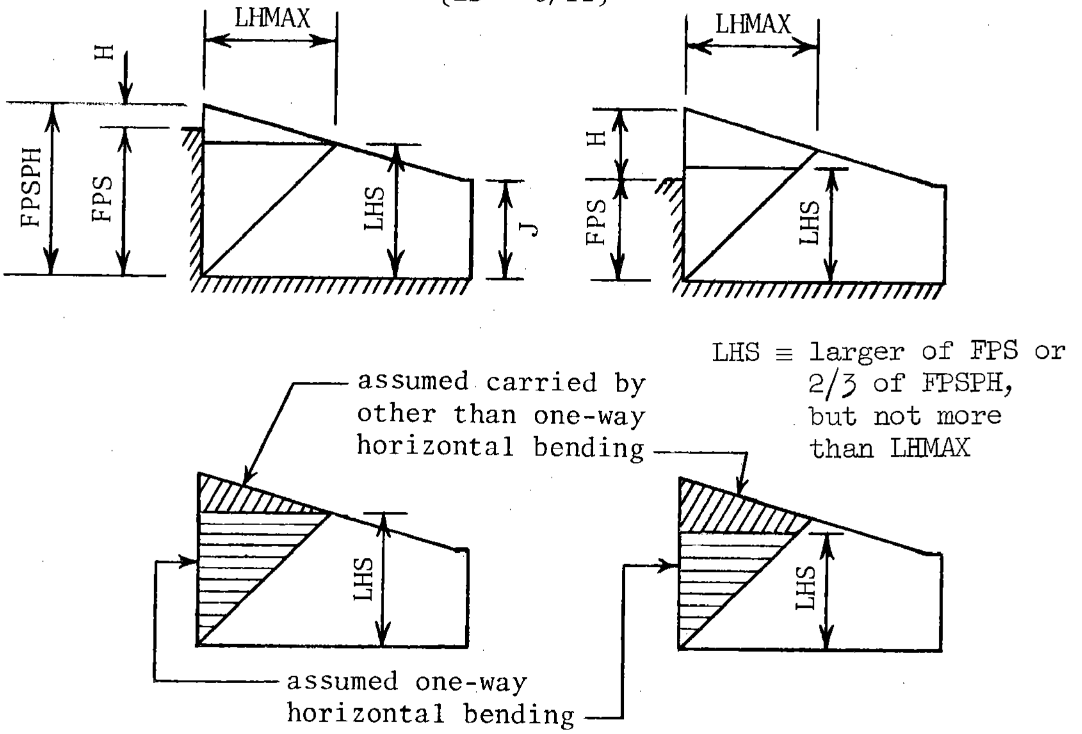


Figure 13. Assumptions for horizontal sidewall bending.

Determination of the required thicknesses for horizontal moment and shear for a particular loading starts at LH = LHS above the bottom of the sidewall. The larger of these thicknesses is used to compute the required TSB for this LH value. After this thickness is determined, LH is decremented 0.5 ft and another set of thicknesses for moment and shear is determined from which a second required TSB is calculated. If the second value for required TSB is greater than the first value, LH is decremented again. This procedure is repeated until the strip producing maximum required TSB for this loading is located. Figure 14 illustrates one of several possible configurations for a typical LH. The loading on the cantiliver strip, LH long, is converted to a combination of uniform and triangular loadings from which moments and shears and hence thicknesses, T, may be determined. Thus, from Figure 14, in ft

```
HBLB = HB + LB/ZPS            but not more than FPSPH
HSW  = J + (FPSPH - J) x (LB - 6/12 - LH)/(LB - 6/12)
HBW  = HB + (LB - LH)/ZPS  but not more than HSW
HWW  = HWING + (HSIDE - HWING) x (LB - LH)/LB
```

so that, letting HDIFF = HBLB - HSIDE, in psf

```
PL = KOW x GMW x HDIFF + (KOW x GBW + 62.4)
                x (HSIDE - LH) - 62.4 x (TAILPS - LH)
```

and next letting HDIFF = HBW - HWW, in psf

$$PU = KOW \times GMW \times HDIFF + (KOW \times GBW + 62.4) \times (HWW - LH) - 62.4 \times (TAILPS - LH)$$

Then

$$PT = PL - PU$$

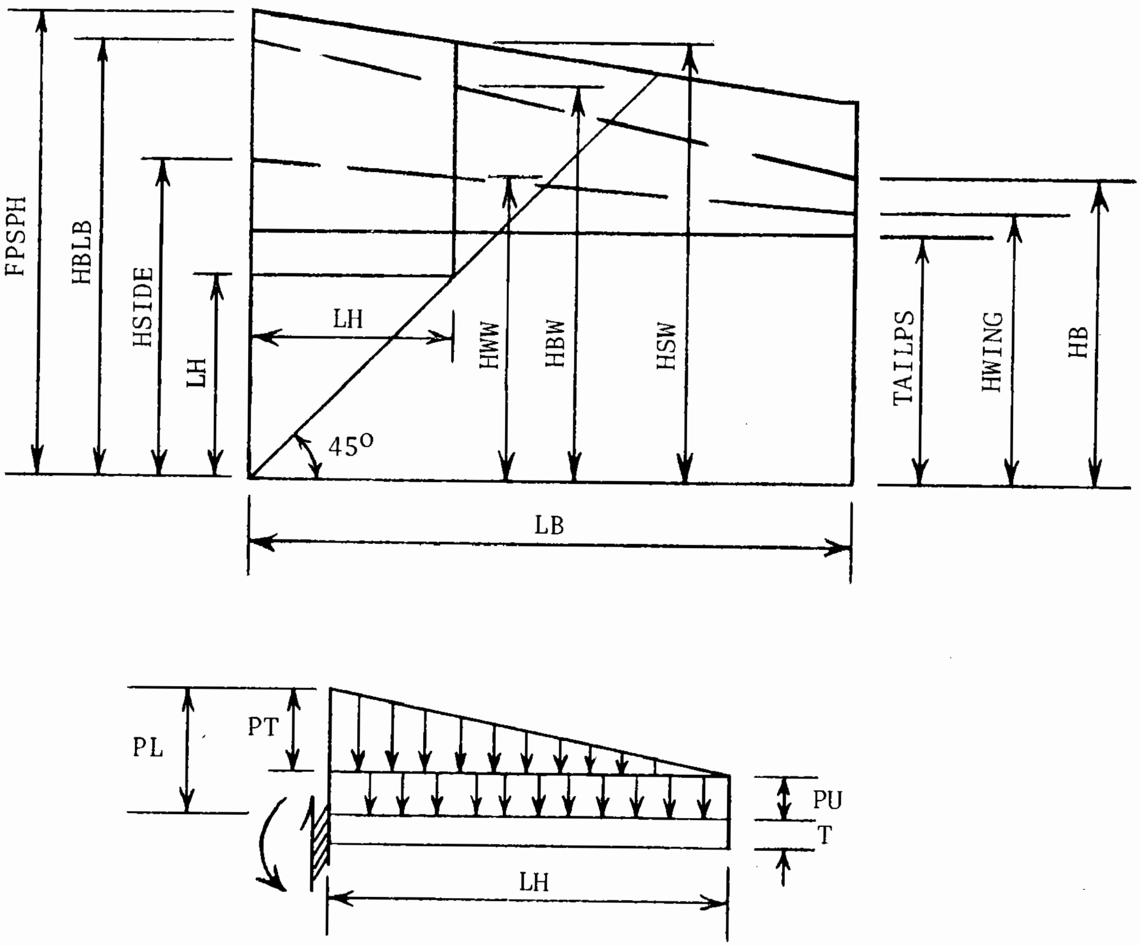


Figure 14. Horizontal bending of the sidewall.

With the required thickness, T, known, the corresponding TSB can be obtained from a consideration of LH, HBAT, and sidewall batter, if any. The effective depth for horizontal bending is, in inches

$$D = T - 3.0.$$

Any of the loadings, M = 1 through 6, may produce the maximum required TSB for horizontal bending, hence each is investigated. The maximum required values of TSB for vertical bending and for horizontal bending are compared to determine the required sidewall bottom thickness, TSB. With TSB known, the required thickness at the top of the sidewall, TSW is determined by subtraction of specified sidewall batter, if any, from TSB.

Apron System Stiffnesses

The resistance of apron panels to loads acting normal to the plane of the apron is very complex and highly indeterminate. Resistance depends on the relative stiffnesses of apron slab, longitudinal sill(s) if any, and transverse sill. The assumption herein is that longitudinal sills, when present, and the transverse sill are sufficiently stiff to produce two-way bending essentially in agreement with theory for non-yielding supports. Basic questions thus require consideration.

(1) How can the stiffness of longitudinal sills and transverse sills be determined?

(2) How stiff do the longitudinal sills and transverse sills need to be to produce the desired behavior?

(3) How can the sills be sized to provide the necessary stiffness?

The various concepts, theories, and procedures necessary to answer these questions are developed below in an order that is convenient for presentation. The actual sequence of design differs from the discussion order. The design sequence is:

(1) establish the initial apron slab thicknesses corresponding to the three possibilities of no longitudinal sill, one longitudinal sill, and two longitudinal sills. These are

TAP1 = larger of (THV1 + 1.) or (TSV + 1.)

TAP2 = larger of (THV2 + 1.) or (TSV + 1.)

TAP3 = larger of (THV3 + 1.) or (TSV + 1.),

(2) determine required transverse sill dimensions, TTOE and HTOE, for the three possibilities,

(3) determine required longitudinal sill dimensions, TBLS, BUTT, and BOTT, for the one and two sill possibilities,

(4) unless the designer has preset the design he desires, compare relative concrete quantities for the three possibilities and select the design with least concrete.

Longitudinal sills and transverse apron bending. This section assumes the transverse sill is adequately stiff and only considers the interaction of longitudinal sills and transverse apron bending.

Analysis of stiffness. -- Transverse bending of the apron depends on the stiffness of the longitudinal sills, if any. Small displacements of a longitudinal sill, relative to the sidewalls, produce large reductions in the support the sill provides the apron. Since transverse bending design of the apron assumes longitudinal sills act essentially as non-yielding supports, the longitudinal sills must be made correspondingly stiff.

The analysis of longitudinal sill stiffness, relative to apron plate stiffness, is an extremely difficult problem. See for example Timoshenko, Theory of Plates and Shells, pages 214-218.

An approximation of the problem is obtained by assuming the apron is subjected to uniform loading, is constructed of one-way transverse strips, and each strip is simply supported at its ends and rests on interior elastic beams, i.e. longitudinal sills. With this analysis it is possible to determine either the required depth of the longitudinal sill, or to determine the maximum span of the longitudinal sill, or to determine some combination of these, where every solution corresponds to some preset minimum acceptable reactive support. Figure 15 shows the assumed construction and resulting loadings and displacements. In the figure:

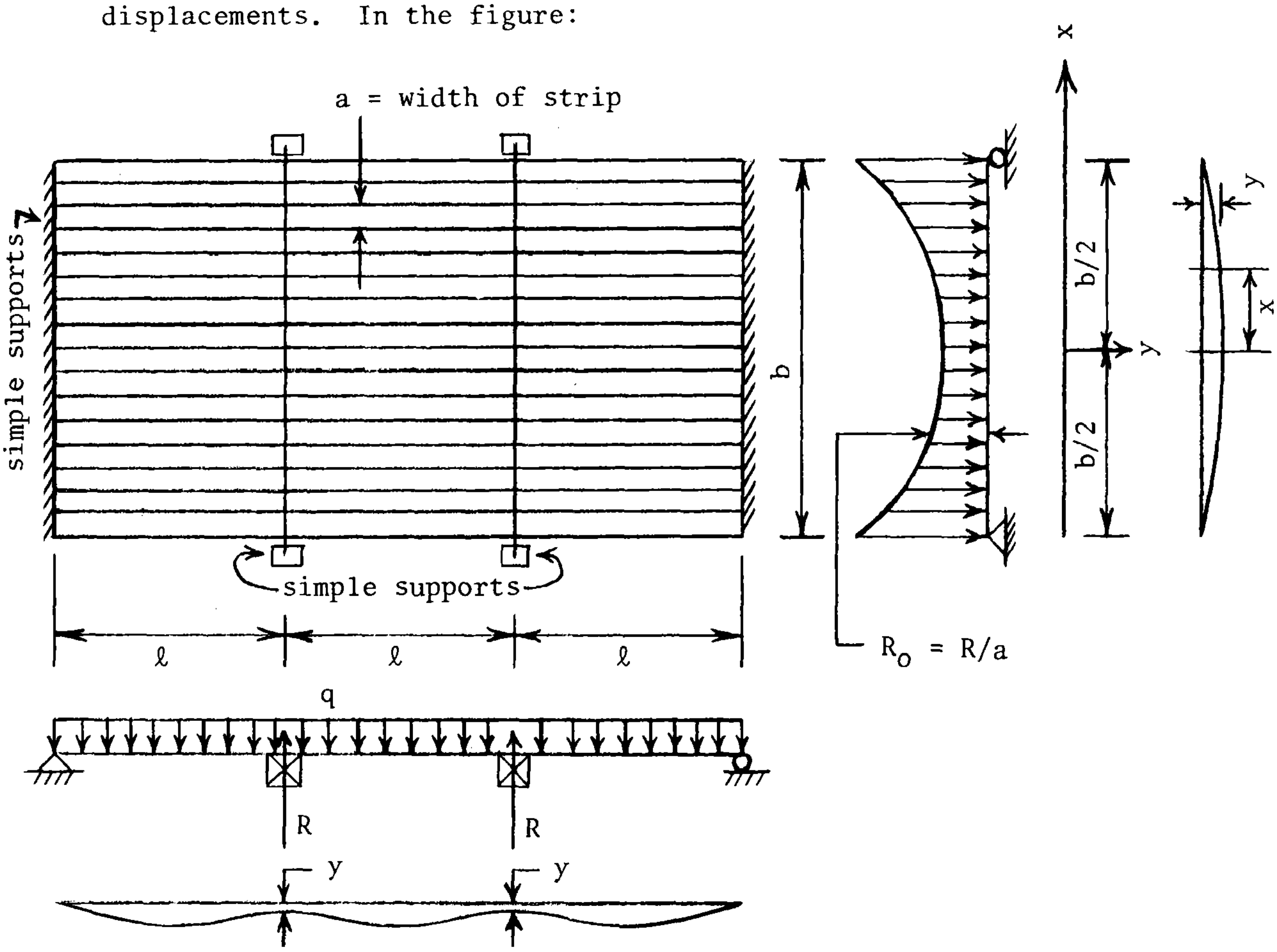


Figure 15. Longitudinal sill analysis - two sills.

w $\equiv$ uniform loading on the apron, in psf
q $\equiv$ uniform loading on each strip = wa, in pℓf
a $\equiv$ width of strip, in ft
t $\equiv$ thickness of strip, in ft
I_1 $\equiv$ moment of inertia of strip = $at^3/12$, in ft^4
I_2 $\equiv$ moment of inertia of elastic beam, i.e., sill, in ft^4
R $\equiv$ reaction the sills provide the strip, in lbs
R_o $\equiv$ varying load on the sill, in pℓf
b $\equiv$ longitudinal sill span, in ft
ℓ $\equiv$ transverse apron spans, in ft
y $\equiv$ displacement of sill at x from midspan, in ft

Thus

$$y = \frac{11}{12}\frac{q\ell^4}{EI_1} - \frac{5}{6}\frac{R\ell^3}{EI_1}$$

or

$$R = \frac{11}{10} q\ell - \frac{6}{5}\frac{EI_1}{\ell^3} y$$

so

$$R_o = R/a = \frac{11}{10}\frac{q\ell}{a} - \frac{6}{5}\frac{EI_1}{a\ell^3} y$$

Let

$$q_o = \frac{11}{10}\frac{q\ell}{a} \text{ and } k = \frac{6}{5}\frac{EI_1}{a\ell^3}$$

then

$$R_o = q_o - ky$$

Now, for the sill

$$EI_2 \frac{d^4y}{dx^4} = R_o = q_o - ky$$

or

$$\frac{d^4y}{dx^4} + 4\beta^4 y = \frac{q_o}{EI_2} \qquad \text{where } 4\beta^4 = \frac{k}{EI_2} = \frac{6}{5}\frac{I_1}{a\ell^3 I_2}$$

The longitudinal sill provides the least support to the apron at midspan of the sill. Thus, from Timoshenko, Strength of Materials, Part II, pages 20-22

$$y)_{x=o} = \frac{q_o}{k}\left(1 - \frac{2 \cos \beta b/2 \cosh \beta b/2}{\cos \beta b + \cosh \beta b}\right)$$

so

$$R_o)_{x=o} = q_o - ky)_{x=o} = q_o\left(\frac{2 \cos \beta b/2 \cosh \beta b/2}{\cos \beta b + \cosh \beta b}\right)$$

or

$$R_o)_{x=o} = \left(\frac{11}{10} w\ell\right)\left(\frac{2 \cos \beta b/2 \cosh \beta b/2}{\cos \beta b + \cosh \beta b}\right)$$

where

$$\beta b = \left\{\frac{3}{10} \cdot \frac{b^4}{a\ell^3} \cdot \frac{I_1}{I_2}\right\}^{1/4} = \left\{\frac{1}{40} \cdot \left(\frac{t}{\ell}\right)^3 \cdot \frac{1}{I_2}\right\}^{1/4} \cdot b$$

If the longitudinal sills were actually non-yielding, the loading on the sill would be constant at

$$R_o)_{non} = \frac{11}{10} w\ell$$

Thus, the question pertaining to how stiff the longitudinal sills are, can be answered as a ratio of how much reactive support is provided to how much reactive support would be provided by non-yielding supports.

Let the stiffness ratio be SR, then

$$SR = R_o)_{x=o}/R_o)_{non}$$

or

$$SR = \frac{2\cos\beta b/2\ \cosh\beta b/2}{\cos\beta b + \cosh\beta b}$$

Required stiffness. -- The question pertaining to how stiff the longitudinal sills should be, can be approached by examining the way transverse apron moments vary with longitudinal sill reactive support. The previous uniformly loaded, three span, continuous beam is used for illustration. Figure 16 shows the continuous beam and one method of

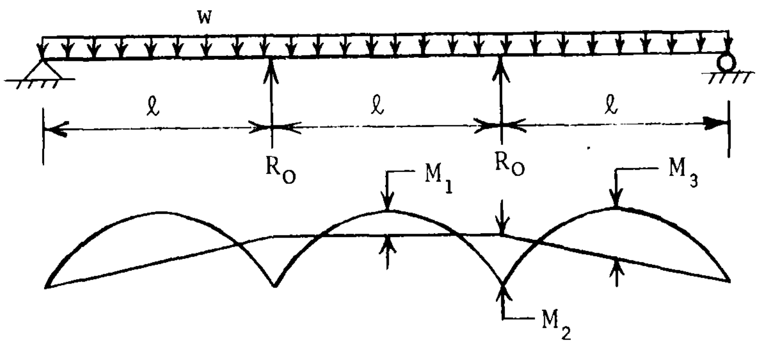


Figure 16. Effect of reactive support on moments, 3 spans.

depicting the moment diagram (for any reasonable R_o value). Moment M_2 decreases as R_o decreases, moments M_1 and M_3 increase as R_o decreases. For non-yielding supports

$$R_o)_{non} = \frac{11}{10} w\ell, \quad M_1)_{non} = \frac{1}{40} w\ell^2, \quad M_3)_{non} = \frac{3}{40} w\ell^2$$

and for any R_o

$$M_1 = \frac{9}{8} w\ell^2 - R_o\ell \qquad \text{and } M_3 = \frac{5}{8} w\ell^2 - \frac{R_o\ell}{2}$$

Changes in both M_1 and M_3 with R_o are studied to determine which is the more sensitive. Let

$$RM_1 = M_1/M_1)_{non} = \frac{9/8\ w\ell^2 - R_o\ell}{1/40\ w\ell^2}$$

so

$$R_o = \frac{w\ell}{40}(45 - RM_1)$$

or

$$SR = R_o/R_o)_{non} = \frac{45 - RM_1}{44}$$

Similarly ,

$$RM_3 = M_3/M_3)_{non} = \frac{5/8\ w\ell^2 - R_o\ell/2}{3/40\ w\ell^2}$$

so

$$R_o = \frac{w\ell}{40}(50 - 6\ RM_3)$$

or

$$SR = R_o/R_o)_{non} = \frac{50 - 6\ RM_3}{44}$$

If:

$RM_1 = 1.5$ then $SR = 0.989$
$RM_1 = 2.0$ then $SR = 0.977$
$RM_3 = 1.5$ then $SR = 0.932$
$RM_3 = 2.0$ then $SR = 0.864$

Hence, M_1 is more sensitive than M_3, and criteria should be tied to the maximum permissible change in M_1.

Selection of the limiting stiffness ratio should recognize that the analysis for SR is conservative because:

(1) the apron slab is a monolithic entity rather than a collection of one-way transverse strips,

(2) rather than being simply supported at the sidewalls, the apron slab has continuity with all its supports,

(3) the longitudinal sills are restrained rather than simply supported ,

(4) the longitudinal sill support is evaluated at midspan, where the support provided is least.

Thus a moment ratio, RM = 1.5 might ordinarily represent an acceptable maximum value. However, in this instance, M_1 is a relatively small moment. Slab thickness is governed by other sections and steel is usually controlled by requirements for temperature and shrinkage. Hence, RM_1 = 2. is deemed satisfactory. Therefore for the case of two longitudinal sills, that is, three transverse apron slabs, the stiffness ratio for an allowable design must equal or exceed SR = 0.977.

Proceeding similarly for the case of one longitudinal sill, that is, two transverse apron slabs

$$R_o)_{x=o} = (\frac{5}{4}\ w\ell)(\frac{2\cos\beta b/2\ \cosh\beta b/2}{\cos\beta b + \cosh\beta b})$$

where

$$\beta b = \left\{\frac{3}{2}\cdot\frac{b^4}{a\ell^3}\cdot\frac{I_1}{I_2}\right\}^{1/4} = \left\{\frac{1}{8}\cdot\left(\frac{t}{\ell}\right)^3\cdot\frac{1}{I_2}\right\}^{1/4}\cdot b$$

So, again

$$SR = \frac{2 \cos \beta b/2 \cosh \beta b/2}{\cos \beta b + \cosh \beta b}$$

In a two-span continuous beam, the moment over the interior support decreases with decreasing reactive support. The midspan moments increase as R_o decreases. Thus for the midspan moments, M

$$R_o)_{non} = \frac{5}{4} w\ell \qquad M)_{non} = \frac{1}{16} w\ell^2$$

and for any R_o

$$M = \frac{3}{8} w\ell^2 - R_o\ell/4$$

so

$$RM = M/M)_{non} = \frac{\frac{3}{8} w\ell^2 - R_o\ell/4}{\frac{1}{16} w\ell^2}$$

or

$$R_o = \frac{w\ell}{4}(6 - RM)$$

and

$$SR = R_o/R_o)_{non} = \frac{6 - RM}{5}$$

Using RM = 1.5 as an acceptable maximum value, since M is an important moment, the stiffness ratio for an allowable design must equal or exceed SR = 0.900 for the two span case.

Sizing the longitudinal sill. -- There remains the question of sizing the longitudinal sills. Referring to Figure 1, a cross section of a longitudinal sill consists of a raised portion of height S above the apron, a portion of the apron slab extending out each side of the sill, and perhaps a dropped portion of depth BOTT below the apron. Figure 17 shows the initial section with BOTT = 0. and a typical section with BOTT > 0. Sill dimensions are suffixed with an L, all values are in ft.

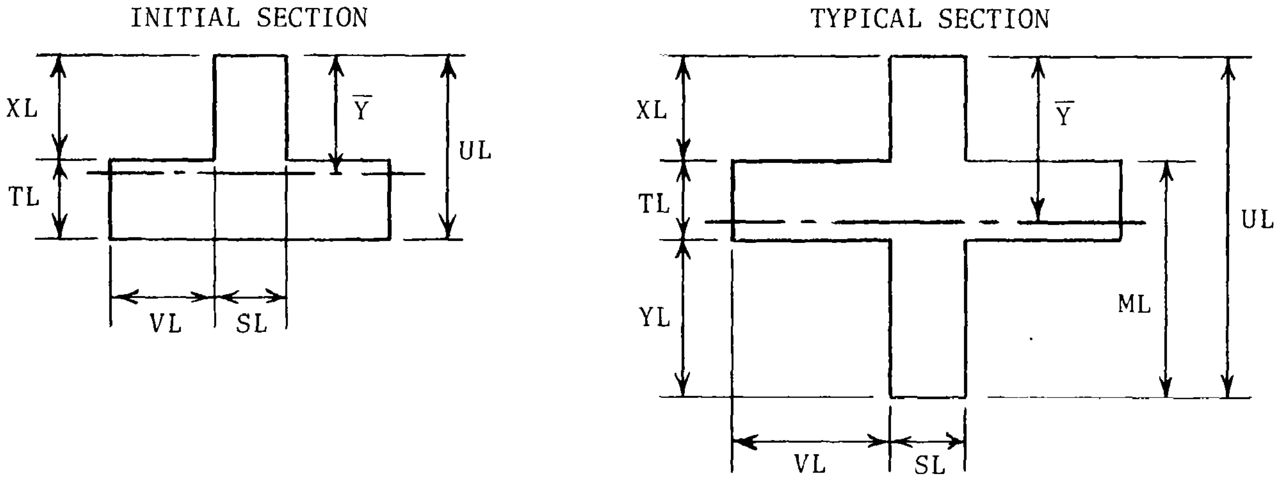


Figure 17. Longitudinal sill cross sections.

For the initial section:

TL = TAP2/12. or TAP3/12. as appropriate
SL = 1.0 $\therefore$ TBLS = 12. inches
XL = S
VL = XL

For the typical section:

TL = TAP2/12. or TAP3/12.
SL = 1.0
XL = S
ML $\leq$ HTOE
YL = ML - TL $\equiv$ BOTT
VL = larger of XL or YL but not more than 4 x TL in accordance with ACI 318-71, 13.1.5

In any particular stiffness determination, the center of gravity of the section is located and the gross moment of inertia about the center of gravity is calculated. From the gross moment of inertia is subtracted $SL(TL)^3/12$ so that the net moment of inertia, I_2, represents the increase in moment of inertia of the longitudinal sill cross section over that of the apron slab without a sill. Thus, in ft^4

$$I_2 = \overline{I} - SL(TL)^3/12$$

where

$$A = SL(UL) + 2VL(TL)$$
$$\overline{Y} = (SL(UL)^2/2. + 2\ VL(TL)(XL + TL/2))/A$$
$$\overline{I} = SL(UL)^3/3. + 2(VL(TL)^3/12 + VL(TL)(XL + TL/2)^2) - A\overline{Y}^2$$

The process of providing longitudinal sills of adequate stiffness for the case of either one or two longitudinal sills is summarized in the following narrative. Figure 18 indicates the sequence of trials and the development of a sill. First, as appropriate, let

SRMIN = 0.900 or 0.977
ℓ = L/2 or L/3
t = TAP2/12 or TAP3/12

Then, as shown in sketch (a), let

b = LS = LB - TTOE/12

and compute

I_2 for the initial section of Figure 17

$$\beta b = \left\{\frac{1}{8} \cdot \left(\frac{t}{\ell}\right)^3 \cdot \frac{1}{I_2}\right\}^{1/4} \cdot b \text{ or } \left\{\frac{1}{40} \cdot \left(\frac{t}{\ell}\right)^3 \cdot \frac{1}{I_2}\right\}^{1/4} \cdot b$$

$$SR = \frac{2 \cos \beta b/2 \cosh \beta b/2}{\cos \beta b + \cosh \beta b}$$

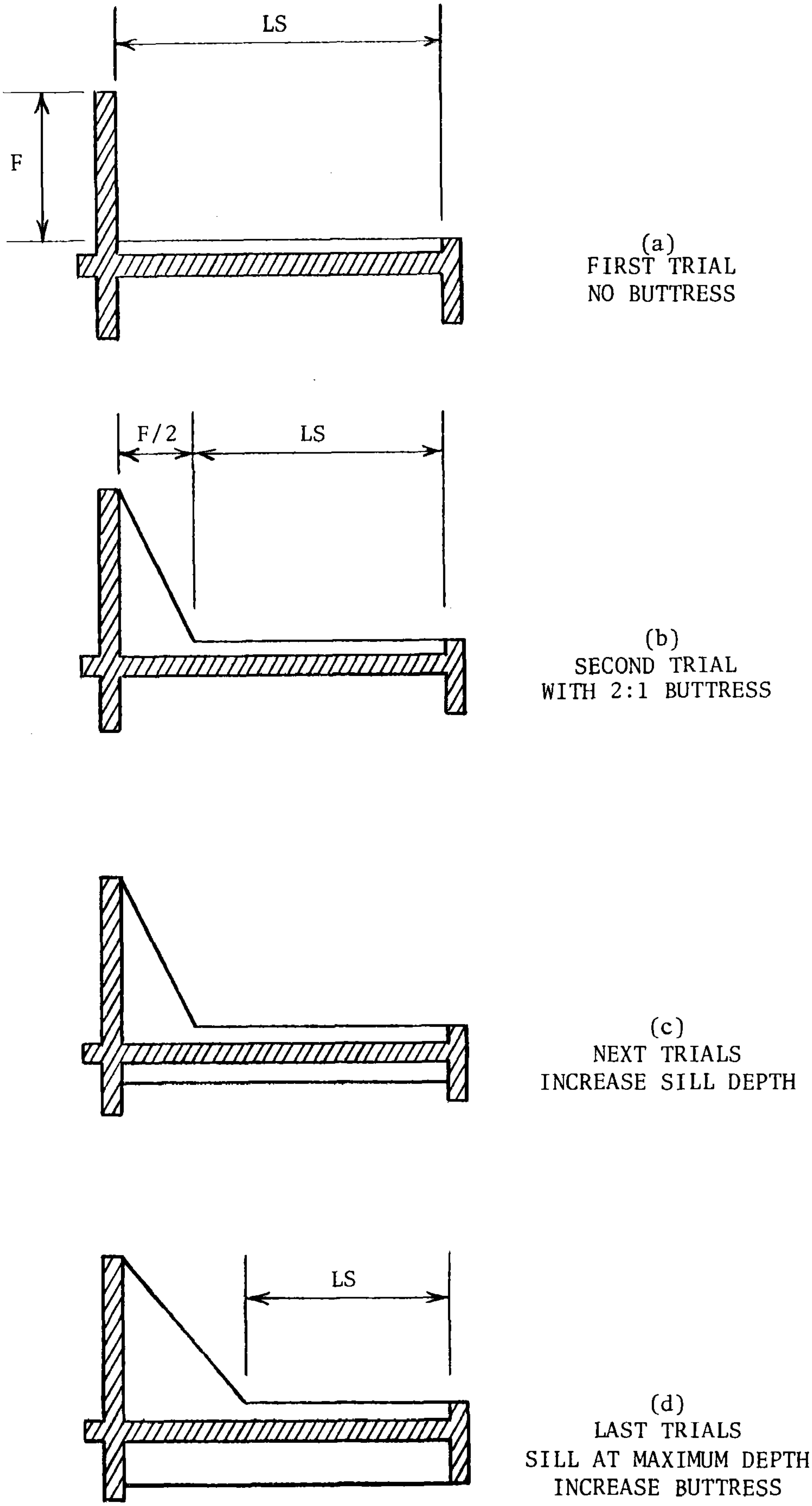


Figure 18. Development of longitudinal sill.

If SR ≧ SRMIN, the longitudinal sill of sketch (a) is adequate and a headwall buttress is not used unless required to effect a decrease in headwall thickness for the case. If SR < SRMIN, a headwall buttress is required. As shown in sketch (b), a buttress with

BUTT = F/2

is tried. Thus b is reset as

b = LS = LB - TTOE/12 - F/2

and βb and SR are recomputed. If SR ≧ SRMIN, the longitudinal sill of sketch (b) is adequate. If SR < SRMIN, a series of trials follows in which a dropped portion is added to the sill section as shown in sketch (c). In any trial compute

YL for the typical section of Figure 17,
I_2,
βb, and
SR.

The process is stopped for any SR ≧ SRMIN, and hence BOTT = YL. If necessary, the process is continued up to a maximum

BOTT = YL = HTOE - TL.

If for the maximum BOTT, SR < SRMIN, then another series of trials follows in which the buttress length, BUTT, is progressively increased, thus decreasing the longitudinal sill span, LS. Eventually an adequately stiff sill must be obtained since stiffness increases as span decreases. In any trial compute

b = LS = LB - TTOE/12 - BUTT
βb, and
SR.

The longitudinal sill dimensions are thus determined as sets of TBLS, BUTT, and BOTT.

Possible effect on line of creep. -- It should be recognized that the use of a dropped portion, BOTT > 0, as a part of the longitudinal sill has one detrimental effect. The weighted creep distance is reduced in the longitudinal cross section containing the sill. For small values of BOTT, the effect is probably not significant. However for large values of BOTT, the user should be aware that a shorter creep path is created than was assumed in the determination of HCUT. In those instances where the creep ratio is critical and where BOTT equals or approaches HTOEN, it may be necessary to take precautions in the immediate vicinity of the longitudinal sill. As an example, it may be advisable to surround the dropped portion with a more resistant material than the general foundation soil.

Transverse sill and apron bending. This section assumes the longitudinal sill(s), if any, is adequately stiff as described in the previous section. In the case of no longitudinal sill, adequate stiffness of the transverse sill depends on the interaction of the transverse sill and longitudinal apron bending. In the presence of a longitudinal sill(s), adequate stiffness of the transverse sill is a function of the interaction of the longitudinal sill(s) with both the transverse sill and transverse apron bending.

No longitudinal sill. -- When no longitudinal sill exists, the analysis for determination of required transverse sill dimensions is very similar to the preceding analyses for longitudinal sill dimensions. The apron is assumed subjected to uniform loading, is constructed of one-way longitudinal strips, and each strip is fixed at the headwall and rests on an elastic beam at the downstream end, i.e., the transverse sill. Figure 19 shows the assumed construction, resulting loadings and displacements.

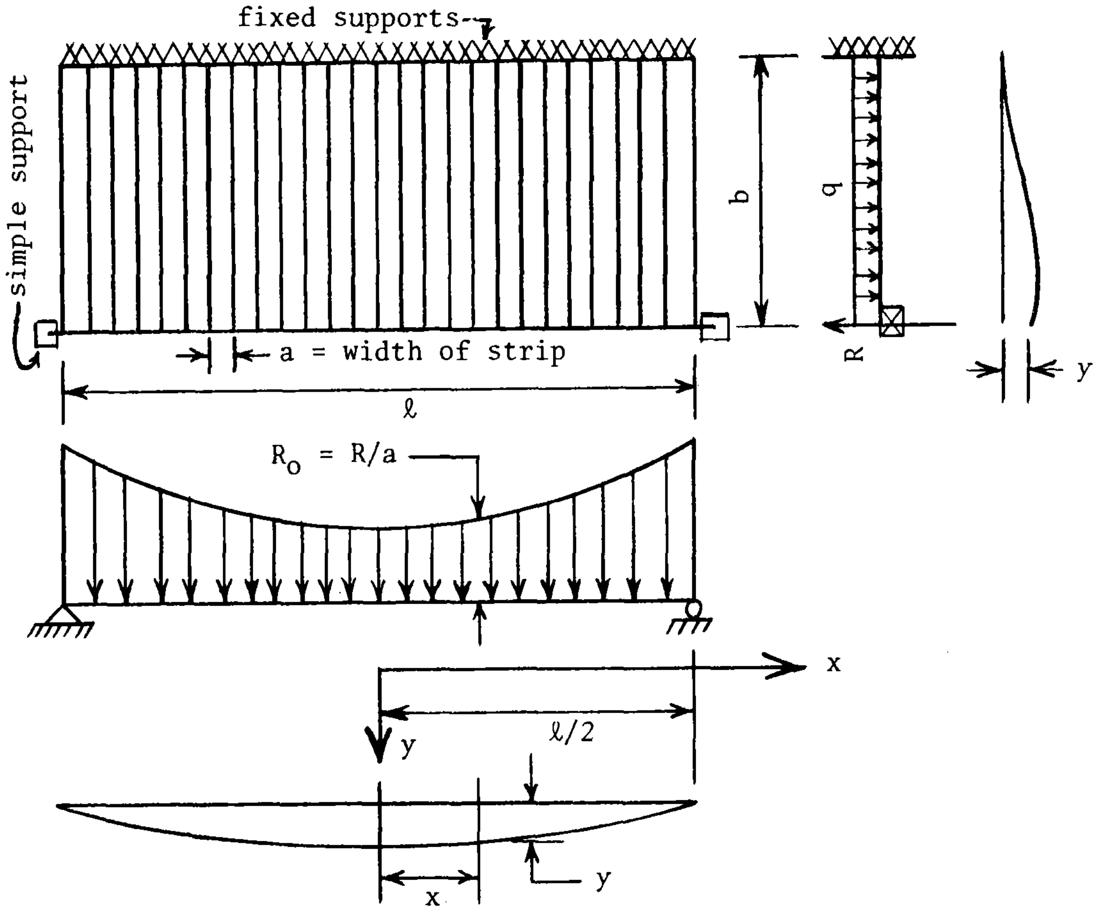


Figure 19. Transverse sill analysis - no longitudinal sills.

Paralleling the longitudinal sill analysis

$$y = \frac{1}{8}\frac{qb^4}{EI_1} - \frac{Rb^3}{3EI_1}$$

so

$$R_o = q_o - ky \quad \text{where } q_o = \frac{3}{8}\frac{qb}{a} \text{ and } k = \frac{3EI_1}{a\ b^3}$$

and

$$R_o)_{x=o} = (\frac{3}{8}\, wb)(\frac{2 \cos \beta\ell/2 \cosh \beta\ell/2}{\cos \beta\ell + \cosh \beta\ell})$$

where

$$\beta\ell = \left\{\frac{3}{4} \cdot \frac{\ell^4}{ab^3} \cdot \frac{I_1}{I_2}\right\}^{1/4} = \left\{\frac{1}{16} \cdot (\frac{t}{b})^3 \cdot \frac{1}{I_2}\right\}^{1/4} \cdot \ell$$

and so, for transverse sills

$$SR = \frac{2 \cos \beta\ell/2 \cosh \beta\ell/2}{\cos \beta\ell + \cosh \beta\ell}$$

Figure 20 shows a single span beam, fixed at one end receiving varying support at the other end. The beam is used to examine the way longitudinal apron moments vary with transverse sill reactive support. From the

Figure 20. Effect of reactive support on moments.

moment diagram, M_2 increases and M_1 decreases as R_o decreases. Thus for the end moment, $M \equiv M_2$

$$R_o)_{non} = \frac{3}{8}\, wb \quad \text{and} \quad M)_{non} = \frac{1}{8}\, wb^2$$

and for any R_o

$$M = \frac{1}{2}\, wb^2 - R_ob$$

so

$$RM = M/M)_{non} = \frac{\frac{1}{2}\, wb^2 - R_ob}{\frac{1}{8}\, wb^2}$$

or

$$R_o = \frac{wb}{8}(4 - RM)$$

and

$$SR = R_o/R_o)_{non} = \frac{4 - RM}{3}$$

Again using RM = 1.5 as an acceptable maximum value, the stiffness ratio for an allowable design must equal or exceed SR = 0.833 for a transverse sill with no longitudinal sill.

Two longitudinal sills. -- When longitudinal sills are used, the transverse sill functions to provide an adequate support for the longitudinal sill. The longitudinal sill analyses, for which it was found that SR = 0.977 is required, assumes the one-way transverse apron strips are independent elements and that the transverse sill provides an essentially non-yielding support. Thus the loading on the longitudinal sill is nearly constant at $(11/10)(w\ell)$ as shown in sketch (b) of Figure 21. In reality, the transverse sill deflects a small amount, δ, at the load points, so that the longitudinal sill loading is more like that shown in sketch (d). Thus for the apron strip adjacent to the transverse sill, the deflection, δ, is

$$\delta = \frac{11}{12}\frac{q\ell^4}{EI_1} - \frac{5}{6}\frac{R\ell^3}{EI_1}$$

On the assumption that the support provided the apron strip adjacent to the transverse sill should not be less than previously found adequate for strips in the middle of the apron, the reactive support should be

$$R \geqq SR \times R_{non} \geqq SR\ \left(\frac{11}{10}\ q\ell\right)$$

so that

$$\delta \leqq \frac{11}{12}\frac{q\ell^4}{EI_1}(1 - SR)$$

The transverse sill deflection under the two longitudinal sill loads is

$$\delta = \frac{5}{6}\frac{W\ell^3}{EI_2}$$

Since the deflection of the apron strip adjacent to the transverse sill and the deflection of the transverse sill must be equal at the longitudinal sill locations, the displacement expressions are equated, and

$$\frac{5}{6}\frac{W\ell^3}{EI_2} = \frac{11}{12}\frac{q\ell^4}{EI_1}\ (1 - SR)$$

With $R \geqq SR \times R_{non}$ required as above, the loading on the longitudinal sill is again nearly constant, so that

$$W \simeq W_{non} \simeq \frac{1}{2}\ \left(\frac{11}{10}\ w\ell\right)b$$

therefore

$$\frac{5}{6} \cdot \frac{1}{2} \cdot \frac{11}{10}\frac{w\ell^4 b}{EI_2} = \frac{11}{12}\frac{q\ell^4}{EI_1}(1 - SR)$$

or

$$\frac{b}{2I_2} \quad \frac{a}{I_1}(1 - SR)$$

Rearranging

$$I_2 \geqq \frac{1}{2}\left\{\frac{1}{12}\ bt^3\right\}\left\{\frac{1}{1 - SR}\right\}$$

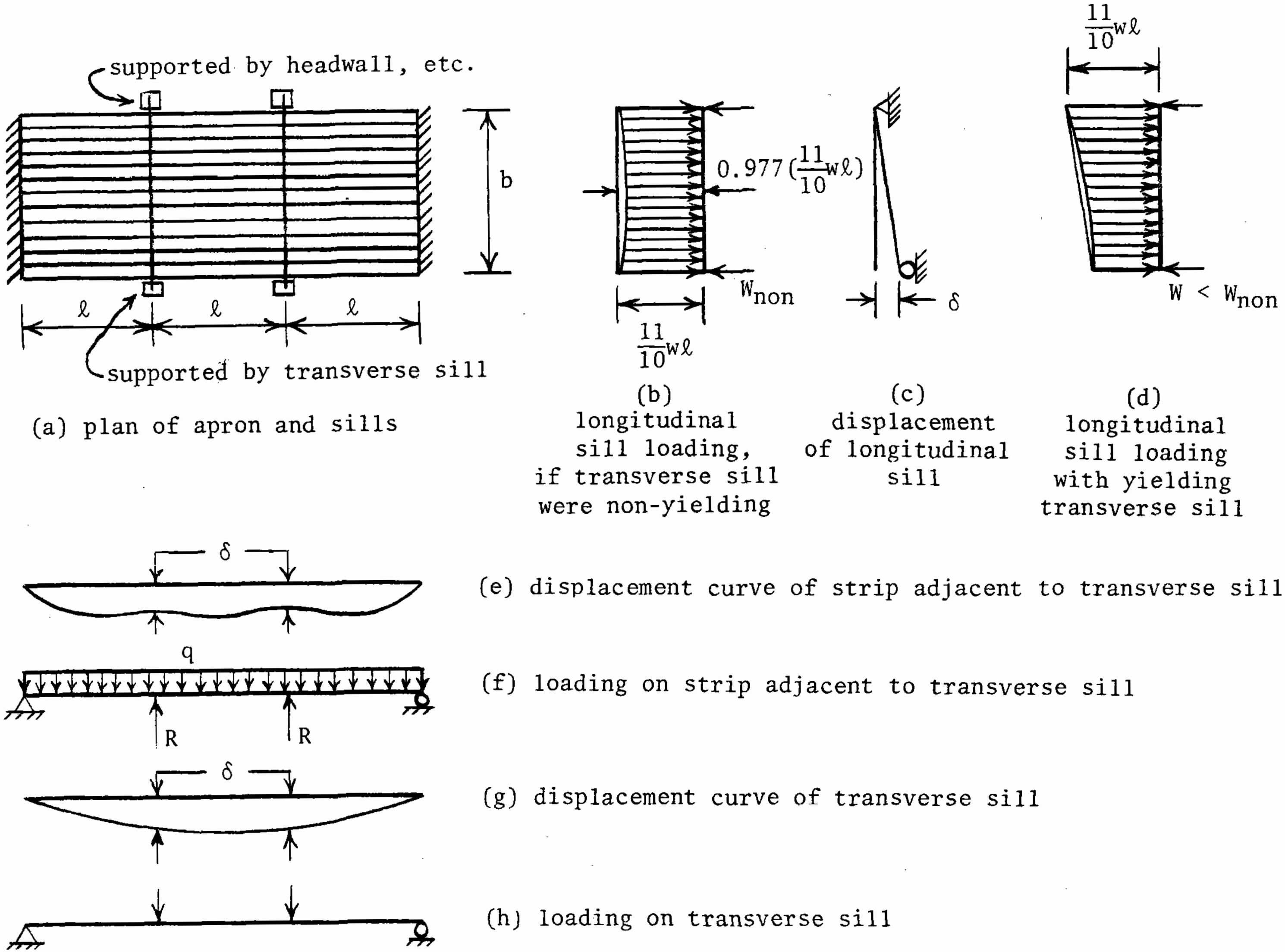


Figure 21. Transverse sill support of longitudinal sills.

The requirement for moment of inertia of the transverse sill, shown on page 39, can be relaxed somewhat on recognition that the apron strip adjacent to the transverse sill actually bends with the transverse sill rather than as an independent element. Assuming the critical transverse apron strip is located about 2/3 of b from the headwall, the deflection of the apron strip at the longitudinal sill locations is approximately 2/3 of the deflection of the transverse sill at its load points. Thus the transverse sill will be adequately stiff, if its moment of inertia, I_2, in ft^4 is

$$I_2 \geqq \frac{1}{3}\left\{\frac{1}{12}\,bt^3\right\}\left\{\frac{1}{1 - SR}\right\}$$

where SR = 0.977 for two longitudinal sills.

One longitudinal sill. -- The analysis for required transverse sill when one longitudinal sill is used, is closely the same as when two longitudinal sills are used. Following similar reasoning, the required moment of inertia, I_2 , is also

$$I_2 \geqq \frac{1}{3}\left\{\frac{1}{12}\,bt^3\right\}\left\{\frac{1}{1 - SR}\right\}$$

but with SR = 0.900 for the one longitudinal sill case.

Sizing the transverse sill. -- A transverse sill cross section is shown in Figure 22. Sill dimensions are suffixed with a T, and

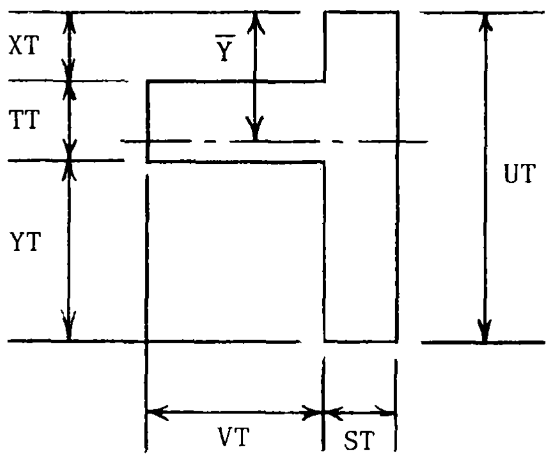


Figure 22. Transverse sill cross section.

all values are in ft. Thus

TT = TAP1/12, TAP2/12, or TAP3/12
ST = TTOE/12
XT = S
YT = HTOE - TT
VT = YT but not more than 4 x TT.

The procedure for checking the adequacy of the given set of TTOE and HTOE and for determining an adequate set of TTOE and HTOE differs depending on the case under consideration.

For the case of no longitudinal sill, let

$SRMIN = 0.833$
$\ell = L$
$t = TAP1/12$
$b = LB - TTOE/12$

then compute

$$I_2 = \bar{I} - ST(TT)^3/12$$

where

$$A = ST(UT) + VT(TT)$$

$$\bar{Y} = (ST(UT)^2/2 + VT(TT)(XT + TT/2))/A$$

$$\bar{I} = ST(UT)^3/3 + VT(TT)^3/12 + VT(TT)(XT + TT/2)^2 - A\bar{Y}^2$$

and compute

$$\beta\ell = \left\{ \frac{1}{16} \cdot \left(\frac{t}{b}\right)^3 \cdot \frac{1}{I_2} \right\}^{1/4} \cdot \ell$$

$$SR = \frac{2 \cos \beta\ell/2 \cosh \beta\ell/2}{\cos \beta\ell + \cosh \beta\ell}$$

If $SR \geqq SRMIN$ the transverse sill is adequate.
If $SR < SRMIN$, increment TTOE and HTOE, recompute ST, YT, and VT and recycle the process until a satisfactory sill is obtained.

For either one or two longitudinal sills, let, as appropriate,

$SR = 0.900$ or 0.977
$b = LB - TTOE/12$
$t = TAP2/12$ or $TAP3/12$

then compute the required I_2 as

$$I_2 \text{ req'd} = \frac{1}{36} bt^3 \left(\frac{1}{1 - SR}\right)$$

and compute the actual I_2 as

$$I_2 \text{ act.} = \bar{I} - ST(TT)^3/12$$

as shown above. If I_2 act. $\geqq I_2$ req'd., the transverse sill is adequate as a support for the longitudinal sill. If I_2 act $< I_2$ req'd., increment TTOE and HTOE, and recycle until a satisfactory sill is obtained. Quite arbitrarily the transverse sill is also checked, and if necessary made adequate, for the no longitudinal sill case but with $\ell = L/2$ or $L/3$ as appropriate.

The transverse sill dimensions are thus determined as sets of TTOE and HTOE. If for any case, TTOE and HTOE require incrementing in excess of

TTOE = 24. and HTOE = 8.,

the design for that case is abandoned.

<u>Comparison of apron systems</u>. If the designer has not designated which design he prefers, relative concrete volumes for the three possible apron systems are computed. The system requiring least concrete volume is selected for subsequent design treatment. The drop spillway components included in the concrete volume calculations are:

(1) headwall - taken as (FPS + HCUT deep) x (corresponding THW1, THW2, or THW3 thick) x (L + 8 long)

(2) toewall - taken as (corresponding HTOE + S deep) x (corresponding TTOE thick) x (L + 2 (TSW/12) long)

(3) apron slab - taken as (LB + 1. - TTOE/12 wide) x (corresponding TAP1, TAP2, or TAP3 thick) x (L + 2(TSW/12) long)

(4) longitudinal sill(s), if any - including buttress, raised portion of sill, and dropped portion of sill, as appropriate.

From the preceding analyses, several observations are possible. For drop spillways with large ratios of LB/L, the apron slab wants to be one-way transversely with no longitudinal sills, and the required transverse sill is small. If longitudinal sills are used when LB/L is large, they must be substantial to match the significant stiffness of transverse apron spans, and in this case the transverse sill would also need to be rather large. For drop spillways with small ratios of LB/L, the apron slab wants to be one-way longitudinally with no longitudinal sills, however a large transverse sill may be required to match the significant stiffness of the longitudinal apron span. If longitudinal sills are used when LB/L is small, they can be rather small since the stiffness of the transverse apron spans is also small, and in this case the transverse sill can also be comparatively small.

<u>Uplift</u>

Weighted creep theory is used to determine uplift under the drop spillway. Uplift pressures and forces are required in flotation calculations, in bearing pressure determinations, and in various flexural computations. Figure 23 presents a typical loading case and gives the corresponding uplift diagram referenced to the bottom of the apron slab. With uplift referenced this way, the portions of the cutoff wall and toewall below the apron are taken at their buoyant weights.

From the figure, with distances in feet and thicknesses in inches

HWFTGN = HWFTG - (TCUT - THW)/24
HCUTN = HCUT - TAP/12
HTOEN = HTOE - TAP/12
LTOT = HWFTG + THW/12 + LB
HDIFF = HEAD - TAILPS

The total weighted creep distance is

TWCD = 2 x (HTOEN + HCUTN) + LTOT/3.

The change in uplift head per foot of weighted creep distance is

CHFT = HDIFF/TWCD

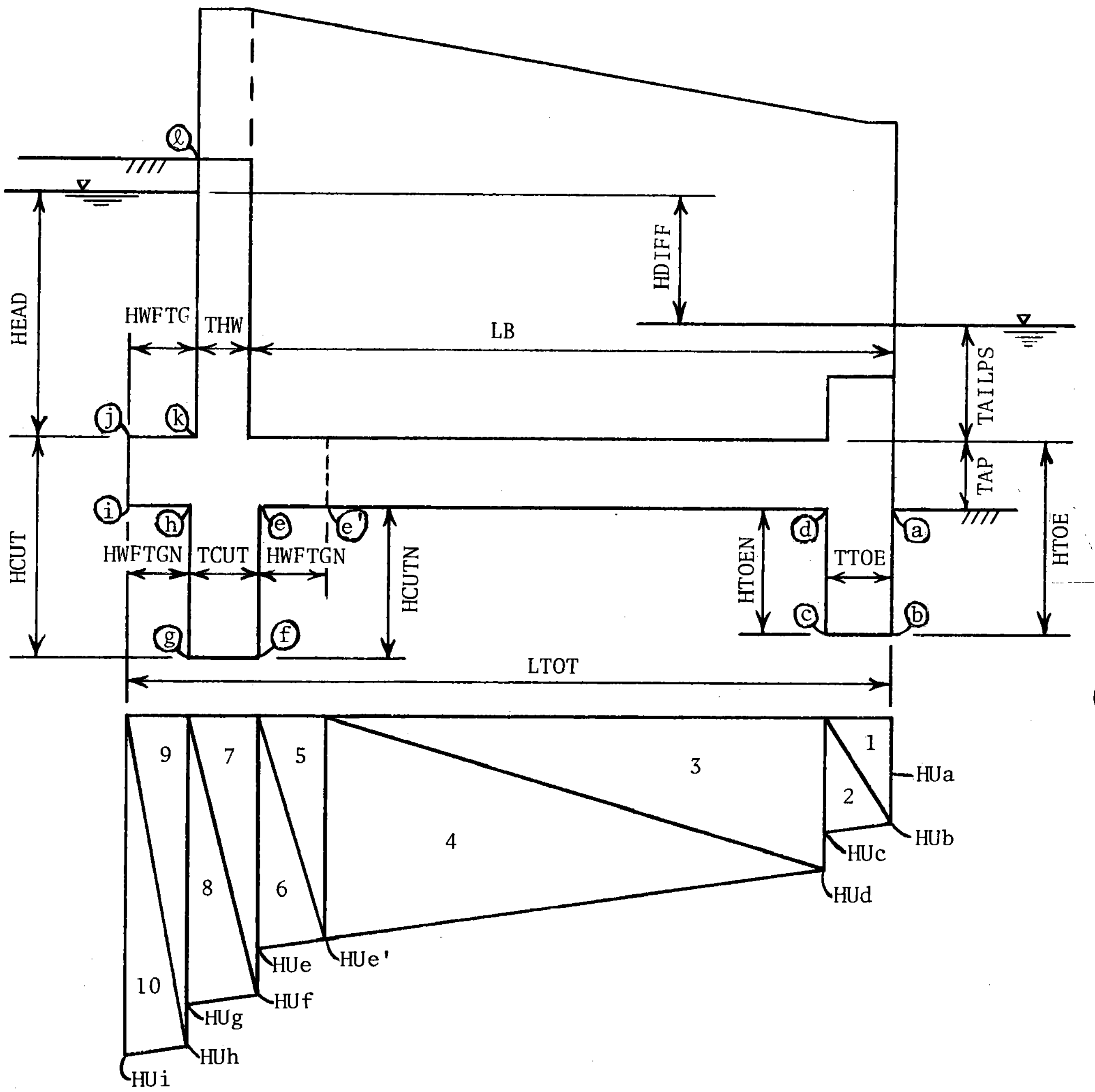


Figure 23. Drop spillway uplift diagram.

So the uplift heads at the designated points in Figure 23 are, in ft of water

HUa = TAILPS + TAP/12
HUb = HUa + HTOEN x CHFT
HUc = HUb + (TTOE/36) x CHFT
HUd = HUc + HTOEN x CHFT
HUe' = HUd + ((LB - HWFTG - TTOE/12)/3) x CHFT
HUe = HUe' + (HWFTGN/3) x CHFT
HUf = HUe + HCUTN x CHFT
HUg = HUf + (TCUT/36) x CHFT
HUh = HUg + HCUTN x CHFT
HUi = HUh + (HWFTGN/3) x CHFT

also

HUi = HEAD + TAP/12

Flotation

Starting with flotation analyses, computations assume the number of longitudinal sills, from none to two, and the associated dimensions and thicknesses have been selected. This selection was either on the basis of least concrete volume or user designation.

Each of the loading cases, M = 1 through 6, is checked for flotation since in a general sense any might control. The downward forces consist of

(a) the weight of the spillway concrete,

(b) the weight of the water in the spillway (the water surface is assumed level at TAILPS above the apron slab, if TAILPS is negative it is taken equal to zero), and

(c) the downward loads acting on headwall footing, headwall extension stub footings, and sidewall footings if present.

Uplift forces are computed by subdividing the uplift diagram as indicated in Figure 23.

Values established prior to flotation analyses are:

TCUT = larger of THW or TTOE
HWFTG = 1.0 + (FPSPH - 12)/8 but not less than 1.0 ft
SWFTG = 0.0
HESTUB = 4.0

For each loading case, the sum of all downward forces, SDOWN, and the sum of the uplift forces, SUP, must satisfy the relation

$$\frac{\text{SDOWN}}{\text{SUP}} \geqq \text{FLOATR}$$

If the flotation requirement is not satisfied, a series of trials is begun in an attempt to increase the downward forces sufficiently. The order of incrementing is

(a) increase HWFTG to 2.0 ft,

(b) increase HWFTG by 0.5 ft increments up to a maximum of HESTUBN = HESTUB - TSW/12,

(c) increase SWFTG by 0.5 ft increments up to a maximum of HESTUBN,

(d) increase TAP by 1.0 inch increments a maximum of 10 times.
If the flotation criteria remains unsatisfied, after all attempts, the design is abandoned.

If, for the selected design, TAP was incremented to satisfy flotation, a number of actions are instituted. First, flotation analyses are performed on the other two possible design configurations. Thus a new set of required apron slab thicknesses is obtained. Next, with the new thicknesses, the preliminary design procedure is recycled starting at the transverse sill analyses to determine new required sets of HTOE and TTOE. These actions are necessary because new TAP values may require new transverse and/or longitudinal sill values, and because the previously selected configuration may no longer be the one producing least concrete.

Bearing pressures

While estimates of maximum probable forces acting on various components of the drop spillway can be made with some degree of certainty, the actual pressure distributions developed to maintain equilibrium are often unknown. This is the case with the distributions of lateral pressures against the upstream and downstream faces of both the cutoff wall and the toewall. Hence in accordance with common practice, apron bearing pressures are obtained by neglecting any moments introduced by forces acting on these walls. That is, moments are taken about the moment center shown in Figure 24, and all forces below this elevation are neglected except for the submerged weights of the two walls and the dropped portion of any longitudinal sill.

The drop spillway is assumed to act as a monolith. Actually, dead load bearing pressures depend on the location of construction joints and the sequence of placing concrete. Resulting bearing pressure distributions and hence load, shear, and moment relations may thus take many forms, although the effect of this variability decreases for other loads in combination with dead load.

Figure 24 illustrates a typical situation. It is assumed that bearing (contact) pressures vary linearly along any section parallel to the longitudinal centerline of the spillway, and that these pressures are constant along any section at right angles to the centerline. Earthfill and water surfaces in front of the headwall are assumed constant over the full out-to-out length of the headwall plus headwall extension stubs.

The spillway bearing area is shown in Figure 24. As indicated, the area is treated as two rectangles where, in ft

LCUT = L + 2 x HESTUB
LTOE = L + 2 x TSW/12 +2 x SWFTG
LHESW = HESTUB - TSW/12 - SWFTG

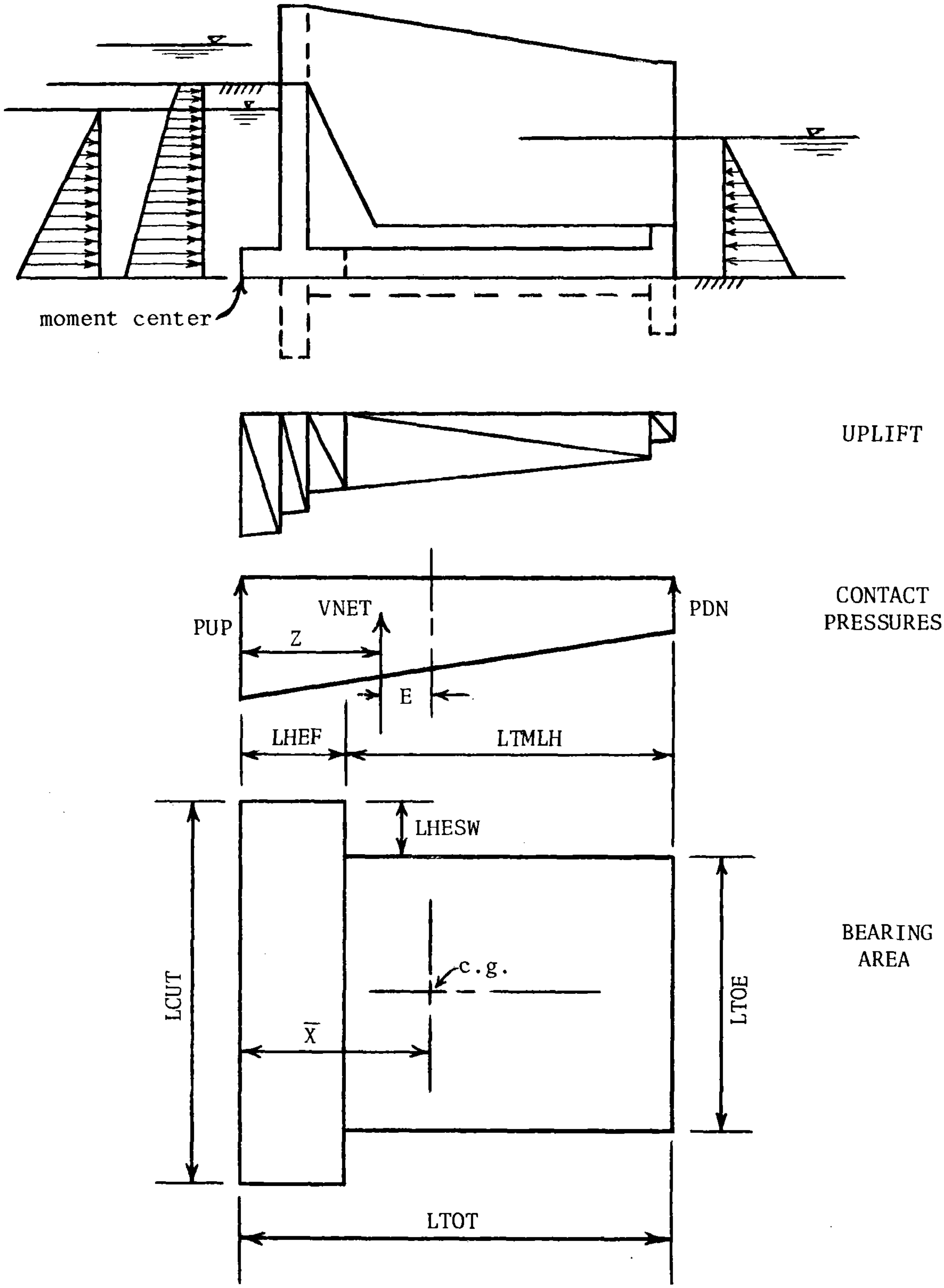


Figure 24. Determination of bearing pressures.

LTOT = HWFTG + THW/12 + LB
LHEF = THW/12 + 2 x HWFTG
LTMLH = LTOT - LHEF

Thus

$$A = LHEF \times LCUT + LTMLH \times LTOE$$
$$\overline{X} = (LCUT \times LHEF^2/2 + LTMLH \times LTOE \times (LHEF + LTMLH/2))/A$$
$$I = LCUT(LHEF)^3/3 + LTOE(LTMLH)^3/12 + LTOE(LTMLH)(LHEF + LTMLH/2)^2 - A\overline{X}^2$$

The location of the resultant of the vertical forces including uplift, VNET, is determined by summing moments about the indicated center of moments. The summation of moments includes the vertical forces considered in the flotation analyses, the lateral forces indicated in Figure 24, a lateral force acting upstream against the earthfill over the downstream headwall extension stub footing (acting on width LHESW), and a lateral force acting upstream against the earthfill over the sidewall footing at the downstream end of the spillway proper (acting on width SWFTG). These last two lateral forces are computed using KOW as the lateral earth pressure ratio. Thus, in lbs

VNET = SDOWN - SUP

and, in ft

$$Z = M/VNET$$
$$E = \overline{X} - Z$$

where M is the resultant moment about the moment center in ft lbs. Then, in psf

$$PAVER = VNET/A$$
$$PUP = PAVER + VNET \times E \times \overline{X}/I$$
$$PDN = PAVER - VNET \times E \times (LTOT - \overline{X})/I$$

Bearing pressures over the base must be everywhere compressive and within the allowable value. The allowable bearing, in psf, is taken as

PALLOW = 2000 + GBW x (HB + TAP/12)

Bearing pressures are computed for each of the loading cases, M = 1 through 8. If any PUP or PDN is negative, an attempt is made to increase the loading on the structure. This is done by incrementing either HWFTG or SWFTG. If any PUP or PDN exceeds the allowable bearing value, an attempt is made to spread the load on the structure. This also is done by incrementing either HWFTG or SWFTG. These footings can be incremented to a maximum value of HESTUBN. Whenever incrementing of the footings is necessary, the preliminary design procedure is recycled to the beginning of the flotation analyses. If bearing criteria remains unsatisfied after all possible incrementing, the design is abandoned.

Apron Slab Analyses

The next task, after determining satisfactory bearing pressures for each of the loading cases M = 1 through 8, is to check the adequacy of the current value of the apron slab thickness, TAP, for these loadings. As previously stated, apron design assumes two-way bending with essentially non-yielding supports.

Panel moments. Moments in apron slab panels are computed as a coefficient, CSS or CLS, times the particular one-way bending moment under investigation. The coefficient is a function of the ratio of short span to long span, SS/LS. The relations were selected after a study of the moment coefficients given in ACI 318-63, Appendix A, Methods 2 and 3 for the design of two-way slabs. Moments in the short direction increase to one-way values as SS/LS decreases to 0.5. They remain at one-way values for smaller SS/LS. Moments in the long direction reduce proportionately as SS/LS decreases. The coefficient relations are

$$CSS = 1.0 \qquad \text{for } SS/LS < 0.5$$
$$CSS = 1.4 - 0.8(SS/LS) \qquad \text{for } SS/LS \geqq 0.5$$
$$CLS = 0.6(SS/LS)^2 \qquad \text{for all } SS/LS$$

where

SS $\equiv$ short span
LS $\equiv$ long span

Note that by definition, SS may be in either the transverse or the longitudinal direction. The same is true for LS. Thus, with moments in ft lbs per ft, let

MS1 $\equiv$ one-way moment in the short direction
ML1 $\equiv$ one-way moment in the long direction
MSD $\equiv$ design moment in the short direction
MLD $\equiv$ design moment in the long direction

so

$$MS1 = f_1(SS)$$
$$MSD = CSS \times MS1$$
$$ML1 = f_2(LS)$$
$$MLD = CLS \times ML1$$

These moments apply to the middle strips of the apron panels. The distribution of moments from side to side of the panel is considered in detail design.

Apron longitudinal bending. The thickness of the apron slab may be governed by longitudinal moment or shear. The longitudinal span is treated as fixed at the headwall and both fixed and simply supported at the toewall. The loads on the slab consist of uniform loadings, due to tailwater over the apron and the weight of the slab, plus uniformly varying loadings, due to uplift pressures and bearing pressures, see Figure 25. Equivalent uniform and triangular loadings, PU and PT, are determined from these loads. The maximum longitudinal moment is one of the end moments. The maximum reaction occurs at the same section as maximum end moment. The moments and reactions of concern, for a span, LL, in ft, where

$$LL = LB - TTOE/12$$

are, in ft lbs per ft and lbs per ft

$$MHF = \left(\frac{1}{12} PU + \frac{1}{20} PT\right) \times LL^2$$

$$MTF = \left(\frac{1}{12} PU + \frac{1}{30} PT\right) \times LL^2$$

$$MHS = MHF + \frac{1}{2} MTF$$

$$RHF = \left(\frac{1}{2} PU + \frac{7}{20} PT\right) \times LL$$

$$RTF = \left(\frac{1}{2} PU + \frac{3}{20} PT\right) \times LL$$

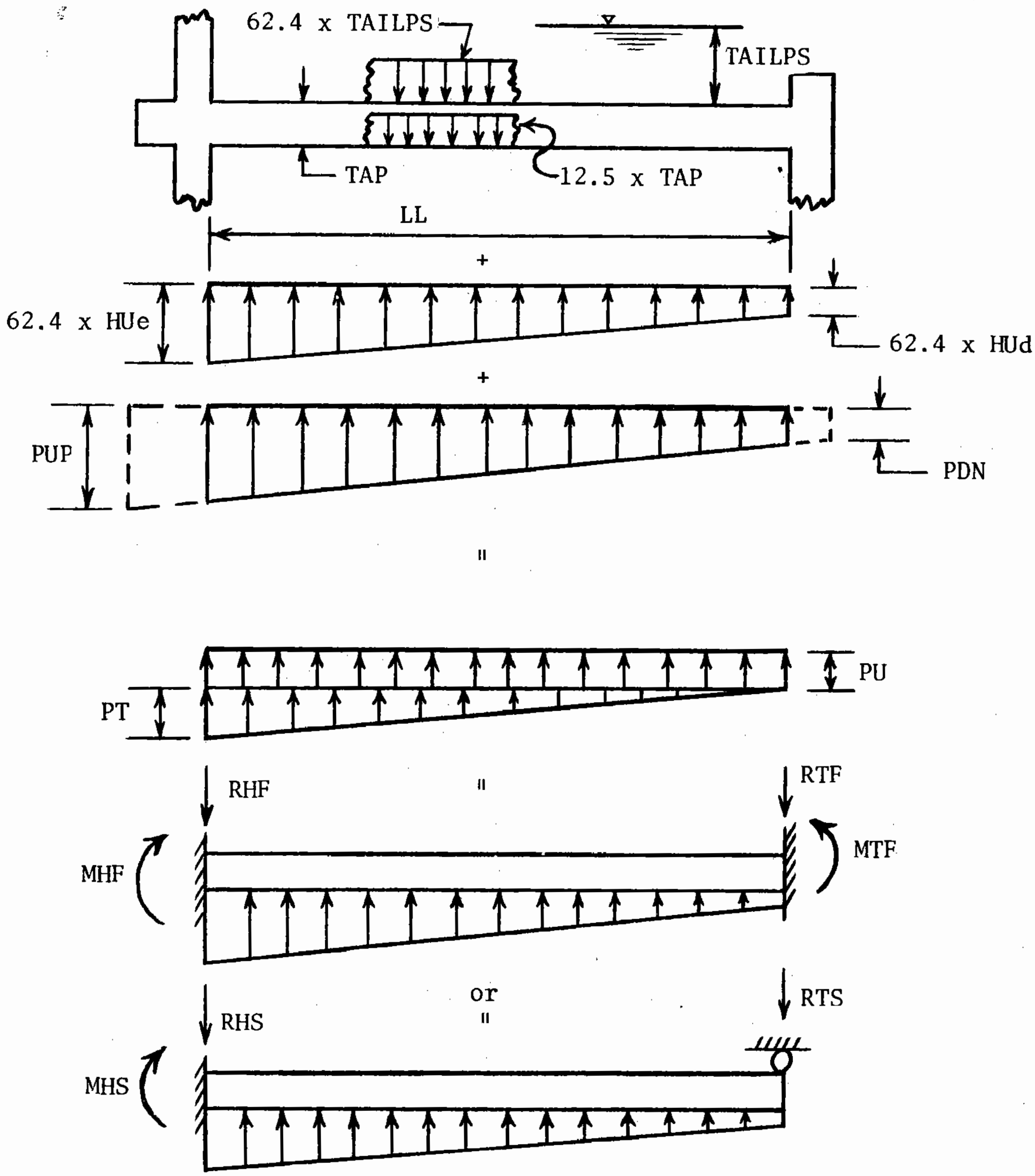


Figure 25. Longitudinal apron span loadings.

$$RHS = (\frac{5}{8} PU + \frac{2}{5} PT) \times LL$$

$$RTS = (\frac{3}{8} PU + \frac{1}{10} PT) \times LL$$

Slab shear is assumed in correspondence with panel moments. Hence the above one-way values are multiplied by CSS or CLS as appropriate.

Thus TAP is checked against the maximum moment and the associated critical shear at D from the face of the support. For tension on the bottom of the slab

D = TAP - 4.0

while for tension on the top of the slab

D = TAP - 3.0

Apron transverse bending. The thickness of the apron slab may be governed by transverse moment or shear. The bending loads on any transverse section of the slab consist of two parts, a uniform normal load and statical end moments. The uniform load, PN, is the algebraic sum of the loads due to bearing pressure, uplift pressure, weight of the slab, and weight of the tailwater over the apron. The statical moments, MS, are introduced to the apron along its longitudinal boundaries by the loads acting on the sidewall and the loads acting on the sidewall footing, if any.

Usually the required transverse thickness of the apron slab will not be more than the apron thickness required at the sidewall by the maximum statical moment. The maximum statical moment is assumed to occur at the same distance from the face of the headwall that produces critical vertical sidewall bending. From sidewall design, this distance is LV, where LV is the smaller of FPSPH/2 or FPS. The statical moment, MS, is actually an applied load that the slab must be capable of resisting. Therefore, MS is not reduced by the CSS or CLS coefficient for this calculation of required apron thickness.

The required transverse thickness of the apron slab is sometimes governed, not by the statical moment at the slab boundary, but by interior moments or shears. Thus for spillways without a longitudinal sill, required thicknesses are checked for moment at midspan and for shear at D from the sidewall. For spillways with one longitudinal sill, required thicknesses are checked for moment at the longitudinal sill and for shear at D from the sill. These moments and shears, including the effects of MS, are multiplied by the appropriate coefficient CSS or CLS. Figure 26 shows the three possible cases of J = 1, 2, or 3, where J is the number of transverse apron spans. The material given in Figure 26 is presented in more completeness than is warrented for preliminary design. This is done because the information is further used during detail design. From Figure 26, where, in ft

LCCS = L + 2(TSW - TSB/2)/12

the moments and reactions of interest are, in ft lbs per ft and lbs per ft, for J = 1

ℓ = LCCS

$M = MS - (PN/8)\ell^2$

$R = (PN/2)\ell$

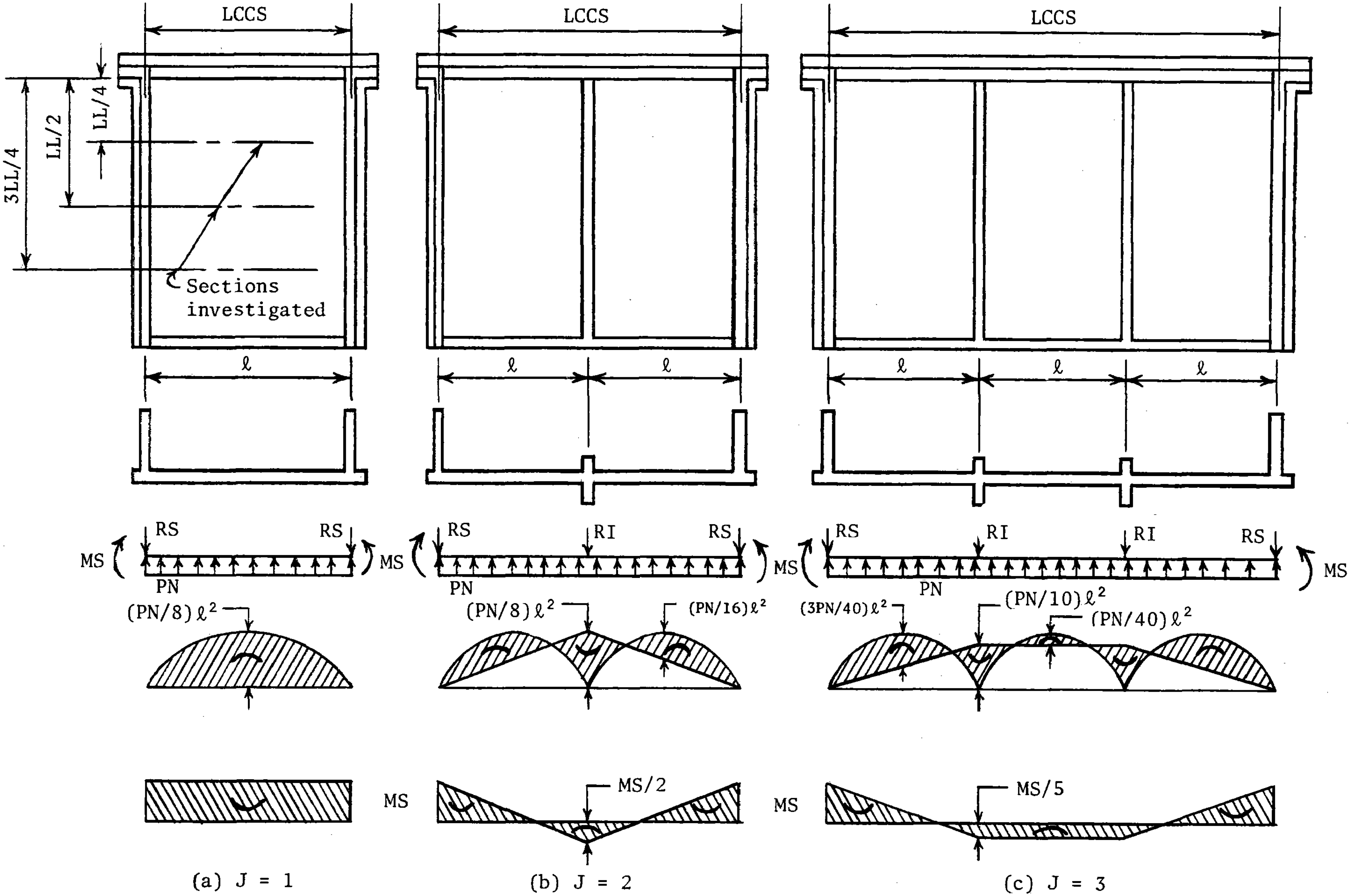


Figure 26. Transverse bending of the apron slab.

and for J = 2

ℓ = LCCS/2

M = - MS/2 + (PN/8)ℓ^2

R = (5PN/8)ℓ - 3MS/2

Required thicknesses for these moments and shears are checked at three transverse sections across the apron slab. These sections are located at distances LL/4, LL/2, and 3LL/4 from the face of the headwall. The effective depth for moments causing tension on the bottom of the slab is, in inches

D = TAP - 3.5

and for moments causing tension on the top of the slab is

D = TAP - 2.5

In checking the thicknesses required by interior transverse moments and shears, emphasis is on the moments and shears caused by the uniform load, PN, acting on the section under investigation. For these calculations, it is conservative to minimize the value used for MS. Thus if the section under investigation is located a distance, X, from the face of the headwall, and H < LHMAX (where LHMAX is defined in Figure 13), the value used for MS is taken as the computed value of MS at the distance LHMAX times the linear reduction ratio X/LHMAX. If X $\geq$ LHMAX, then MS is used at its full computed value at X.

If the current value of the apron slab thickness is found to be inadequate in any of the above apron slab analyses, TAP is increased to the indicated satisfactory value. Then the design is recycled to the transverse sill analyses for possible new transverse and/or longitudinal sill values and subsequent volume comparisons.

Sliding Analyses

Each of the loading cases, M = 1 through 6, is investigated for stability against sliding. Figure 27 illustrates a typical situation. As with bearing analyses, earthfill and water surfaces in front of the headwall are assumed constant over the full out-to-out length, LCUT, of the headwall plus headwall extension stubs. The lateral earth pressure ratio, KOH, is used above the bottom of the apron slab and KOF is used below

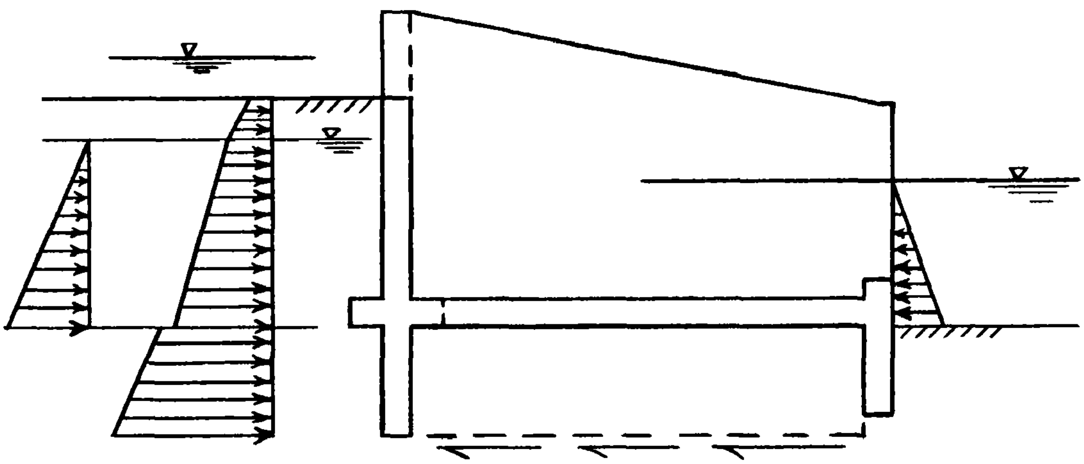


Figure 27. Sketch for sliding analyses.

the bottom of the apron slab. Both driving and resisting hydrostatic distributions are shown in Figure 27 to cease at the elevation of the bottom of the apron slab. This is of course untrue. Due to seepage, pressure differences between upstream and downstream sides of the cutoff wall and the toewall will usually exist. These pressure differences are neglected. This is done in the belief that, in view of other uncertainties, such refinement is unwarrented.

A horizontal plane of sliding is assumed at the elevation of the bottom of the cutoff wall. The buoyant weight of the foundation material above this plane and under the drop spillway is included as vertical weight contributing to the frictional resistance to sliding. As previously stated, the resistance of any channel material downstream of the toewall is neglected.

The resistance of the headwall extension stub to downstream sliding is depicted in Figure 28. Resisting lateral earth pressures are computed using the passive pressure ratio, KPW, above the elevation of the bottom of the apron slab and KPF below the elevation of the bottom of the apron slab. The hydrostatic resisting force is computed using a head, HWW, which is obtained from the heads, HSIDE and HWING. The hydrostatic distribution is shown as non-existent below the apron slab level as previously explained. The resisting pressures shown in Figure 28 act over the width, LHESW, shown in Figure 24.

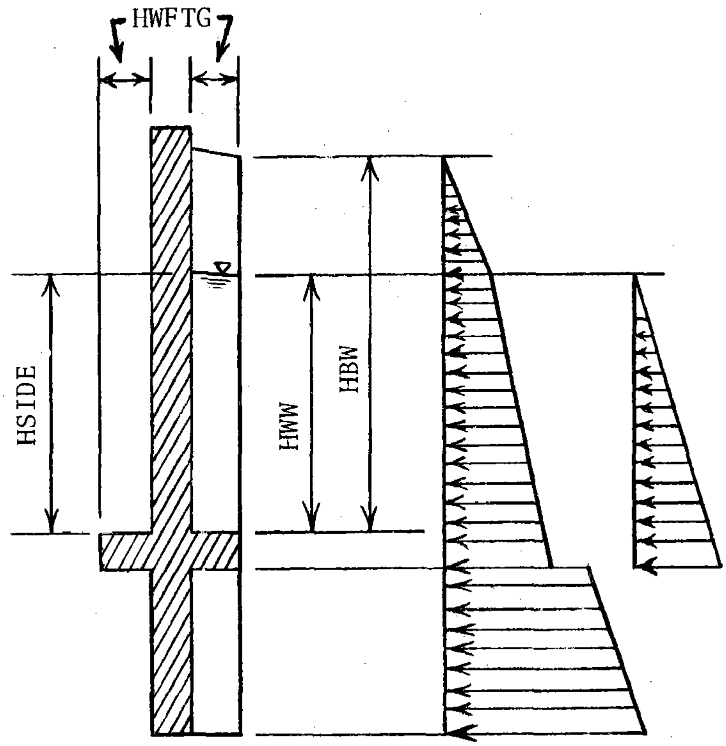


Figure 28. Section through headwall extension stub.

Another resisting lateral force remains to be considered. This is the lateral force acting against the earthfill over the sidewall footing at the downstream end of the sidewall footing. The lateral earth pressure ratio, KOW, is used since this earthfill simply rides with the sidewall footing.

For each loading case investigated, the relation

$$\frac{\text{VNET x CFSS}}{\text{HNET}} \geq \text{SLIDER}$$

must be satisfied, where VNET and HNET are the algebraic sums of the vertical and horizontal forces respectively. If for any loading, the relation is not satisfied, an attempt is made to increase the sliding resistance of the structure. This is done by first incrementing HWFTG which increases the weight on the structure. The maximum value of HWFTG is set at

HESTUBN = 4.0 - TSW/12.

If these attempts are unsuccessful, then the headwall extension stubs are increased by one foot increments. This increases the passive forces against the headwall extension stubs. Whenever incrementing is necessary, the preliminary design procedure is recycled to the beginning of the flotation analyses. If sliding criteria remains unsatisfied after all possible incrementing, the design is abandoned.

Headwall Extension Stub

The essential features of the analysis for checking the adequacy of the headwall extension stub thickness are indicated in Figure 29. This analysis is quite arbitrary. The critical section is taken a distance, YHES, below the top of the extension stub where

YHES = FPSPH - HESTUBN but not less than H.

The earthfill on the upstream side of the stub is taken level at crest elevation, so

YUP = YHES - H.

The earthfill on the downstream side of the stub is

HBW = HB + LB/ZPS but not more than FPSPH,

so

YDN = YHES - (FPSPH - HBW) but not less than YUP. Note that bending of opposite sense, that is YDN < YUP, is not treated. Water pressures each side of the stub are neglected. The lateral earth pressure ratio is taken as KOH on the upstream side and as KPW on the downstream side of the stub. Thus net uniform loading, shear at the face of the sidewall, and moment at the face of the sidewall are in psf, lbs per ft, and ft lbs per ft

PNET = KPW x GMW x YDN - KOH x GMH x YUP

V = PNET x HESTUBN

M = V x HESTUBN/2

With V and M known, the required stub thickness can be found. Here D = T - 2.5, where T is the required thickness.

If the headwall extension stub thickness required by either V or M exceeds the current thickness of the headwall, THW, the design must be recycled. First, all headwall thicknesses THW1, THW2, and THW3 are compared to the required stub thickness and increased if necessary. Then the design is recycled to the point of selecting the system resulting in least concrete volume.

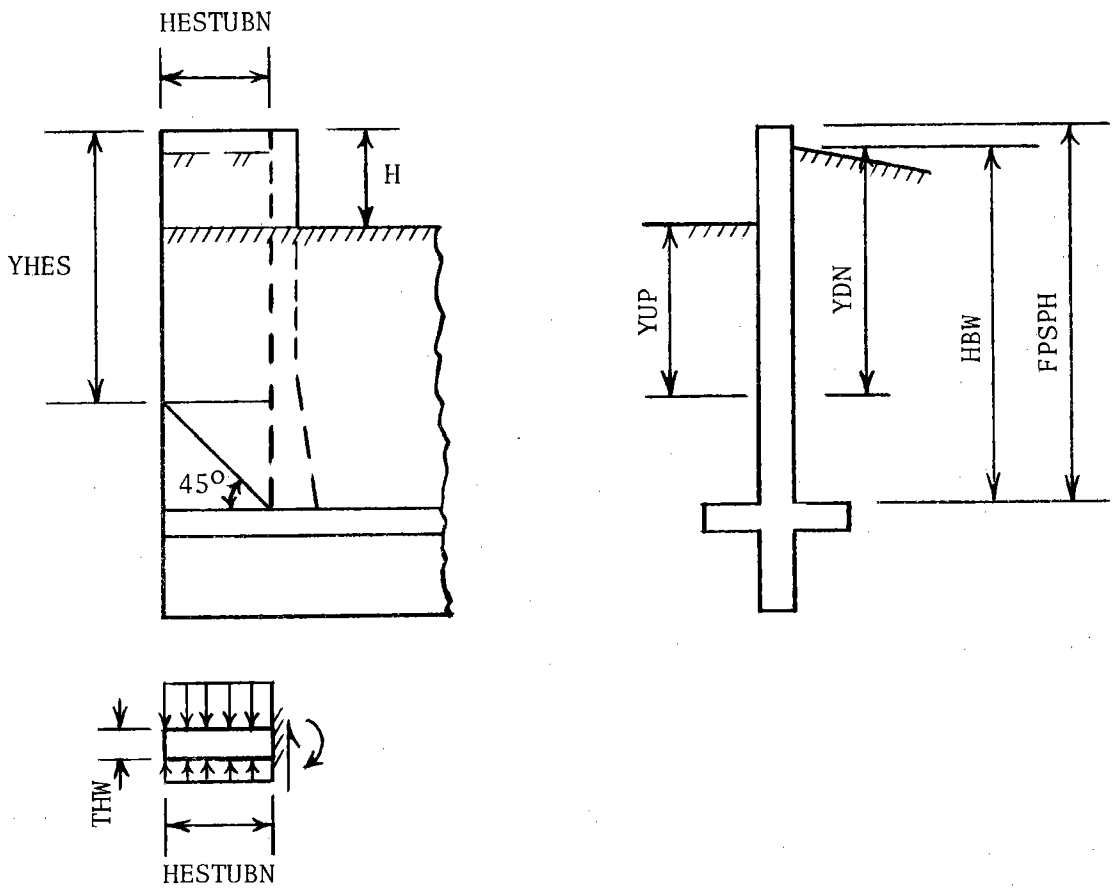


Figure 29. Headwall extension stub analysis.

Toewall Bending Analyses

The toewall thickness, TTOE, may need to be greater than either that originally established by user choice or default value, or that required in conjunction with adequate transverse sill stiffness. The analysis used to check the adequacy of TTOE is indicated in Figure 30. The toewall must be able to resist the cantilever bending that might be induced by passive resistance of the channel material downstream of the toewall. Assume passive earth pressure against the downstream side of the toewall and zero earth pressure against the upstream side. Neglect water pressures each side and use moist unit soil weight. Thus from Figure 30 the cantilever shear and moment, at the elevation of the bottom of the apron slab, are

$$V = KPF \times GMF \times HTOEN \times ((S + TAP/12) + HTOEN/2)$$

$$M = KPF \times GMF \times \overline{HTOEN}^2 \times ((S + TAP/12)/2 + HTOEN/3)$$

The minimum thicknesses, T, in inches are given by

$$T = V/(840. + (KPF \times GMF \times (S + TAP/12))/12) + 2.5$$

and

$$T = (0.003683 \times M)^{1/2} + 2.5$$

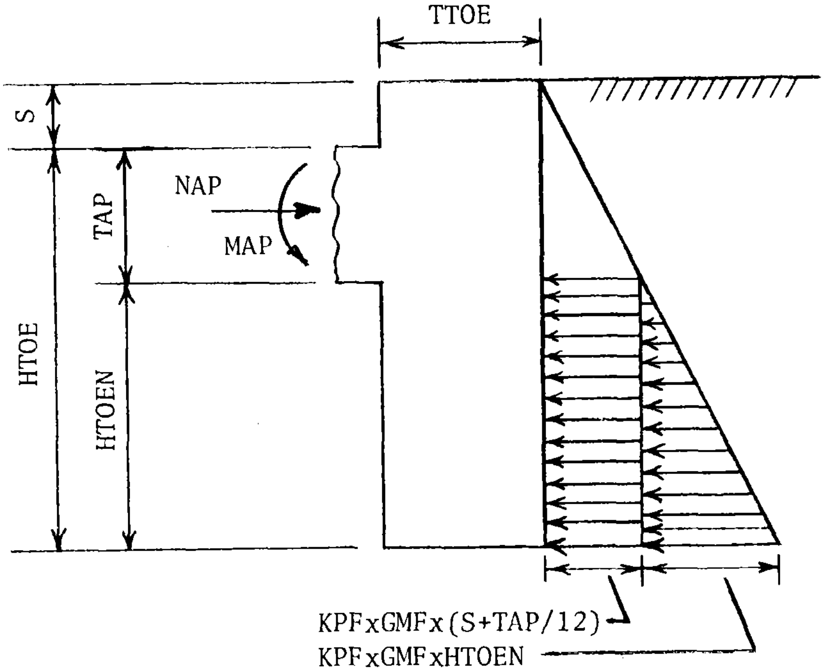


Figure 30. Toewall cantilever bending.

If either thickness, T, exceeds the current toewall thickness, the design must be recycled. First, TTOE is increased as needed. Next, TCUT is increased to TTOE, if necessary, since TCUT is not taken less than TTOE. Then the design is recycled to the beginning of the flotation analysis unless the apron thickness is inadequate (which takes precedence) as discussed below.

On the assumption that passive resistance may be developed on the downstream side of the toewall, the current thickness of the apron slab, TAP, may be insufficient to resist the moment and thrust, MAP and NAP, shown in Figure 30. The actual magnitudes of MAP and NAP are uncertain since the eccentric loading on the downstream side will be resisted in part by torsional resistance developed in the transverse sill. The following analysis conservatively assumes that the eccentric loading is entirely resisted by MAP and NAP. Thus

$$\begin{aligned} NAP &= 1/2 \times KPF \times GMF \times (S + HTOE)^2 \\ MAP &= NAP \times (2/3 \times HTOE - S/3 - TAP/24) \\ MS &= MAP + NAP \times (TAP/2 - 3.)/12 \\ &= NAP \times (2/3 \times HTOE - S/3 - 0.25) \end{aligned}$$

so that the required apron thickness is

$$T = (0.003682 \times MS)^{1/2} + 3.$$

If T exceeds the current TAP, the apron thickness is increased to T, and the design is recycled to the transverse sill analysis.

Cutoff Wall Bending Analyses

The cutoff wall provides a major share of the structure's ability to resist sliding. The magnitude and distribution of the resisting force acting against the cutoff wall is very uncertain. The resisting force, per ft of weir length, is therefore conservatively taken equal to the total driving force, per unit length, acting against a typical section of headwall. Figure 27 shows this driving force. Figure 31 repeats the driving force and shows the resisting force uniformly distributed over the net depth of the cutoff wall.

With the above assumptions, the required thickness of the cutoff wall for cantilever shear and moment, for each of the loading cases M = 1 through 6, is determined. Here, D = T - 3.0. If the required thickness exceeds the current thickness of the cutoff wall, the design must be recycled. TCUT is increased as necessary, and the design is recycled to the beginning of the flotation analyses unless the apron slab thickness is inadequate (which takes precedence) as discussed below.

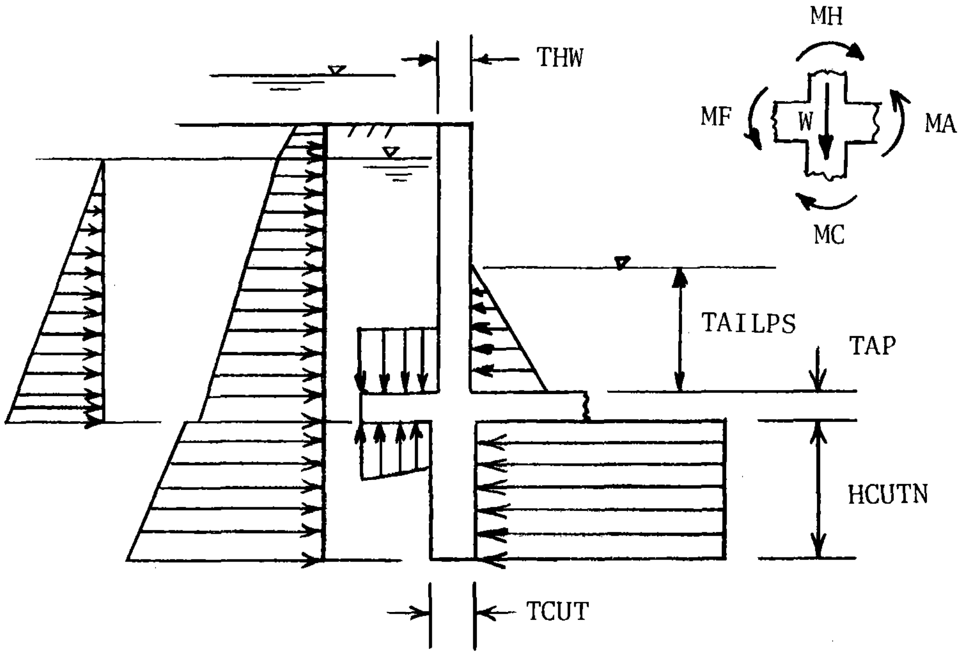


Figure 31. Cutoff wall bending

The bending of the cutoff wall, in accordance with the foregoing assumptions, together with associated flexure in the headwall and in the headwall footing causes a resisting moment to be developed in the apron slab at its interface with the headwall and cutoff wall. The resisting moment for any loading M = 1 - 6, is given by

$$MA = MC + MH - MF - W \times TCUT/24$$

as shown in Figure 31. All moments are for a unit width and

MC $\equiv$ moment due to forces on cutoff wall
MH $\equiv$ moment due to forces on headwall
MF $\equiv$ moment due to forces on the headwall footing, and
W $\equiv$ weight of headwall and buoyant weight of cutoff wall.

The moment contribution from the forces on the headwall is computed as the statical moment times the vertical moment coefficient, CMV, discussed in headwall analyses. Thus MH reflects the effect of horizontal bending in the headwall due to buttress and/or sidewall supports. With MA known, the required apron thickness is

$$T = (0.003682 \times MA)^{1/2} + 4.0$$

If T exceeds the current TAP, the apron thickness is increased to T, and the design is recycled to the transverse sill analyses.

Wingwalls

Design criteria and procedure for these straight drop spillway wingwalls parallels that for the SAF stilling basin wingwalls given in TR-54, "Structural Design of SAF Stilling Basins." Two adaptations of the material presented in TR-54 are necessary to make this accommodation. The first deals with earthfill slope parameters. The second deals with water loadings.

Refer to Figures 34 and 35, pages 44 and 45 of TR-54. Figure 34 shows a typical wingwall design section at a distance, X, from the articulation joint which separates wingwall from sidewall. Two earthfill slopes are shown in the figure. The first slope, ZNW, results directly from the earthfill against the wingwall. The second slope, ZNS, results from the earthfill against the sidewall and is thus related to the slope, ZPS. For SAF basins, ZPS is defined by J, HB, and the length of the basin. For drop spillways, ZPS is specified by the user and so is an independent variable. While the top surface of a SAF sidewall is horizontal, the top surface of a drop spillway sidewall is usually sloping. Figure 32 shows the slope of this surface, ZTOP, and the distance, XIN, which is the horizontal distance from the downstream end of the sidewall to the point of intersection of earthfill slope with the top surface of the sidewall. Thus the effective slope parameter, ZPSE, to be used in the design of drop spillway wingwalls should range between the specified ZPS and ZTOP. Apparently the slope ZPSE should be a function of ZPS, ZTOP, and XIN.

Thus

$$ZTOP = \frac{LB}{FPSPH - J}$$

If ZPS $\geqq$ ZTOP, then

$$ZPSE = ZPS$$

If ZPS < ZTOP, then from Figure 32, neglecting the 6 inch level distance at the downstream end of the sidewall

$$HSW = HB + XIN/ZPS \simeq J + XIN/ZTOP$$

or

$$XIN = \frac{J - HB}{1/ZPS - 1/ZTOP}$$

If XIN $\leqq \sqrt{2}J$, a parabolic transformation in effective slope parameter, ZPSE, is used, thus

$$ZPSE = ZPS + (ZTOP - ZPS)\left(\frac{\sqrt{2}J - XIN}{\sqrt{2}J}\right)^2$$

but if XIN > $\sqrt{2}$J

ZPSE = ZPS.

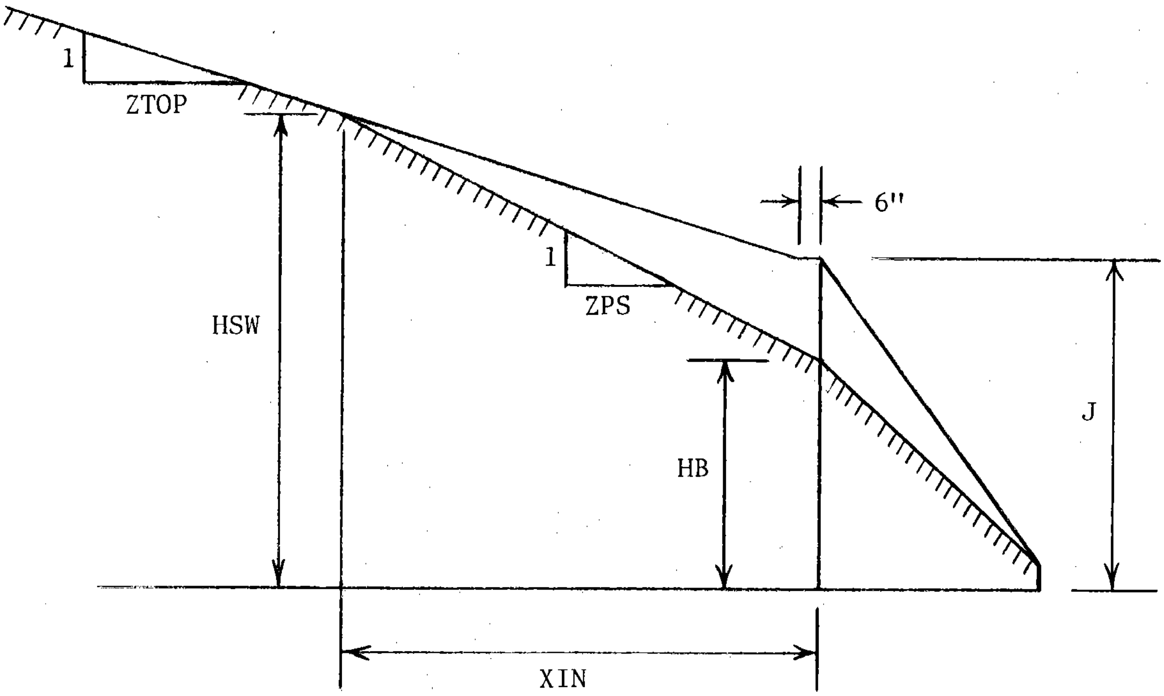


Figure 32. Adaptation of earthfill slope parameter.

The parabolic relation was selected because it results in a reasonable variation in wingwall design as HB values approach J from some small value. The $\sqrt{2}$J limit is arbitrary.

Refer to Figure 36, page 46 of TR-54. For drop spillway wingwalls, the loadings M = 1 through 7 are used instead of the five water loadings shown in Figure 36. In addition to the five loadings, the figure also shows a typical water loading situation. This sketch applies for any particular drop spillway loading if DW is redesignated as TAILPS and HW is redesignated as HWING.

With these adaptations, the criteria and procedures given in TR-54 apply to the design of these drop spillway wingwalls.

Detail Designs

Each detail design begins with the set of trial dimensions obtained in the preliminary design. Thicknesses are incremented, and the design of the involved structural component is recycled, whenever it is discovered compression steel would otherwise be required to hold bending stresses to allowable working values.

Controlling steel area and spacing, or perimeter, values are determined at numerous locations throughout the spillway. With few exceptions, each location is checked for each of the loadings, M = 1 through 8. The required steel area for moment and direct force, and the required steel spacing for shear are obtained as explained in TR-42.

Steel area and spacing envelopes are provided for headwall, sidewall, and apron slabs. These envelopes, together with computed minimum steel area and maximum steel spacing at particular slab locations, should adequately define necessary slab steel. Slab steel areas given always meet or exceed the temperature and shrinkage requirements given on page 12.

Minimum total steel area and perimeter are computed at critical locations in headwall buttresses, longitudinal sills, and transverse sills. Information is provided regarding the distribution of steel between the given points. These three structural components will often require web steel for diagonal tension. Indices are given from which web steel area and spacing may be determined, if web steel is required.

Various schematic steel layouts are included in the figures that follow. These layouts define the locations for which steel requirements are determined. The layouts also indicate the assumed orientation of the several steel grids in a structural component. The designer makes the actual choice of steel size, spacing, and layout. If he selects a layout that is significantly different from that assumed in design, he should recognize the associated effect on steel requirements.

No particular attempt has been made to indicate steel anchorage requirements in the following sketches. A basic premise is that every bar must be adequately developed each side of every section. Further, attention must be paid to the requirements of steel continuity at corners and between members. The spillway cannot perform as anticipated if unforeseen discontinuities are present. When web steel is required for buttresses, longitudinal sills, and/or transverse sills, it may be provided in any convenient form. The essentials are that it enclose the main tensile steel and be adequately anchored or developed.

Headwall Steel

The vertical steel requirement in the upstream face of a headwall panel is defined in Figure 33. Steel areas for moment and direct force are determined for the four points in the midspan vertical strip of panel. Steel spacing for flexural bond is determined for point 4 at the bottom of the strip. Panel moments and shears are computed as explained in preliminary design. The vertical distribution of this strip steel is given in Figure 33 by the vertical envelopes at midspan. A suggested transverse distribution of vertical steel in the panel is indicated by the horizontal envelopes for steel at the bottom of the panel.

Headwall Panel Values

$A \equiv FPS = F + S \qquad B \equiv L, \text{ or } L/2, \text{ or } L/3$

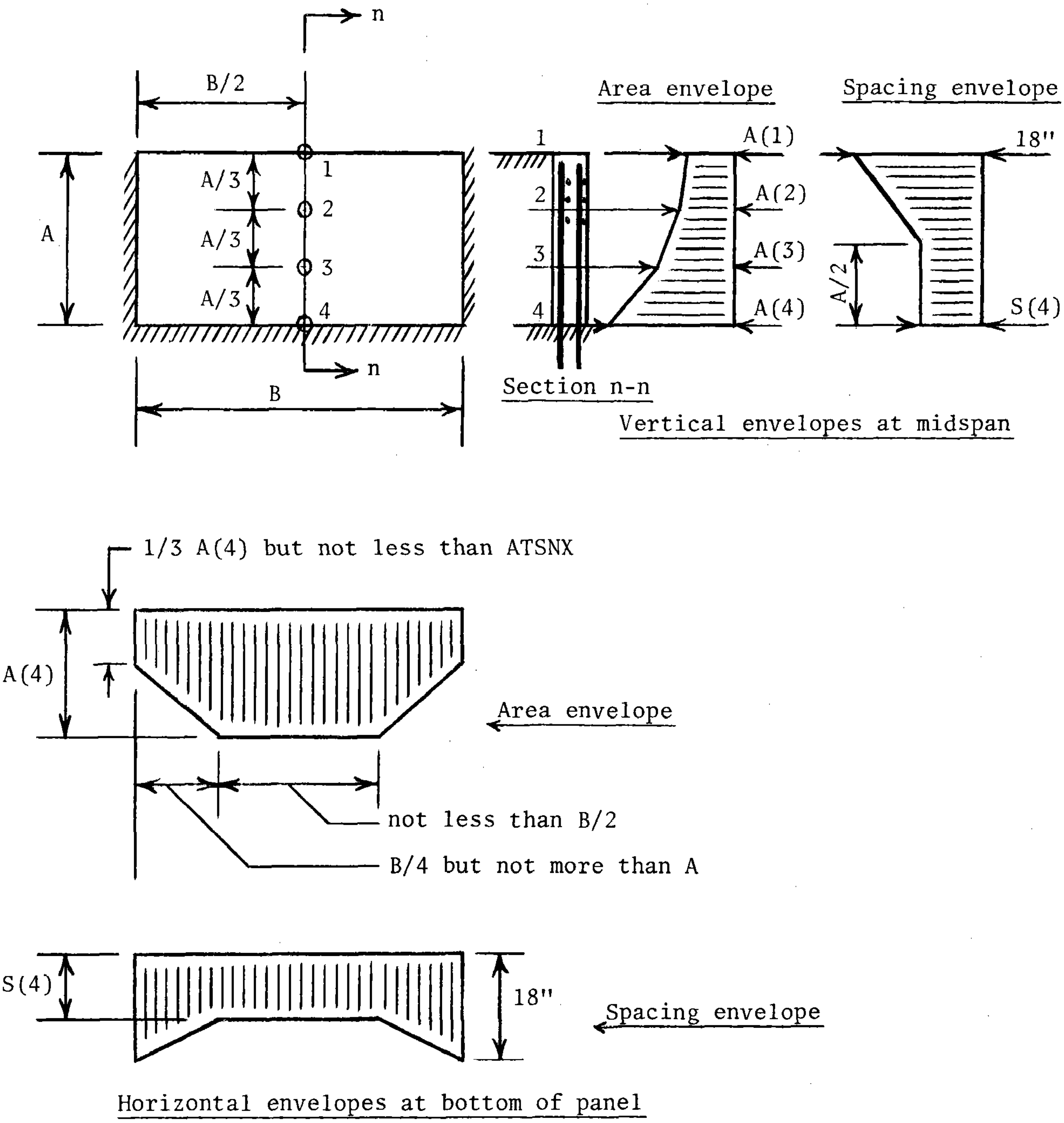


Figure 33. Vertical steel in headwall panel.

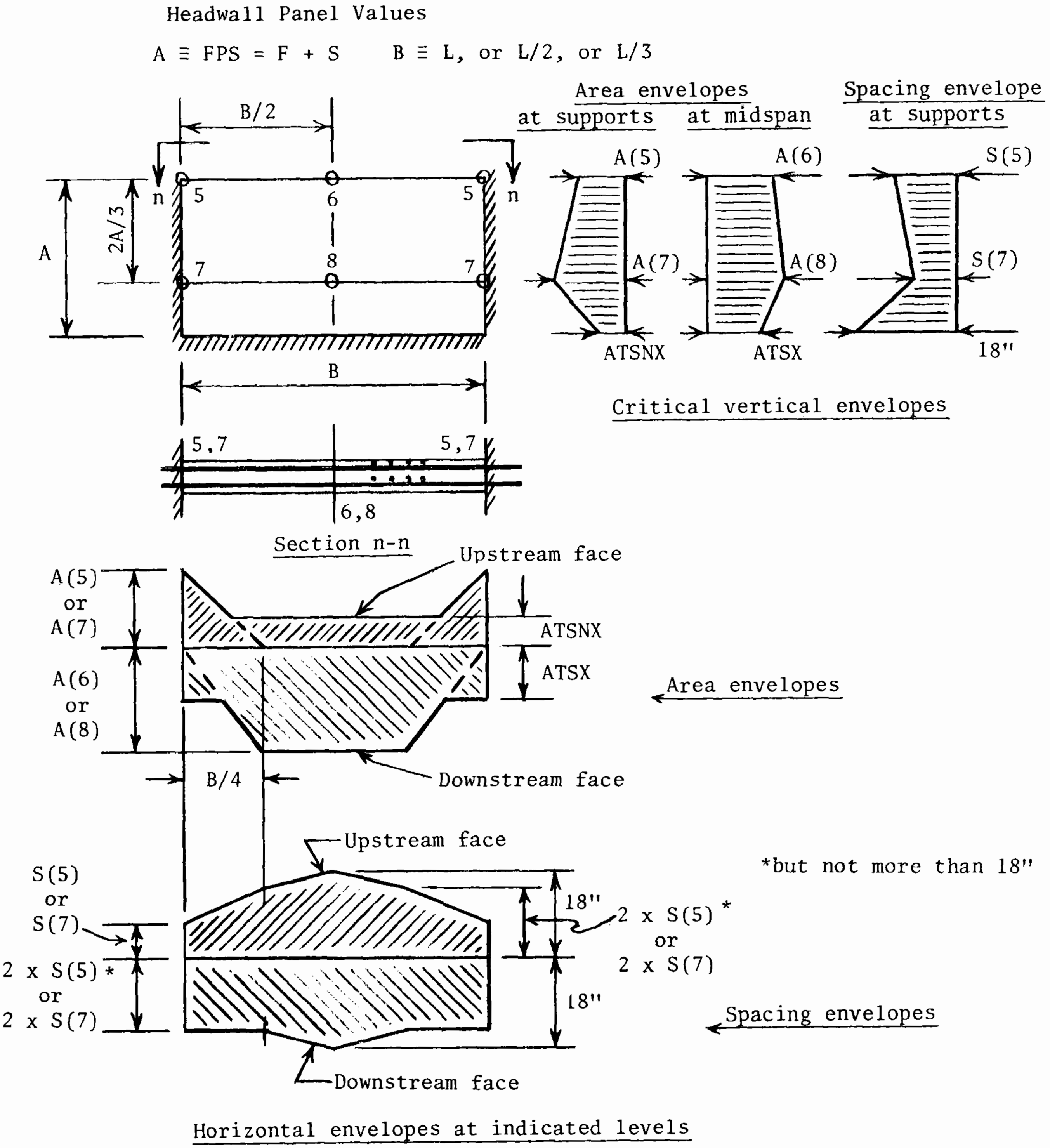


Figure 34. Horizontal steel in headwall panel.

The horizontal steel required in a headwall panel is defined in Figure 34. Steel areas required for moment, taking direct force as zero, are determined for three points in each of two horizontal strips in the panel. The strips are located at the top of the panel and two thirds down from the top. Steel spacings are determined at the side supports of the panel at the two strip levels. Panel moments and shears are computed as explained in preliminary design. The horizontal distribution of each strip steel, based on the three computed points, is given in Figure 34 by the horizontal envelopes shown. A suggested vertical distribution of horizontal steel in the panel is indicated by the vertical envelopes for steel at the supports and at midspan.

As noted in preliminary design, headwall panels are treated as two-way slabs free at the top and fixed along the other three sides. Actually, because of the continuity between headwall, sidewall, and headwall extension stub, horizontal bending restraints may be more or less than fixity. Thus adjustments to the above values may be desirable. This subject is discussed further under the section "Headwall-sidewall steel adjustments."

Sidewall Steel

The vertical steel required in the back face of the sidewall is defined in Figure 35. Steel areas for moment and direct force are determined for four points in each of three vertical strips in the sidewall. Steel spacing for bond is determined at the bottom of each strip. Moments and shear in each strip are computed as explained in preliminary design for the strip at LV from the face of the headwall, see Figures 11 and 12.

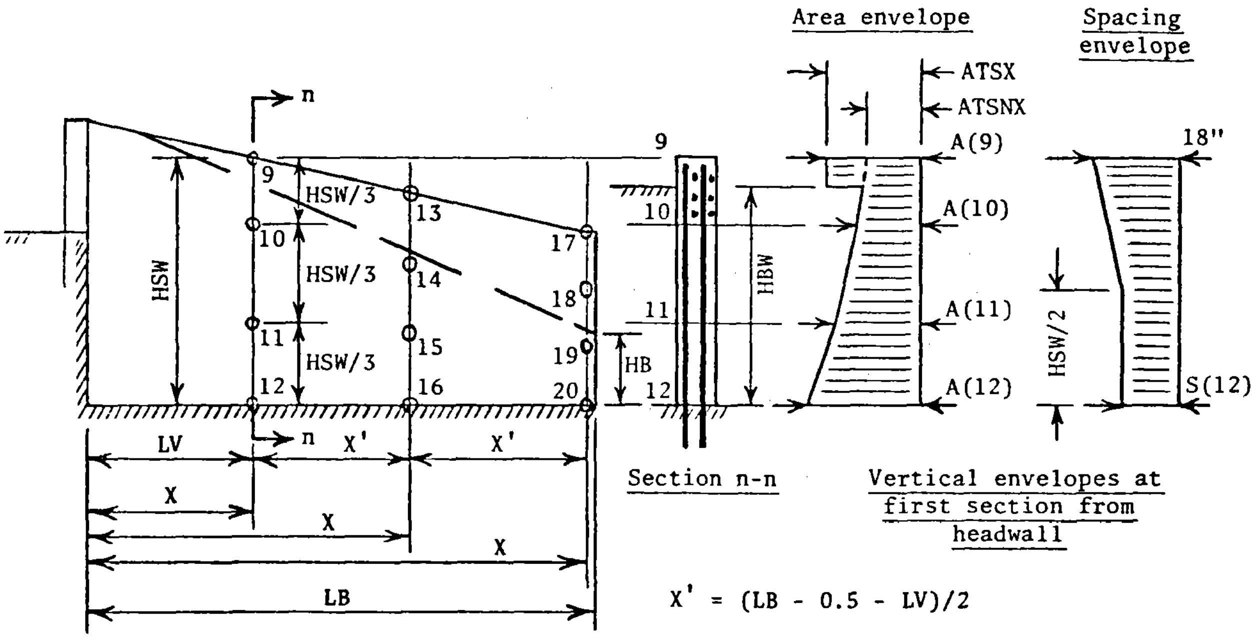


Figure 35. Vertical steel in sidewall.

Typical vertical distributions of the steel in any strip are indicated in Figure 35 by the vertical envelopes for the *section at LV from the* headwall. Longitudinal requirements for vertical steel may be assumed to vary linearly between the strips and then to remain constant upstream of the section at LV. Alternately, but less conservatively, the steel requirement upstream of LV may be assumed to vary parabolically to zero at the headwall. The latter assumption should not be used when FPS is small relative to FPSPH. Appropriate T and S requirements should be observed.

The horizontal steel required in the back face of the sidewalls is defined in Figure 36. Steel areas required for moment, taking direct force as zero, are determined for three points in each of three horizontal strips in the sidewall. The strips run between the vertical support and the assumed 45^{o} cut. Steel spacing is determined at the vertical support for each strip. Loads, moments, and shears on the strip are computed as explained in preliminary design, see Figures 13 and 14. The top strip is LHS above the top of the apron. The middle strip is LHM above the apron. Maximum required steel area occurs in this strip. The lowest strip is midway between LHM and the top of the apron. Typical longitudinal distributions of the steel at any strip are indicated in Figure 36 by the envelopes for the horizontal strip at LHS. Below LHS, the required vertical distribution of horizontal steel may be assumed satisfied by linear variations between the strips and then linear reduction to T and S requirements at the top of the apron. Above LHS, the horizontal steel requirement may be assumed constant at LHS values except for possibly more severe T and S requirements toward the top of the sidewall. Further, the horizontal steel in the back face of the sidewalls, above weir crest

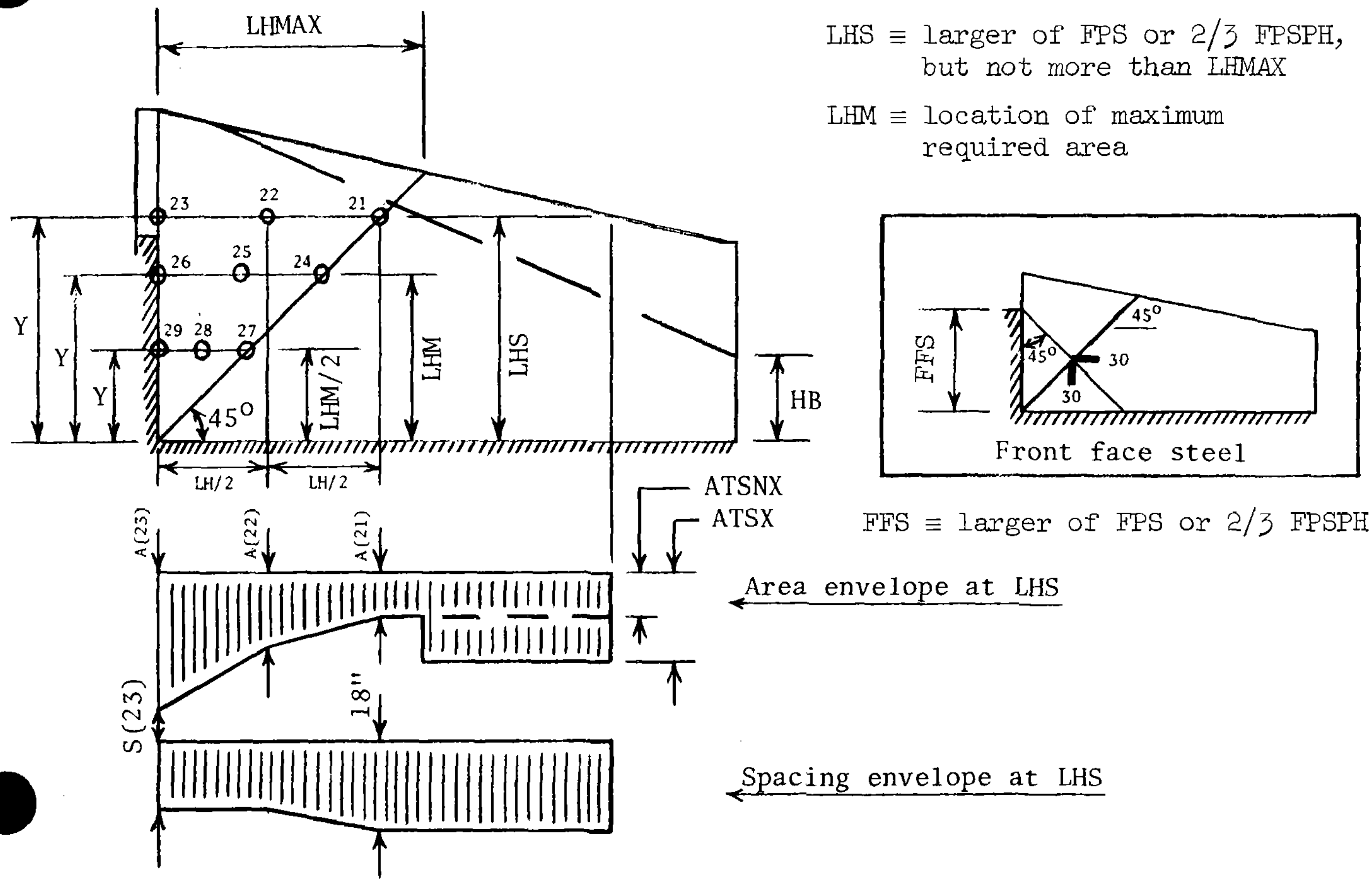


Figure 36. Horizontal steel in sidewall.

elevation, must be capable of resisting the bending induced by the headwall extension stubs. This bending, together with a consideration of the possible effects of continuity between headwall, sidewall, and headwall extension stubs, is discussed further under the section, "Headwall-sidewall steel adjustments."

Preliminary design refers to the existence of positive moments in the sidewall in the region of the assumed 45^{o} cut. The front face tension is oriented approximately normal to the ficticious cut. The sidewall must be capable of resisting these stresses or actual front face cracks may occur.

Figure 36 contains an inset sketch. This sketch shows location 30 at the intersection of the assumed cut and a 45^{o} line sloping downward from the height, FFS. This height is arbitrarily taken as the larger of FPS or 2/3 of FPSPH. Horizontal and vertical positive steel areas required at this location are computed as follows. Assume a fixed ended strip with span FFS$\sqrt{2}$ and uniform loading, QN. Take QN as the net pressure on the strip at location 30. The maximum positive moment, in ft lbs per ft, is

$$MP = QN(FFS\sqrt{2})^2/24 = QN(FFS)^2/12$$

The required area normal to the assumed cut, in sq. in. per ft, is

$$AP = \frac{M}{f_s jd} \simeq \frac{12 \times MP}{20000 \times 0.87 \times D} = 0.00069\ MP/(T - 3.0)$$

where T is the sidewall thickness at location 30. With the assumption that the requirement for positive steel can be met by a combination of horizontal and vertical steel, required steel is

$$A(30) = \sqrt{2}(AP/2) \simeq 0.00049\ MP/(T - 3.0)$$

both horizontally and vertically. This steel should be provided all along the ficticious cut except that areas may decrease to T and S requirements near the headwall-apron corner.

Apron Steel

Apron slab longitudinal and transverse steel requirements are considered separately. As described in preliminary design, one-way moments and shears are multiplied by the coefficient CSS or CLS as appropriate.

Longitudinal steel. The longitudinal steel requirement in an apron panel is defined in Figure 37. Steel areas required in the middle strip for moment, taking direct force as zero, are determined at the faces of both supports and at the interior quarter points of the longitudinal span. Steel spacing requirements are determined at both supports. Moments and shears are computed as explained in preliminary design. Again, the longitudinal span is treated as fixed at the headwall and both fixed and simply supported at the toewall. The more severe requirement is used at each location. Longitudinal distribution of top and bottom middle strip steel is given in Figure 37 by area envelopes at midspan. A suggested transverse distribution of longitudinal steel in the panel, except for A(32) and A(39), is indicated by the transverse envelope provided in the figure. Due to the uncertainties surrounding apron bending induced by the cutoff wall and the toewall, it is suggested that transverse distributions of longitudinal steel be constant along the locations indicated by A(32) and A(39). Minimum values of A(34) and A(37) are set at one-half A(32) and A(39) respectively, to help assure adequate transfer of these bending effects into the apron.

Apron Panel Values

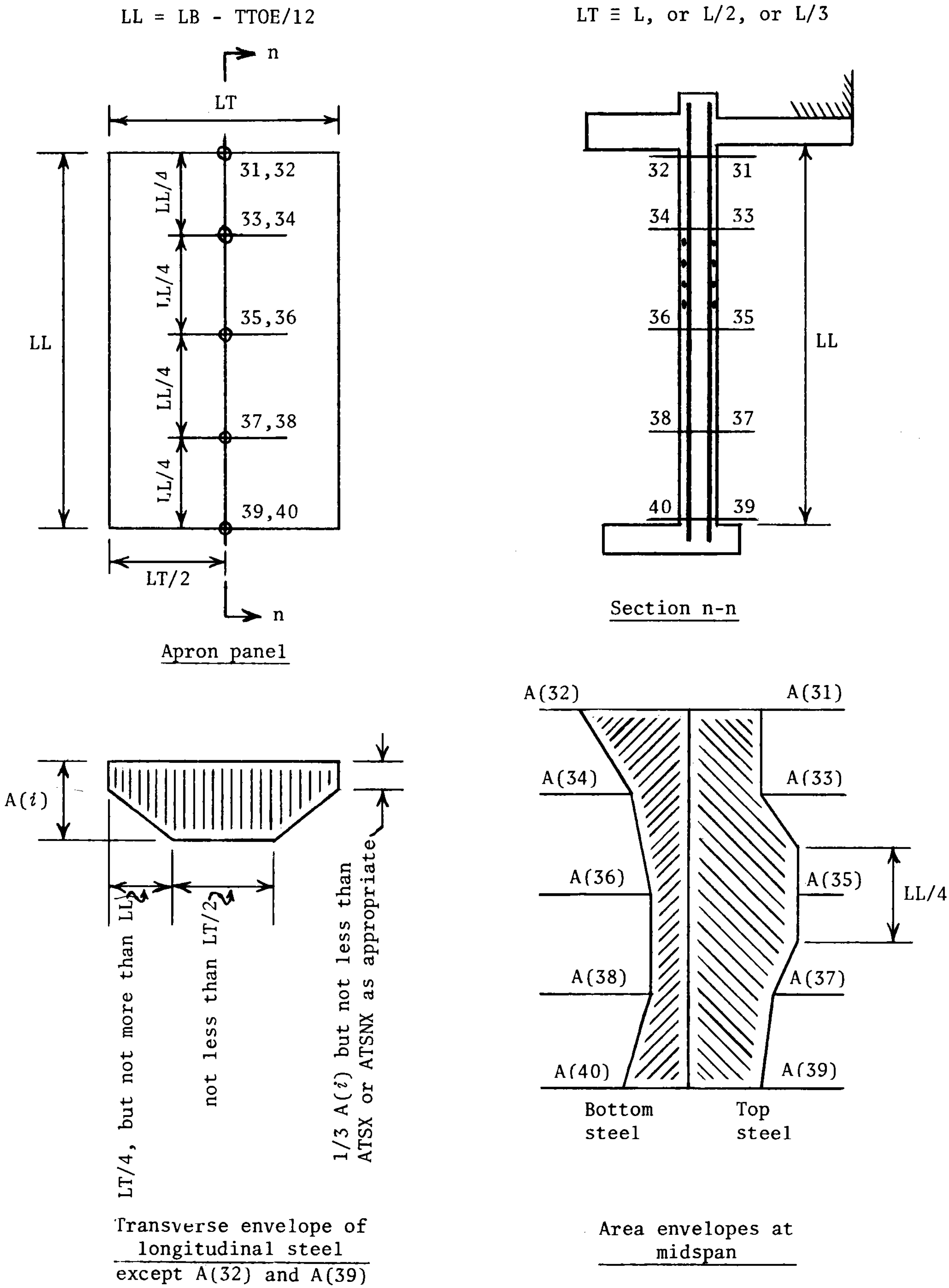


Figure 37. Apron longitudinal steel.

Longitudinal span reactions at the toewall are preserved for subsequent transverse sill design. Likewise, longitudinal span quarter point loadings are saved for the transverse apron steel design which follows.

Transverse steel. The transverse steel required in the apron slab is defined in Figure 39. Steel requirements are determined for three transverse strips. The strips are located at the longitudinal span interior quarter points. Steel areas required for moment and direct force are determined at the face of longitudinal sills, if any. Steel spacing requirements are determined at all supports.

Moments and shears are computed as explained in preliminary design. Steel area requirements at the sidewalls are determined without applying the reduction coefficient CSS or CLS. The coefficient is applied to all other calculations for transverse steel.

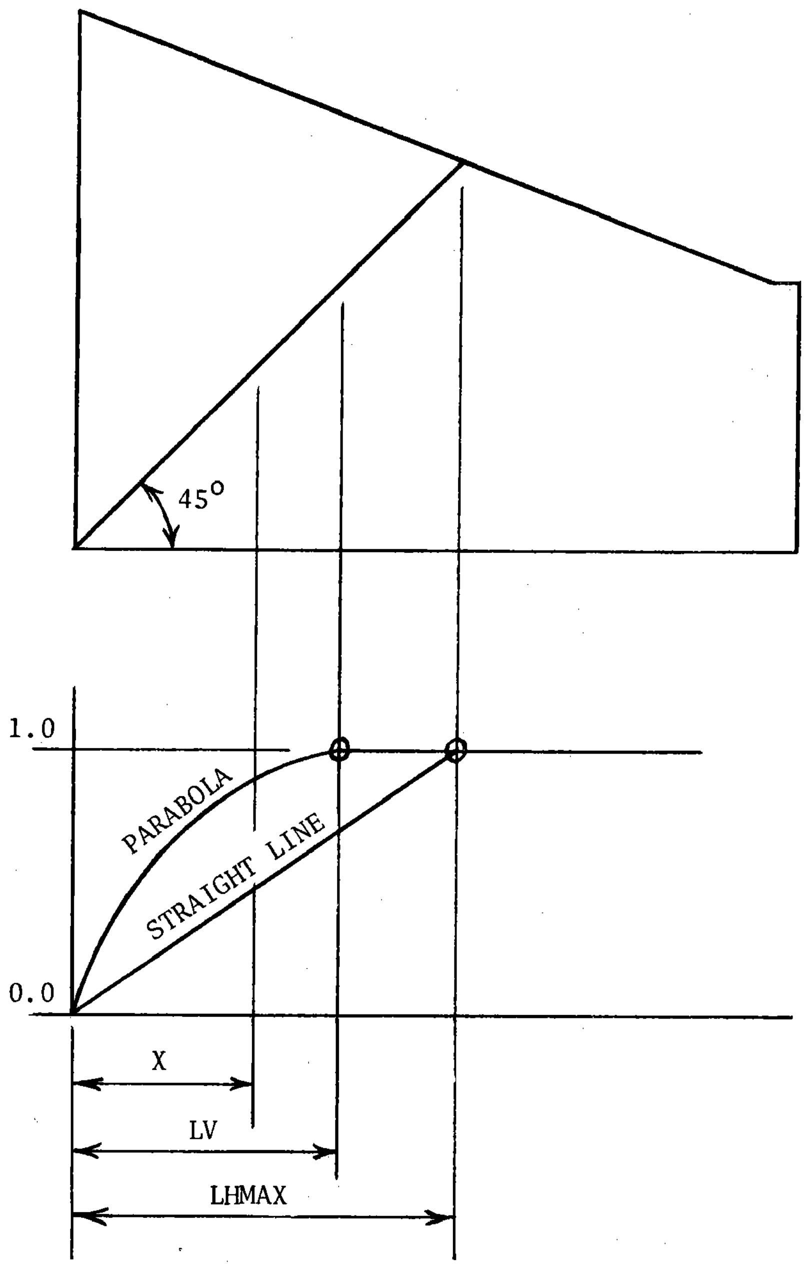
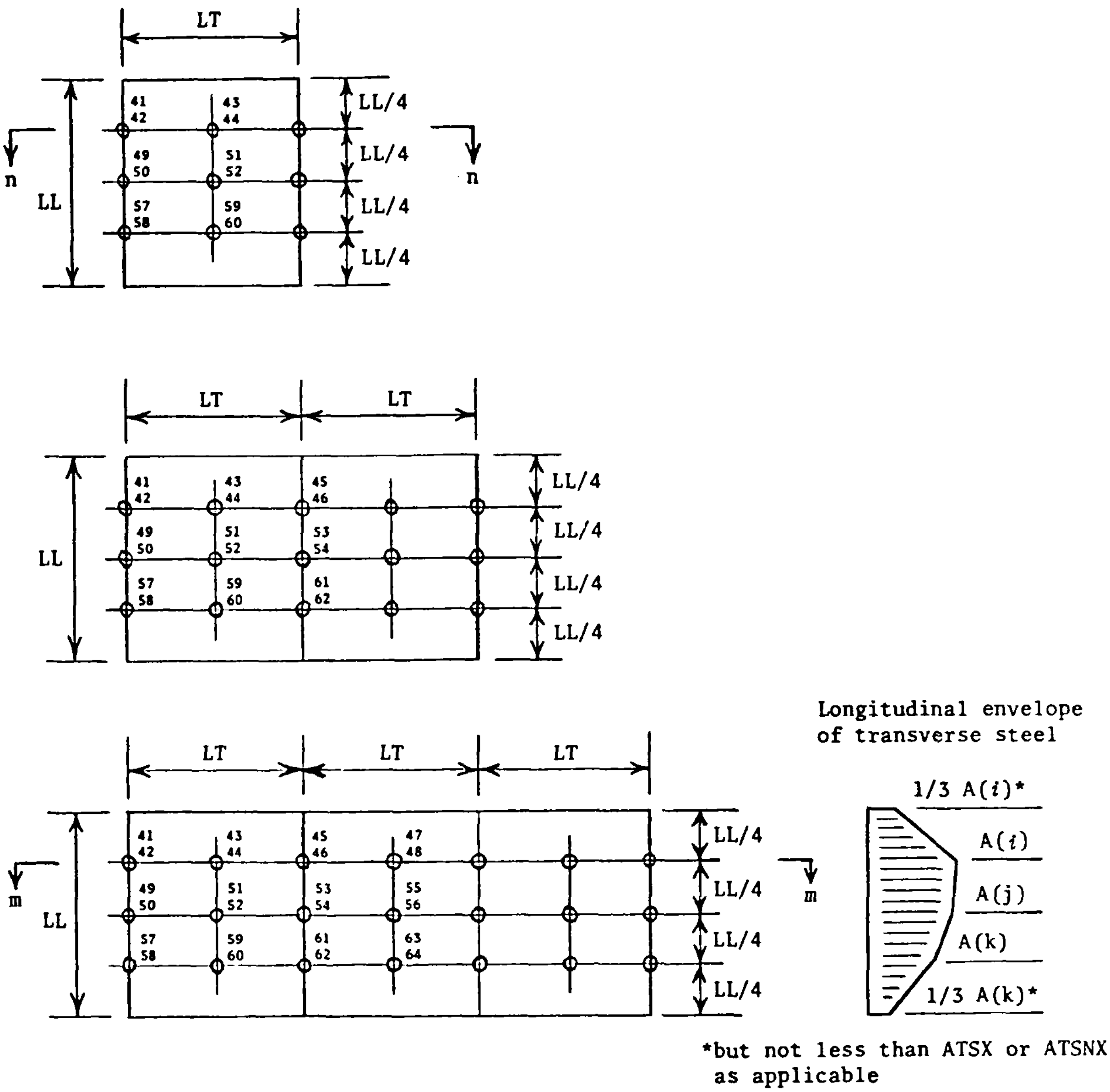


Figure 38. Effect of statical sidewall moments.

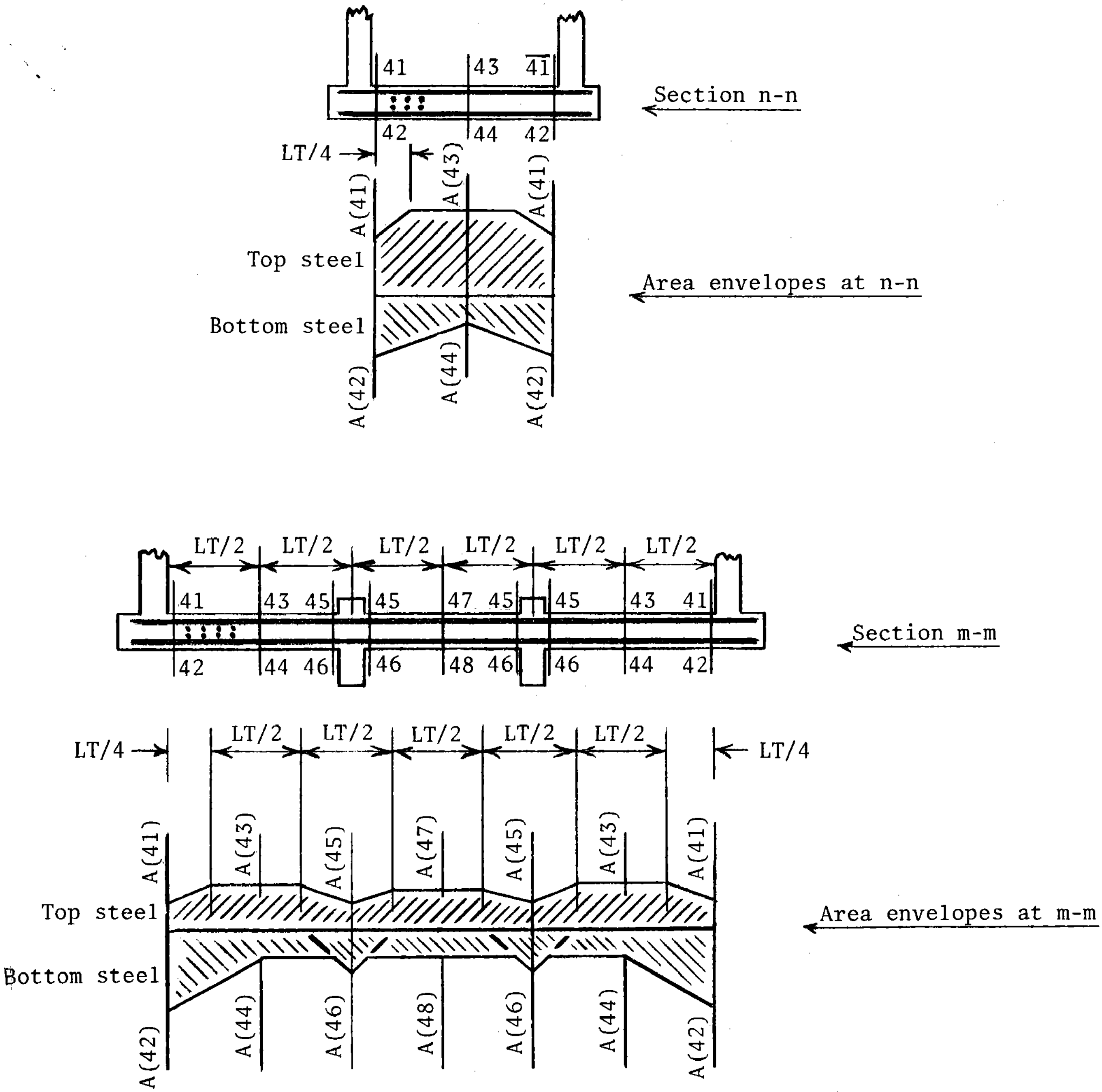
Apron Panel Values

LL = LB - TTOE/12 LT = L, or L/2, or L/3



a. Plan layout.

Figure 39. Apron transverse steel.



b. Transverse cross sections.

Figure 39. Apron transverse steel.

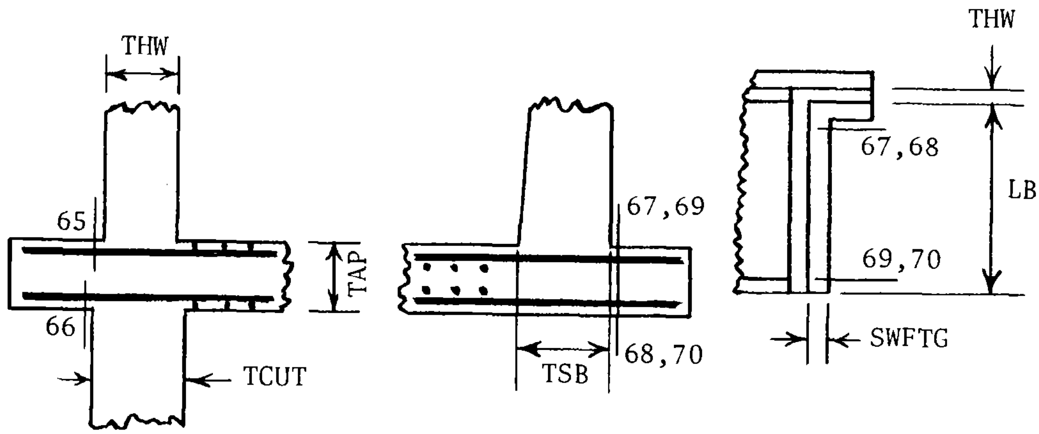
When the strip under investigation is located within the distance, LHMAX, from the face of the headwall, the effect of the statical sidewall moment, MS, on interior transverse steel requirements is quite indeterminate. Therefore, to bracket probable maximum and minimum effects of these sidewall moments, two approximate analyses are performed on any strip located at X < LHMAX. The first analysis minimizes the effect of MS. The value of MS used in the analysis is the computed value of MS at the distance LHMAX times the linear reduction ratio, X/LHMAX, see Figure 38. The second analysis maximizes the effect of MS. If LV < X < LHMAX, the value of MS used in the analysis is the full computed value of MS at the distance X. If X < LV, the value of MS used in the analysis is the computed value of MS at the distance LV times the parabolic reduction ratio (2 - X/LV)(X/LV). The more severe requirement is used at each location. For strips located at X > LHMAX, the value of MS used is the full computed value at X.

Figure 39 illustrates typical transverse distributions of top and bottom steel. The area envelopes shown are those for the first quarter point strips from the headwall for cases of LT = L and LT = L/3. A suggested longitudinal distribution of transverse steel is indicated by the typical longitudinal envelope provided in the figure.

Transverse strip reactions at longitudinal sills are preserved to develop loadings for subsequent longitudinal sill design.

Headwall and Sidewall Footing Steel

Top and bottom steel requirements for headwall footings are determined by treating the headwall footing as a pure cantilever of unit width. Downward loading consists of the weight of the footing itself and the weight of the overburden. Upward loading consists of uplift and contact bearing pressures. Figure 40 indicates this steel.



(a)
Headwall footing

(b)
Sidewall footing

Figure 40. Headwall and sidewall footing steel.

If sidewall footings exist, top and bottom steel requirements are determined at two locations. One location is immediately adjacent to the downstream headwall extension stub footing. The other location is TTOE/12 ft from the downstream end of the sidewall. Both sections are treated as unit width cantilevers with associated downward and upward loadings. Figure 40 shows these locations. Steel requirements may be assumed to vary linearly between the locations.

Buttress Design and Steel

If a buttress is required, its minimum proportions are established in preliminary design. A buttress may be required either to insure adequate longitudinal sill stiffness or to allow a reduction in required headwall thickness. The height of the buttress is F and the depth of the buttress is BUTT; both are measured relative to the top of the longitudinal sill. The thickness is TBLS; its initial value is set at 12 inches. TBLS and/or BUTT will be incremented in detail design if necessary.

Loadings. Loads are brought to the buttress as horizontal shears from adjacent headwall panels. Figure 41 shows the headwall loading converted to a combination of triangular and uniform pressures as discussed in preliminary design. These triangular and uniform pressures produce horizontal shears at the buttress. The assumed magnitudes and vertical distributions of these shears are shown in Figure 41. The shears combine to cause a resultant force on the buttress. The resultant in turn creates shear, VB, and moment, MB, in the buttress at the elevation of the top of the longitudinal sill.

Flexural analyses. Although the buttress is primarily a cantilever flexural member, a column type reduction factor, R, is applied to the allowable bending compressive stress when the length of the compression face, BL, is sufficiently long. The reduction factor is

$$R = 1.32 - 0.006 \times BL/(0.30 \times TBLS/12) \leq 1.0$$

from ACI 318-63, eq. (9-2) where

$$BL = (F^2+BUTT^2)^{1/2} \leq \sqrt{2}\ F$$

The allowable compressive stress parallel to the sloping face of the buttress is therefore, in psi

$$FPALL = 0.4f_c' \times R = 1600. \times R$$

Steel and concrete stresses are evaluated on a horizontal cross section. Hence the allowable compressive normal stress on a horizontal section is

$$FNALL = FPALL \times \cos^2\theta = 1600. \times R \times \cos^2\theta$$

where

$$\cos\theta = F/BL$$

Note that for these stress checks, it is assumed the effective buttress depth can not exceed the depth lying within a one-to-one slope from the weir crest.

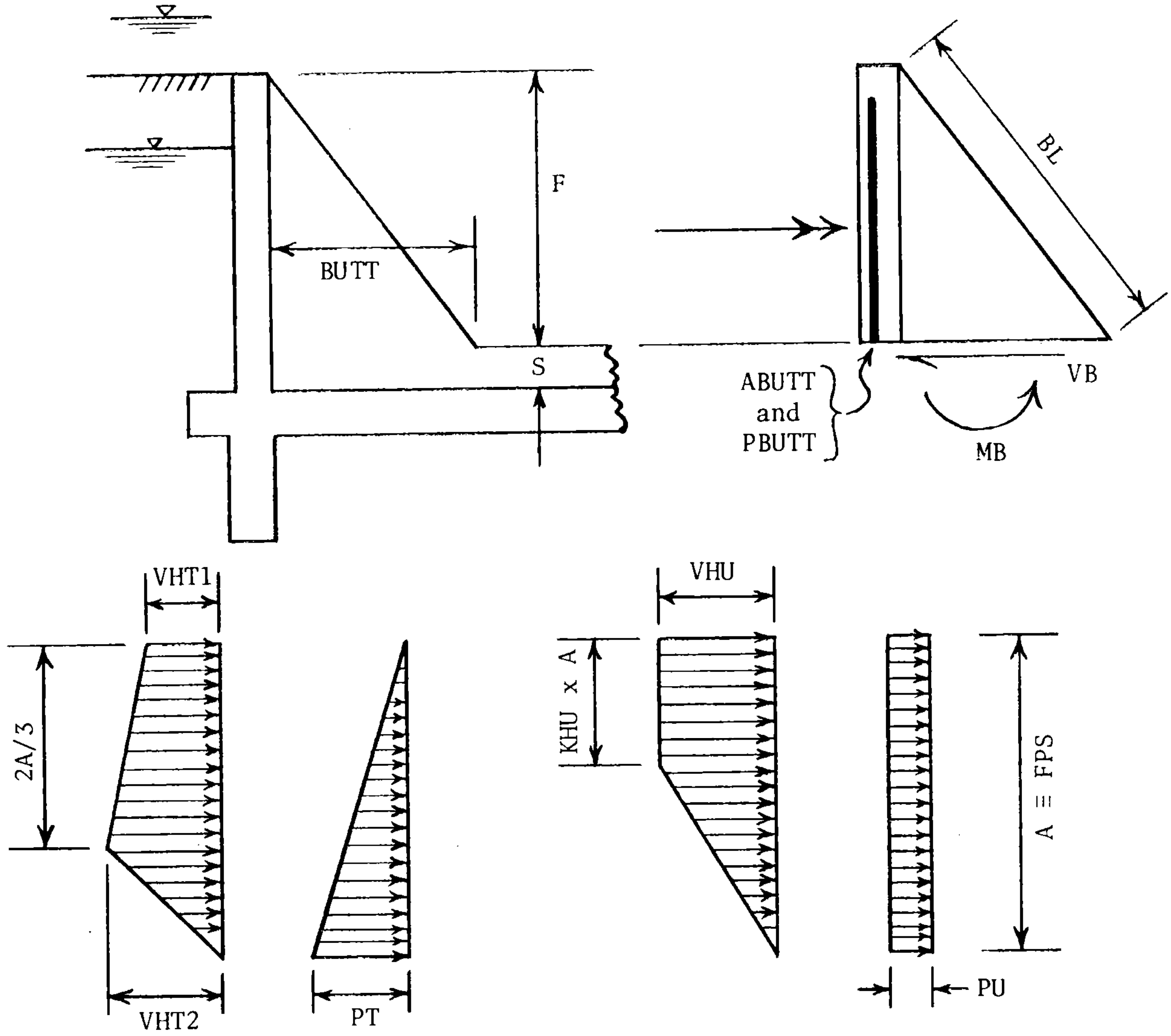


Figure 41. Buttress loadings.

The minimum buttress depth, in inches, is determined from

$$DBMIN = ((24 \times MB)/(FNALL \times k \times j \times TBLS))^{1/2}$$

where

MB ≡ buttress moment, in ft lbs
FNALL ≡ allowable normal stress determined above, in psi
TBLS ≡ buttress thickness, in inches

and

$$k = \frac{FNALL}{FNALL + 20000/8}$$

$$j = 1 - k/3$$

The actual buttress depth, in inches, is

$$DACT = 12. \times BUTT + THW - 4.0$$

If DACT < DBMIN, then BUTT is increased accordingly but not more than BUTT = F. If the increase in BUTT is insufficient, then TBLS is incremented to make up the deficiency.

The required buttress steel area, in sq. inches, is determined from

ABUTT = (12 x MB)/(20000 x j x DACT)

where k, and hence j, is found from the relation shown and explained in TR-42, page 31.

The required buttress steel perimeter, in inches, is determined from

PBUTT = VB/(304. x j x DACT)

which assumes the steel size will not exceed a #8 bar.

Diagonal tension analyses. Web steel is required whenever the nominal shear stress, as a measure of diagonal tension, exceeds $1.1\sqrt{f'_c}$ = 70 psi. The shear stress is computed at the elevation of the top of the longitudinal sill. No credit is taken for the horizontal component of the inclined flexural compressive stresses.

If web steel is required, the ratio of computed required web steel area to spacing is determined from

AV/S = (VB - 70. x TBLS x DACT)/(20000. x DACT)

Combinations of acceptable web steel sizes and spacings may be obtained from the above ratio.

Whether or not web steel is required may be determined from the ratio of the shear on the section to the shear the section can take without web steel. This ratio is determined as

V/VC = VB/(70. x TBLS x DACT)

When 0. ≦ V/VC ≦ 1.0	web steel is not required.
When 1.0 < V/VC ≦ (3.0/1.1 = 2.73)	every potential 45^{o} crack must be crossed at least once.
When 2.73 < V/VC ≦ (5.0/1.1 = 4.55)	every potential 45^{o} crack must be crossed at least twice.

When web steel is required at the elevation of the top of the longitudinal sill, a question remains as to web steel requirements above this elevation. If it is assumed the resultant loading on the buttress is uniformly distributed over the height of the buttress, then analysis will show that the ratio AV/S remains essentially constant throughout the buttress height.

Longitudinal Sill Design and Steel

The longitudinal sill is treated as fixed at the toe of the buttress and both fixed and simply supported at the transverse sill. The sill cross section is rectangular with thickness, TBLS, and depth equal to the sum of S + TAP/12 + BOTT. Loading on the sill is trapezoidal, it is converted to a combination of triangular and uniform loads, see Figure 42. The sill loads, PU and PT, are obtained from the apron

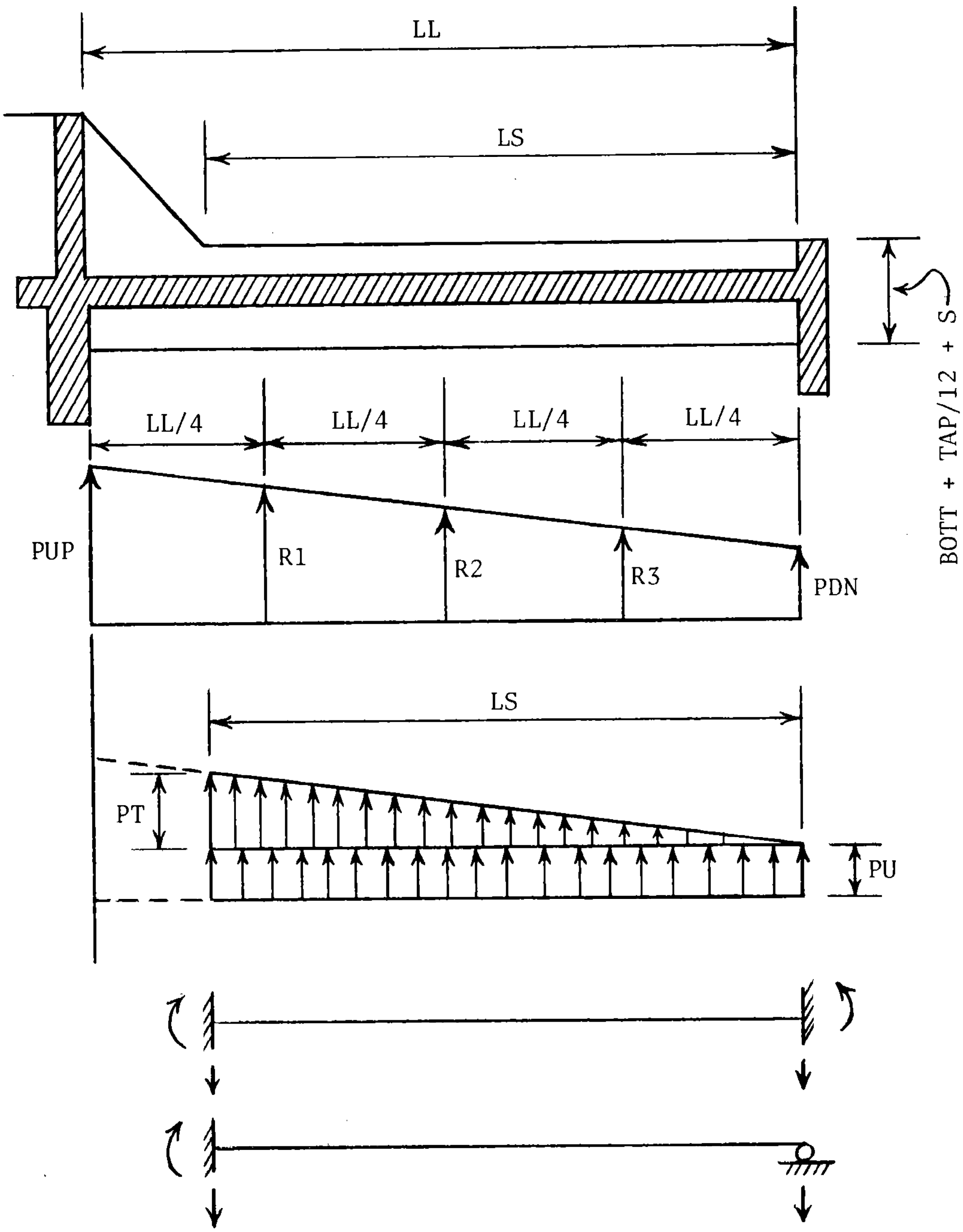


Figure 42. Longitudinal sill design.

reactions, PUP and PDN, minus the weight of the portions of the sill that project above and below the apron slab. PUP and PDN are obtained from the longitudinal sill quarter point, transverse strip reactions previously preserved. PUP and PDN are obtained by straight line connection of strip reactions, R1, R2, or R3, that indicate the greatest load on the sill. TBLS and/or BOTT will be incremented in detail design if necessary. BOTT may not exceed HTOE - TAP/12, and TBLS is arbitrarily limited to a maximum of 24 inches. If both limits are reached, then LS will be decreased, by increasing BUTT, until a satisfactory solution is obtained.

Longitudinal sill steel is defined in Figure 43. Steel areas for moment are determined at the supports and at midspan of LS. Steel perimeters for flexural bond are determined at both supports from the relation

$$P = V/(U \times 0.875 \times D)$$

where

- P ≡ required perimeter, in inches
- V ≡ shear at support, in lbs
- U ≡ allowable bond stress, taken as 246 psi for top steel and 347 psi for bottom steel, thus bond will be satisfactory for bars not exceeding #7
- D ≡ effective depth, in inches

For top steel

$$D = (S + BOTT) \times 12 + TAP - 3.0$$

and for bottom steel

$$D = (S + BOTT) \times 12 + TAP - 4.0$$

Web steel parameters are determined at both supports. Refer to discussion of diagonal tension analyses under buttress design. In the case of longitudinal sills, the ratios, V/VC and AV/S, may be used to construct diagrams from which web steel requirements throughout the span, LS, may be determined. Figure 44 illustrates how this may be done assuming the portion of a shear diagram under consideration is triangular and the value of V/VC at the support is 1.76. From the figure, the theoretical length requiring web steel is

$$LWEB = LZERO \times 0.76/1.76$$

Figure 43 shows suggested web steel envelopes for longitudinal sills. These envelopes account for possible trapezoidal rather than uniform loads on the sill.

Longitudinal sill reactions at the transverse sill, for both fixed and simple supports, are saved for the transverse sill design.

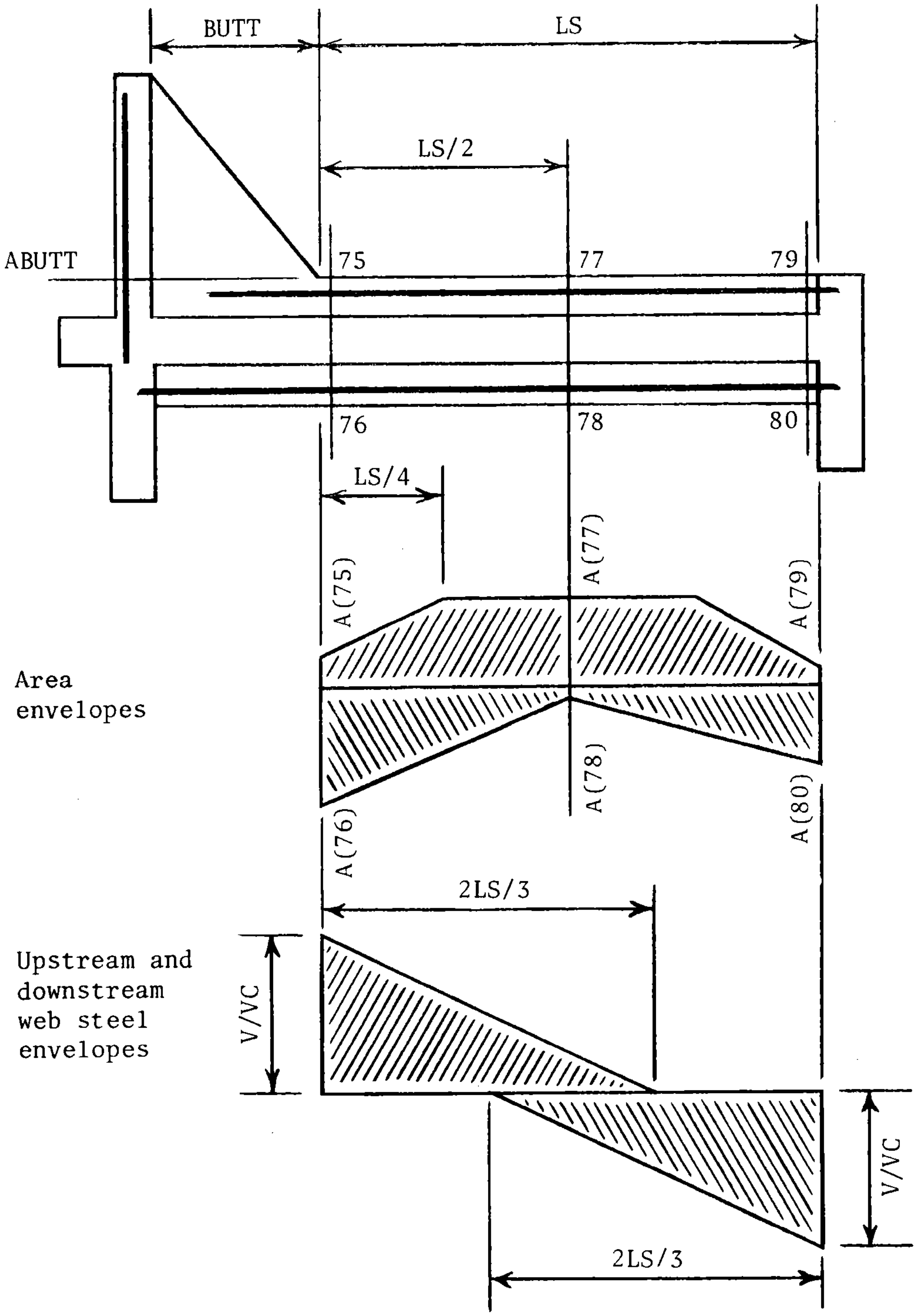


Figure 43. Longitudinal sill steel.

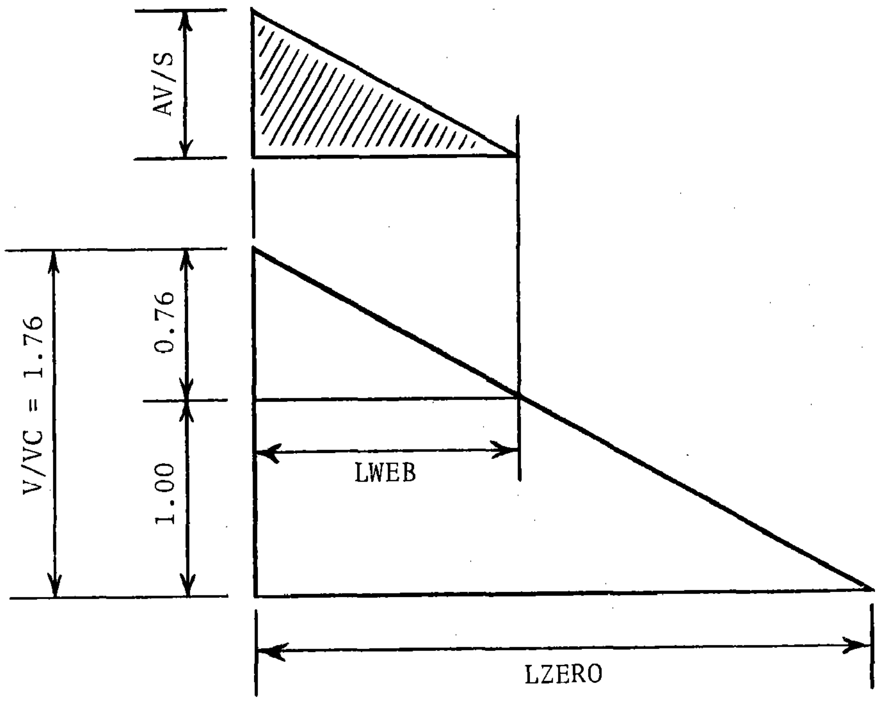


Figure 44. Region of required web steel.

Transverse Sill Design and Steel

The transverse sill provides support for the apron in longitudinal bending and also acts as support for the longitudinal sill(s), if any. Thus the loading on the sill is a combination of uniform plus either no, one, or two concentrated loads. The apron longitudinal bending and longitudinal sill loads are taken at their fixed reaction values and also at their simple reaction values. The uniform loading is the algebraic sum of apron longitudinal bending reaction, average contact bearing pressure on the sill, average uplift pressure, sill dead weight, and tailwater weight on the sill. The sill cross section is rectangular with thickness, TTOE, and depth equal to the sum of S + HTOE. The transverse sill is treated as a fixed ended beam and also as a restrained beam with end moments at one-half their fixed end values. HTOE and/or TTOE will be incremented in detail design if necessary. Arbitrarily, TTOE is limited to a maximum of 24 inches and HTOE will not be incremented more than 2 feet.

Transverse sill steel is defined in Figure 45. Steel areas for moment are determined at the supports and at midspan. Steel perimeters for flexural bond are determined at the supports. For these computations, for top steel

D = (S + HTOE) x 12. - 3.0

and for bottom steel

D = (S + HTOE) x 12. - 4.0

Flexural shear web steel parameters are determined at the supports. Figure 45 gives three suggested web steel envelopes for transverse sills. The envelope to use depends on the number of longitudinal sills. Each envelope is conservative for its intended use.

It should be recognized that the transverse sill is subjected to unknown amounts of torsion. This torque is induced by combinations of: vertical

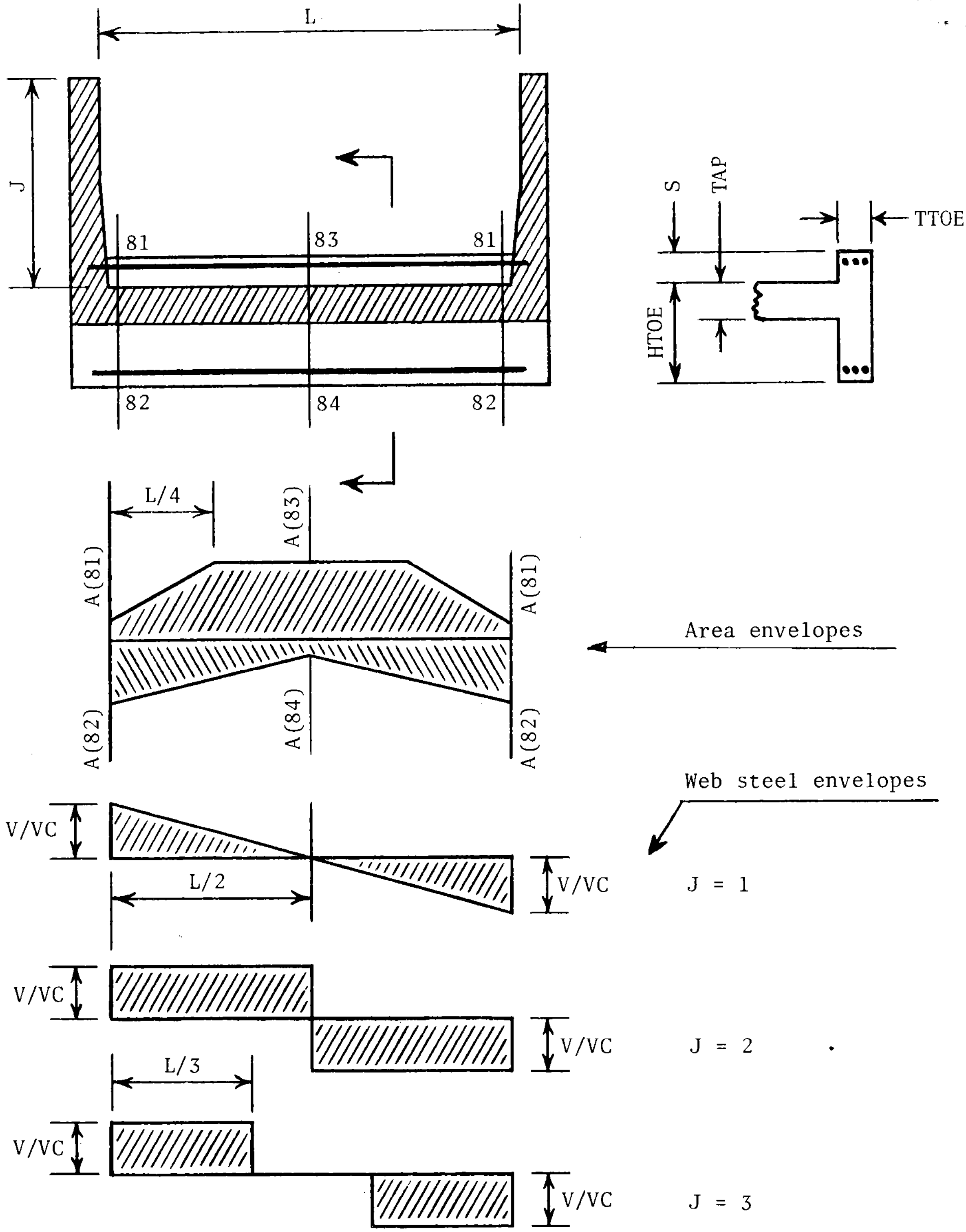


Figure 45. Transverse sill steel.

reactions and end moments from apron longitudinal bending, vertical reactions and end moments from the longitudinal sill(s) if any, and/or eccentricity of the forces and moments induced by possible horizontal loads acting against the toewall. Hence it is advisable that at least nominal closed loops be provided throughout the transverse sill even when no need of flexural shear web steel is indicated.

Toewall, Cutoff Wall, Headwall Extension Stub Steels

As presented in preliminary design, the toewall must be able to resist the cantilever bending that might be produced by passive resistance of the downstream channel material. The required steel area and maximum steel spacing at location 72 are computed from the shear and moment described earlier, see Figure 30. The steel area required in the apron slab at location 39 due to MAP and NAP, caused by toewall bending, is computed and compared with that required by apron panel bending previously described. Figure 46 defines this steel. In the event of scour of material away from the downstream face of the toewall, significant toewall bending of opposite sign to that discussed above may occur. This bending would be resisted by the vertical steel in the upstream face of the toewall. The computed amount of steel required depends on the conditions assumed by the designer. Arbitrarily, an area of A(72)/2, but not less than A(72) x KOF/KPF, is suggested as a minimum amount.

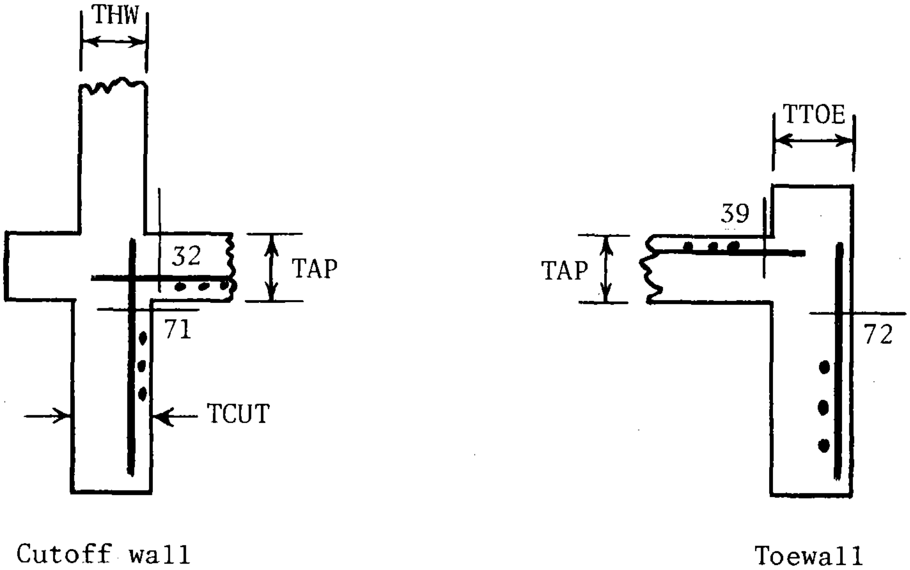


Figure 46. Cutoff wall and toewall steels.

The cutoff wall must be able to resist the cantilever bending produced by the assumed loading. See discussion in preliminary design and Figure 31. Steel area and spacing requirements are determined at location 71 for the conditions indicated. The steel area required in the apron slab at location 32 due to MA, caused by the interaction of moments at the headwall, apron, and cutoff wall joint, is computed and compared with that required by apron panel bending previously described. Figure 46 defines this steel. Significant cutoff wall bending of opposite sign to that discussed above may sometimes occur. This bending would be resisted by the vertical steel in the upstream face of the cutoff wall. The amount of steel required is very uncertain. As an upper limit, the amount would not need to exceed the area required to resist a moment, in ft lbs per ft, of

$$M = KOF \times GMH \times (FPS + TAP/12) \times (HCUTN)^2/2.$$

The headwall extension stub is subjected to horizontal cantilever bending. The analysis is presented in preliminary design, see Figure 29. Steel areas and spacings are determined at two elevations. The lower elevation is at the assumed critical section. The higher location is at crest elevation. Figure 47 defines this steel. Required horizontal steel may be assumed to vary linearly between these two elevations and to decrease to T and S requirements above and below them.

Vertical steel requirements should be considered in the headwall extension stub along its interface with the headwall footings. This steel serves to balance cantilever bending in the cutoff wall and to resist vertical bending in accordance with the 45° concept. By this concept, the required area of this steel at the outer edge of the stub, in the downstream face, would need to be something in excess of A(74).

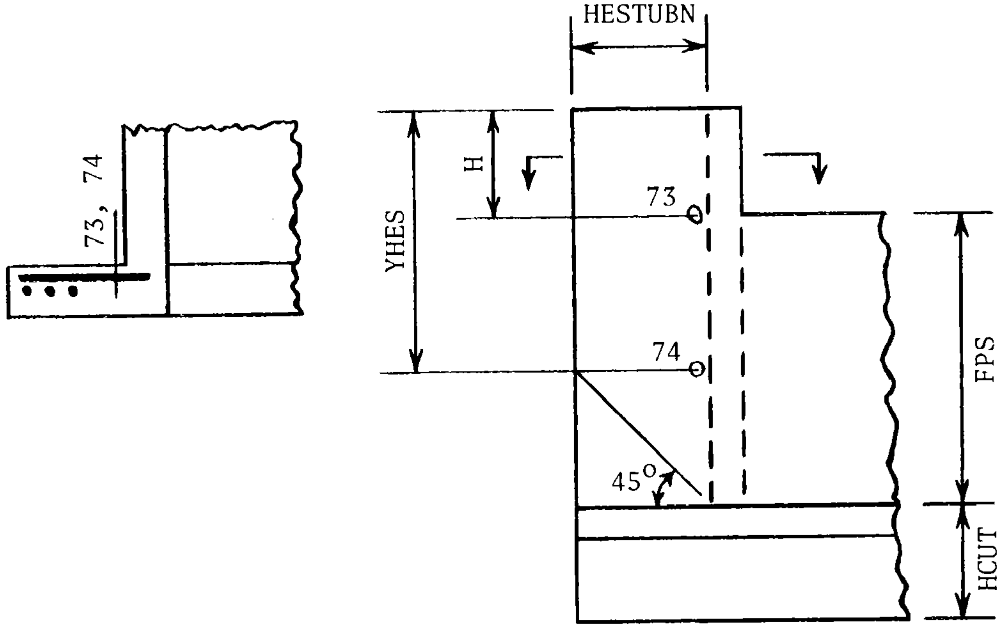


Figure 47. Headwall extension stub steel.

Headwall-Sidewall Steel Adjustments

As previously noted, horizontal bending in headwall panels is based on the assumption of fixed supports. Also, horizontal bending in the sidewalls is based on the assumption of cantilever bending with an assumed 45° cut through the sidewall. Further, horizontal bending in headwall extension stubs is based on a possible limiting condition induced by resistance to sliding. Thus, to this point, adequate consideration has not been given the question of possible moment unbalance along the junction of headwall, headwall extension stub, and sidewall.

Above the elevation of the weir crest, the question is readily handled. As a minimum, there must be sufficient horizontal steel in the back face of the sidewall to balance the horizontal bending needs of the headwall extension stub. Headwall extension stub bending assumes the development of passive pressures against the downstream face of the stub. Hence, by definition, these are maximum pressures. Thus stub bending, at and above the weir crest, is a maximum. Note that in general

$$M = f_s A_s jd = f_s j A_s d \propto A_s d \propto A_s t$$

where the symbols have their usual reinforced concrete theory meanings. Thus the resisting moment at a section is approximately proportional to the product of tensile steel area and section thickness. The amount of steel, A(73)BHE, required to balance the headwall extension stub steel, A(73), is determined by equating headwall extension stub moment to sidewall resisting moment. Thus

A(73)BHE = A(73) x THW/TSW

The really unsettled question is how to satisfactorily treat the effect of continuity between headwall, headwall extension stub, and sidewall below the elevation of the weir crest. One of the main difficulties deals with the stiffnesses of adjoining headwall, headwall extension stub, and sidewall horizontal strips. The following approximate analysis is used to determine limiting steel area values in the sidewall and headwall at and below the weir crest elevation. The headwall extension stub strip is assumed to be without stiffness. Headwall and sidewall strip stiffnesses are assumed to be proportional to wall thicknesses cubed and independent of length. A one cycle moment balancing procedure is used. Thus for adjoining horizontal strips, let the balancing moment required for equilibrium be given by

$$\mathrm{MBAL} = \mathrm{MHW} + \mathrm{MHE} - \mathrm{MSW}$$

where MHW, MHE, and MSW are the respective strip moments at the elevation under investigation. Then, algebraically, the adjusted moment in the headwall at this elevation is

$$\mathrm{MHWA} = \mathrm{MHW} - \frac{(\mathrm{THW})^3}{(\mathrm{THW})^3 + (\mathrm{TSW})^3} \times \mathrm{MBAL}$$

and the adjusted moment in the sidewall is

$$\mathrm{MSWA} = \mathrm{MSW} + \frac{(\mathrm{TSW})^3}{(\mathrm{THW})^3 + (\mathrm{TSW})^3} \times \mathrm{MBAL}$$

These moment relations are converted to corresponding steel areas through the assumption that resisting moment at a section is closely proportional to the product of steel area and section depth. Therefore, the adjusted required steel area in the headwall is

$$\mathrm{AHWA} = \mathrm{AHW} - \frac{(\mathrm{THW})^3}{(\mathrm{THW})^3 + (\mathrm{TSW})^3} \times \mathrm{MBAL/THW}$$

and the adjusted required steel area in the sidewall is

$$\mathrm{ASWA} = \mathrm{ASW} + \frac{(\mathrm{TSW})^3}{(\mathrm{THW})^3 + (\mathrm{TSW})^3} \times \mathrm{MBAL/TSW}$$

Where now, in terms of areas and thicknesses

$$\mathrm{MBAL} = (\mathrm{AHW} + \mathrm{AHE}) \times \mathrm{THW} - \mathrm{ASW} \times \mathrm{TSW}$$

in which AHW, AHE, and ASW are the respective strip steel areas at the elevation under investigation. They are obtained by interpolation of the steel areas previously determined for the headwall, headwall extension stub, and sidewall.

Limiting steel area values, i.e., adjusted required steel areas, are computed at the supports of the three sidewall horizontal strips previously described. However, if the height(s) of any sidewall strip(s) above the apron exceeds FPS, the corresponding height(s) and support steel area(s) are adjusted to FPS equivalent values before computation of limiting steel areas begin.

Limiting steel area values are computed at the exterior supports of the two headwall horizontal strips previously described. When the limiting steel area value for the exterior support of a headwall horizontal strip differs from the steel area value computed in accordance with fixed supports, the steel area requirement at interior locations in the horizontal strip are also changed. The top headwall strip is used for illustration, see Figure 34. Let

$\Delta A = AHWA - AHW$

and note

$AHW = A(5)$.

If the headwall is without a buttress, then as can be seen from appropriate moment diagrams

$A(5)LSA = A(5) + \Delta A$

$A(7)LSA = A(7) - \Delta A$.

If the headwall has one buttress, then

$A(5)LSA = A(5) + \Delta A$	at sidewall supports
$A(5)LSA = A(5) - \Delta A/2$	at the interior support
$A(7)LSA = A(7) - \Delta A/4$	at midspans

If the headwall has two buttresses, then

$A(5)LSA = A(5) + \Delta A$	at sidewall supports
$A(5)LSA = A(5) - \Delta A/5$	at interior supports
$A(7)LSA = A(7) - 2\Delta A/5$	at midspans of exterior spans
$A(7)LSA = A(7) + \Delta A/5$	at midspan of interior span

In assessing the merit of the above analyses for determining limiting steel area values, several criticisms should be considered.

(1) Resisting moment is assumed proportional to tensile steel area times section depth.

(2) Support face moments are used as though they are moments at the joint. That is, the effects of various face shears and direct forces are neglected in summing moments.

(3) The steel areas used (AHW, AHE, and ASW) are maximum values for the particular functions. They are used as though they are simultaneous values, i.e., occur for the same loading condition.

(4) The analyses from which the steel areas were previously computed, are approximate and conservative.

(5) Changes in horizontal bending produce secondary changes in vertical bending, etc., etc.

In view of these imperfections in theory, caution in design dictates that the larger steel area requirement, e.g., A(5)LSA or A(5), at a particular location be met.

Wingwall Steel

As previously stated for preliminary design, design criteria and procedures for straight drop spillway wingwalls parallels that given in TR-54 for the design of SAF stilling basin wingwalls. This is also true of detail design. Refer to Figures 47 and 48, and pages 58 through 62, of TR-54 for steel point locations and associated discussion of steel requirements.

Concrete Volumes

Concrete volumes, in cubic yards, are computed for both preliminary and detail designs. The scheme used is basically the same as presented in TR-54 for SAF stilling basins. The volumes are given in two parts. The first is the volume of the spillway proper, exclusive of wingwalls. The second is the volume of the two wingwalls including several adjustments. These adjustments account for the mating of (1) sidewall and wingwall, (2) spillway and wingwall toewalls, and (3) spillway and wingwall footings.

Spillway Volumes

The spillway volume is readily obtained. Certain assumptions are made to facilitate computing this volume. It is assumed that the sidewalls end abruptly at the vertical plane containing the downstream face of the toewall, see Figure 1. It is further assumed that the spillway toewall ends abruptly at the vertical plane containing the outside face of the sidewall.

It should be noted the spillway volume does not include the volumes of either floor blocks, toewall fillet, or cutoff wall fillets. Floor block proportions are subject to variation as are fillet sizes.

Wingwall Volumes

The computation of the wingwall volume with its adjustments is somewhat complicated. Figure 2 shows a typical wingwall layout. First the wingwall volume is computed without adjustments. As with the spillway proper, certain assumptions are made to facilitate computing this volume. It is assumed that the wingwall and wingwall toewall begin at the articulation joint and extend outward a span of (J-1). It is further assumed that the spillway proper is without sidewall footings. The wingwall volume without adjustments thus consists of the volumes of (1) the wingwall, (2) the wingwall toewall, and (3) the wingwall footing with its extension back to the spillway sidewall.

The adjustments subsequently applied to the wingwall volume depend on the corner detail indicated in Figure 2 and shown to larger scale in Figure 48. The thickness of the wingwall toewall, TWT, is the larger of the thickness of the spillway toewall, TTOE, or the thickness of the wingwall, TWW. The level distance, LEVEL, which locates the articulation joint with respect to the corner of the sidewall, and the distance BACK, which serves to define the wingwall footing extension back to the face of the sidewall, are given in inches by

$$\mathrm{LEVEL} = \mathrm{TSW}\sqrt{2} - \mathrm{TWW}$$
$$\mathrm{BACK} = \mathrm{TWT} - \mathrm{TWW}$$

when $\mathrm{TSW} \geqq \mathrm{TWW}\sqrt{2}$, otherwise

$$\mathrm{LEVEL} = \mathrm{TSW}/\sqrt{2}$$
$$\mathrm{BACK} = \mathrm{TWT} - \mathrm{LEVEL}$$

See Figure 48, sketches (a) and (b).

Instead of the sidewall ending abruptly at the vertical plane containing the downstream face of the toewall, the sidewall makes a 45° turn and ends at the articulation joint as is shown in Figure 48. Thus a volume

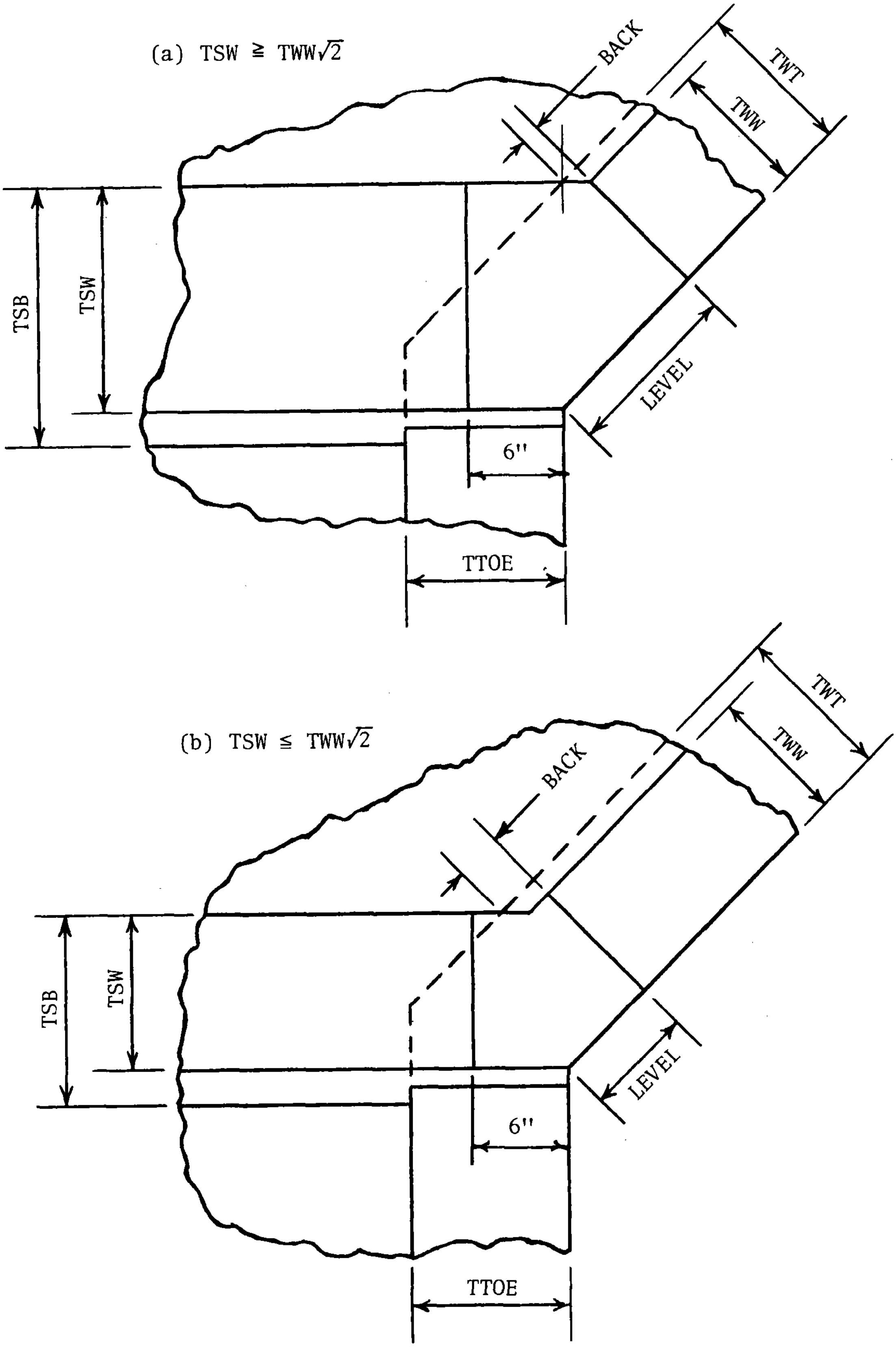


Figure 48. Corner detail, wingwall-to-sidewall.

correction or adjustment is necessary. It is applied herein to the wingwall volume.

Also, instead of the spillway toewall ending at the outside face of the sidewall, the spillway toewall mates with the wingwall toewall in a 45° turn as is shown in Figure 48. Thus another volume adjustment is required. This adjustment is also applied to the wingwall volume.

Sometimes the spillway proper will have sidewall footings. Such footings extend to the vertical plane containing the downstream face of the toewall. Thus an adjustment volume, VFTG, is necessary that will take account of any wingwall footing that is in space already occupied by the spillway sidewall footing. Refer to Figure 53, page 67, TR-54 for possible configurations and pertinent wingwall variables. Let the volume, VWING, be the wingwall volume exclusive of VFTG; then the completely adjusted wingwall volume is

QUANT = VWING - VFTG.

If for any reason VFTG can not be computed, QUANT is set to zero and a message is given. This does not mean the design is unsatisfactory. Rather, it means that some design decision is necessary concerning the layout of the wingwall footing. Values of both QUANT and VWING are reported so that the wingwall volume exclusive of correction for sidewall footings is always available.

Computer Designs

Input

Two lines of alphameric information must preceed all other data in the input for a computer job. The two lines are used to provide information such as site number, watershed, state, date of design, and other information desired by the requesting office.

A computer job may include many design runs. From one to twelve lines of input data are required for each design run. A design run is made for a particular set of design conditions and takes one of two forms. The first form gives preliminary designs of the spillway and the wingwalls. The second form gives detail designs of the spillway and the wingwalls.

The input data provided per design run consists essentially of values for the primary design parameters and, if desired, values for the secondary parameters. Table 3 shows the lines that may be provided per design run together with the specific design parameters contained on the first line and the last ten lines.

Table 3. Input values per design run

H	F	S	J	L	LB	DESIGN	DFALTS
DFALT1	DFALT2	DFALT3	DFALT4	DFALT5	DFALT6	DFALT7	-
CREEPR	FLOATR	SLIDER	BAT	SWLDRN	-	-	-
HB	ZPS	HTOE	TTOE	CFSS	CFSC	-	-
KOH	GMH	GSH	-	-	-	-	-
KOF	GMF	GSF	KPF	-	-	-	-
KOW	GMW	GSW	KPW	-	-	-	-
DW2	HEAD2	TAIL2	HEAD1	-	-	-	-
DWM2	HEADM2	TAILM2	-	-	-	-	-
DWM3	HEADM3	TAILM3	-	-	-	-	-
DWM4	HEADM4	TAILM4	-	-	-	-	-
DWM5	HEADM5	TAILM5	-	-	-	-	-

The first line is always required. It contains the primary parameters H, F, S, J, L, and LB. DESIGN is used to designate whether the user wishes preliminary designs or detail designs and also whether the user wants to specify the number of transverse apron slabs, i.e., number of longitudinal sills, or wants the computer to make the selection based on least concrete volume. If DESIGN = 0, a preliminary design is performed and the computer selects the number of transverse apron slabs. If DESIGN = 1, 2, or 3, a preliminary design is performed with 1, 2, or 3 transverse apron slabs. If DESIGN = 100, a detail design is performed and the computer selects the number of transverse apron slabs. If DESIGN = 101, 102, or 103, a detail design is performed with 1, 2, or 3 transverse apron slabs. If DFALTS = 0, all secondary parameters are assigned default values and the next eleven lines must be omitted. If DFALTS > 0, some or all secondary parameters are assigned user values and the next line of input data must be provided.

If DFALT1 = 0, the line of input data starting with CREEPR must be omitted. If DFALT1 > 0, the line of input data containing values of CREEPR through SWLDRN must be provided.

If DFALT2 = 0, the line of input data starting with HB must be omitted. If DFALT2 > 0, the line of input data containing values of HB through CFSC must be provided. Similarly for DFALT3 and the line of input data starting with KOH, also DFALT4 and KOF, DFALT5 and KOW.

DFALT6 and DFALT7 are associated with water parameters, refer to Table 2. If DFALT6 = 0, the line of input data starting with DW2 must be omitted, and loadings M = 1 and 6 will have default values. If DFALT6 > 0, the line of input data containing values of DW2 through HEAD1 must be provided, and loadings M = 1 and 6 will have user supplied values. If DFALT7 = 0, the last four lines of input data must be omitted, and loadings M = 2, 3, 4, and 5 will have default values constructed from loadings M = 1 and 6. If DFALT7 > 0, each of the last four lines of input data must be provided, and loadings M = 2, 3, 4, and 5 will have user supplied values.

Thus the number of lines of data that must be provided per design run will vary depending on whether default values are acceptable or whether the user wishes to supply certain secondary parameter values. Note that although various lines may be omitted, those supplied must be complete and in the order indicated.

Output

The output for each design run, whether preliminary or detail design, repeats the two alphameric lines of input and displays the design parameter values used for that run. These parameters are tabulated and identified at the beginning of the design. Water parameters are listed separately. They include each of the water loading cases, M = 1 through 7. Values of DW, HEAD, and TAIL + S are listed for each of the loadings.

Messages. The execution of a design run is not attempted when the computer recognizes input parameters are unacceptable. When this happens, the output references a message giving the reason the run was not executed. These messages follow.

Message No. 1
TAIL2 + S is more than J.

Message No. 2
DW2 is more than H.

Message No. 3
J is more than F + S + H.

Message No. 4
ZPS is less than 0.707.

Message No. 5
HB is more than J.

Message No. 6
TAIL2 is more than F + DW2.

Message No. 7
HEAD2 is more than F + S + H.

Message No. 8
LB is less than arbitrary minimum of F + S.

Message No. 9
TTOE is less than 10. inches.

Message No. 10
HTOE + S is less than arbitrary minimum of 4 ft.

Message No. 11
HB is negative.

Sometimes an executing design can not be completed. This may occur during preliminary design or more rarely during detail design. When this happens, the design is cancelled and the output contains a message which identifies the source of the difficulty, if possible.

Preliminary designs. Figure 49 contains the output for two preliminary designs. The first design uses default values for all secondary parameters. The second design uses default values for all water parameters.

Except as otherwise noted, output values consist of distances, thicknesses, and concrete volumes. Units are feet, inches, and cubic yards respectively.

MONOLITHIC DESIGN = 0, 1, 2, or 3, written below the water loading list, indicates whether the number of TRANSVERSE APRON SLABS = 1, 2, or 3 was selected by the computer or specified by the user. MONOLITHIC DESIGN = 0 means selection was by the computer, > 0 means user specified.

The alpha group, C - HS - FA - SC - A, is an indicator or guide that is provided the user as a matter of interest. It is not required information. Each letter represents a design function or element. Each corresponding integer indicates which loading, M = 1 through 8, controlled the function or element. The letters and their meanings are, in order

C ≡ creep analyses, used to determine HCUT
H ≡ headwall thickness
S ≡ sidewall thickness
F ≡ flotation analysis
A ≡ apron thickness by panel design
S ≡ sliding analysis
C ≡ cutoff wall thickness; zero means wall was not incremented from its initial value
A ≡ cutoff wall effect on apron; zero means apron was not altered

The remainder of the spillway and wingwall preliminary design follows established nomenclature and units with one exception. The distance LEVEL in the wingwall design is given in inches.

Detail designs. The output for the detail design of a straight drop spillway includes several parts. Preliminary design results are repeated. The output gives final distance and thickness values (these will usually be identical to the preliminary design values). The output includes a listing of steel requirements for the various components of the structure. Pertinent schematic steel layouts, distribution curves, and envelopes, presented in the text should be referenced to properly identify output values.

MONOLITHIC DESIGN = 100, 101, 102, or 103 indicates whether the number of TRANSVERSE APRON SLABS = 1, 2, or 3 was selected by the computer or specified by the user. MONOLITHIC DESIGN = 100 means selection was by the computer, > 100 means user specified.

All spillway slab steel areas, A(N), are given in sq. in. per ft of width. Required steel areas for temperature and shrinkage are given for headwall and sidewall slabs. Values are given for concrete surfaces that are not exposed (ATSNX) and surfaces that are exposed (ATSX). The first set of values under SIDEWALL STEEL is for the thickness, TSW, and the second set is for the thickness, TSB. Headwall buttress, longitudinal sill, and transverse sill steel, ABUTT or A(N), are total area values given in sq. inches.

Spillway slab steel spacings, S(N), are given in inches center-to-center of bars. Headwall buttress, longitudinal sill, and transverse sill steel perimeters, PBUTT or P(N), are total perimeter values given in inches. Headwall buttress, longitudinal sill, and transverse sill web steel indices, AV/S and VC/V, are given in sq. in. per in. and lbs per lb, respectively.

Sidewall horizontal and vertical distances, X and Y, are given in ft. These distances locate the unit width strips for which steel requirements are determined.

Wingwall detail designs follow the design of the spillway proper. These are presented essentially as shown and described in TR-54.

Three example detail designs are provided. These are shown as Figures 50, 51, and 52. Each figure requires two pages. The first page contains the detail design through sidewall steel requirements. The second page contains the remainder of the detail design. The drop spillway of Figure 50 has one transverse apron slab, i.e., no longitudinal sill. The drop spillway of Figure 51 has two transverse apron slabs, i.e., one longitudinal sill. The drop spillway of Figure 52 has three transverse apron slabs, i.e., two longitudinal sills.

Miscellaneous Notes

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                              STRAIGHT DROP SPILLWAY
                                STRUCTURAL  DESIGN
                  ELASTIC ANALYSIS AND WORKING STRESS DESIGN ARE USED

                SPECIAL DESIGN PREPARED BY THE DESIGN UNIT AT HYATTSVILLE, MD.
                                       FOR

                 EXAMPLE SPECIAL DESIGNS FOR DROP SPILLWAY TECHNICAL RELEASE
                 JOAN FOR ESA - - - - - - - - - - - - - - - - - - -  10/2/76

              DESIGN PARAMETERS
H=  6.00  J = 20.00  CREEPP= 5.00  HB  =  7.72  GMH= 120.  GSH= 140.  KOH= 0.80              BAT = 0.0
F= 12.00  L = 20.00  FLOATR= 1.50  ZPS =  2.00  GMW= 120.  GSW= 140.  KOW= 0.80  KPW= 2.00  CFSS= 0.550
S=  2.00  LB= 40.00  SLIDER= 1.00  HTOE=  4.00  GMF= 120.  GSF= 140.  KOF= 0.80  KPF= 2.00  CFSC= 0.350
                     SWLDRN= 0.0   TTOE= 10.00

           DW2= 6.00  HEAD2=12.02  TAIL2= 7.02  HEAD1= 5.00

           M     DW         HEAD        TAIL+S
           1    6.00        12.02        9.02
           2    5.16        11.04        8.04
           3    4.26         9.98        6.98
           4    3.24         8.79        5.79
           5    2.04         7.39        4.39
           6    0.0          5.00       -1.00
           7    0.0         -1.00       -1.00

              MONOLITHIC DESIGN =   0. FOLLOWS

              TRIAL VALUES
TRANSVERSE APRON SLABS = 1                 C-HS-FA-SC-A= 6-15-16-11-0                        QUANT= 255.97
THW= 16.00  TSW= 25.00  TCUT= 23.00  HCUT=  8.86  HWFTG= 2.00  BUTT=  0.0    TBLS=  0.0   HESTUB=   4.00
TAP= 29.00  TSB= 25.00  TTOE= 10.00  HTOE=  4.00  SWFTG= 0.0                 BOTT=  0.0     HBAT=  10.00

              WINGWALL DESIGN - TRIAL VALUES                                                 QUANT=  33.10
TWW= 10.00  TWF= 10.00  BUP= 10.50  BDN=  6.00  LEVEL= 25.35  WPROJ= 17.68  WWLB= 13.94   VWING=  33.10
=======================================END PRELIMINARY DESIGN=============================================



==========================================================================================================

                              STRAIGHT DROP SPILLWAY
                                STRUCTURAL  DESIGN
                  ELASTIC ANALYSIS AND WORKING STRESS DESIGN ARE USED

                SPECIAL DESIGN PREPARED BY THE DESIGN UNIT AT HYATTSVILLE, MD.
                                       FOR

                 EXAMPLE SPECIAL DESIGNS FOR DROP SPILLWAY TECHNICAL RELEASE
                 JOAN FOR ESA - - - - - - - - - - - - - - - - - - -  10/2/76

              DESIGN PARAMETERS
H=  4.80  J =  7.28  CREEPR= 4.00  HB  =  3.22  GMH= 120.  GSH= 136.  KOH= 0.67              BAT = 0.0
F=  6.00  L = 39.60  FLOATR= 1.33  ZPS =  2.00  GMW= 120.  GSW= 136.  KOW= 0.67  KPW= 2.00  CFSS= 0.550
S=  1.28  LB= 19.80  SLIDER= 1.00  HTOE=  4.00  GMF= 120.  GSF= 136.  KOF= 0.67  KPF= 2.00  CFSC= 0.350
                     SWLDRN= 0.0   TTOE= 10.00

           DW2= 4.80  HEAD2= 8.40  TAIL2= 5.62  HEAD1= 2.78
           SOME HEADWALL HEADS HAVE BEEN ADJUSTED

           M     DW         HEAD        TAIL+S
           1    4.80        12.08        6.90
           2    4.13        11.41        6.11
           3    3.41         6.77        5.27
           4    2.59         5.81        4.31
           5    1.63         4.69        3.19
           6    0.0          2.78       -1.00
           7    0.0         -1.00       -1.00

              MONOLITHIC DESIGN =   0. FOLLOWS

              TRIAL VALUES
TRANSVERSE APRON SLABS = 2                 C-HS-FA-SC-A= 2-12-12-11-0                        QUANT=  90.69
THW= 10.00  TSW= 11.00  TCUT= 11.00  HCUT=  5.49  HWFTG= 3.08  BUTT=  0.0    TBLS= 12.00  HESTUB=   5.00
TAP= 15.00  TSB= 11.00  TTOE= 10.00  HTOE=  4.00  SWFTG= 2.50                BOTT=  0.0     HBAT=   3.64

              WINGWALL DESIGN - TRIAL VALUES                                                 QUANT=   4.79
TWW= 10.00  TWF= 10.00  BUP=  6.00  BDN=  1.50  LEVEL=  7.78  WPROJ=  5.63  WWLB=  8.75   VWING=   5.94
=======================================END PRELIMINARY DESIGN=============================================
```

Figure 49. Computer output, preliminary designs.

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                                        STRAIGHT DROP SPILLWAY
                                          STRUCTURAL  DESIGN
                          ELASTIC ANALYSIS AND WORKING STRESS DESIGN ARE USED

                      SPECIAL DESIGN PREPARED BY THE DESIGN UNIT AT HYATTSVILLE, MD.
                                                  FOR

                       EXAMPLE SPECIAL DESIGNS FOR DROP SPILLWAY TECHNICAL RELEASE
                       JOAN FOR ESA - - - - - - - - - - - - - - - - - - - - 2/3/77

                 DESIGN PARAMETERS
H=  7.20  J = 10.85  CREEPR= 4.00  HB  =  4.48  GMH= 120.  GSH= 136.  KOH= 0.67               BAT = 0.0
F=  6.00  L = 31.80  FLOATR= 1.33  ZPS =  2.00  GMW= 120.  GSW= 136.  KOW= 0.67  KPW= 2.00  CFSS= 0.550
S=  1.92  LB= 31.80  SLIDER= 1.00  HTOE=  4.00  GMF= 120.  GSF= 136.  KOF= 0.67  KPF= 2.00  CFSC= 0.350
                     SWLDRN= 0.0   TTOE= 10.00

              DW2= 7.20  HEAD2=11.84  TAIL2= 8.42  HEAD1= 3.42
              SUBMERGED FLOW OCCURS FOR SOME DISCHARGES
              SOME HEADWALL HEADS HAVE BEEN ADJUSTED

              M      DW          HEAD         TAIL+S
              1     7.20        15.12         10.34
              2     6.19        14.11          9.16
              3     5.11        13.03          7.90
              4     3.89        11.81          6.47
              5     2.45         6.28          4.78
              6     0.0          3.42         -1.00
              7     0.0         -1.00         -1.00

                 MONOLITHIC DESIGN = 100. FOLLOWS

                 TRIAL VALUES
TRANSVERSE APRON SLABS = 1                  C-HS-FA-SC-A= 4-34-14-31-0                    QUANT= 158.46
THW= 11.00  TSW= 15.00  TCUT= 12.00  HCUT=  4.91  HWFTG= 2.75  BUTT=  0.0   THLS=  0.0   HESTUR=   4.00
TAP= 23.00  TSB= 15.00  TTOE= 10.00  HTOE=  4.00  SWFTG= 2.50               BOTT=  0.0     HBAT=   5.43

                 DETAIL DESIGN
TRANSVERSE APRON SLABS = 1                                                                QUANT= 158.46
THW= 11.00  TSW= 15.00  TCUT= 12.00  HCUT=  4.91  HWFTG= 2.75  BUTT=  0.0   TBLS=  0.0   HESTUR=   4.00
TAP= 23.00  TSB= 15.00  TTOE= 10.00  HTOE=  4.00  SWFTG= 2.50               BOTT=  0.0     HBAT=   5.43

                 STEEL REQUIREMENTS
0       HEADWALL STEEL
        ATSNX=  0.13                 ATSX =  0.26

        A( 1)=  0.13                 A( 5)=  0.69
        A( 2)=  0.13                 A( 6)=  0.31
        A( 3)=  0.41                 A( 7)=  0.69
        A( 4)=  1.09                 A( 8)=  0.31
        S( 4)= 18.00                 S( 5)= 12.73                 S( 7)= 18.00

0       SIDEWALL STEEL
        ATSNX=  0.18                 ATSX =  0.36
        ATSNX=  0.18                 ATSX =  0.36

            X=  7.56                     X= 19.43                     X= 31.30
        A( 9)=  0.36                 A(13)=  0.36                 A(17)=  0.36
        A(10)=  0.18                 A(14)=  0.18                 A(18)=  0.36
        A(11)=  0.56                 A(15)=  0.18                 A(19)=  0.18
        A(12)=  2.03                 A(16)=  0.79                 A(20)=  0.18
        S(12)= 15.72                 S(16)= 18.00                 S(20)= 18.00

            Y= 10.08                     Y=  9.08                     Y=  4.54
        A(21)=  0.18                 A(24)=  0.18                 A(27)=  0.18
        A(22)=  0.25                 A(25)=  0.26                 A(28)=  0.18
        A(23)=  1.11                 A(26)=  1.13                 A(29)=  0.47
        S(23)= 18.00                 S(26)= 18.00                 S(29)= 18.00

        A(30)=  0.36
```

Figure 50. Computer output, detail design, no longitudinal sill.

```
0       APRON LONGITUDINAL STEEL
        A(31)=  0.55    A(33)=  0.55    A(35)=  0.55    A(37)=  0.55    A(39)=  0.55
        A(32)=  1.03    A(34)=  0.52    A(36)=  0.28    A(38)=  0.28    A(40)=  0.59
        S(31)= 18.00                                                    S(39)= 18.00
        S(32)= 18.00                                                    S(40)= 18.00

        APRON TRANSVERSE STEEL
        A(41)=  0.55    A(43)=  0.61
        A(42)=  1.09    A(44)=  0.28
        S(41)= 18.00
        S(42)= 18.00

        A(49)=  0.55    A(51)=  0.55
        A(50)=  0.72    A(52)=  0.28
        S(49)= 18.00
        S(50)= 18.00

        A(57)=  0.55    A(59)=  0.55
        A(58)=  0.28    A(60)=  0.28
        S(57)= 18.00
        S(58)= 18.00

0       HEADWALL FOOTING STEEL
        A(65)=  0.28    S(65)= 18.00    A(66)=  0.28    S(66)= 18.00

0       SIDEWALL FOOTING STEEL
        A(67)=  0.28    S(67)= 18.00    A(69)=  0.28    S(69)= 18.00
        A(68)=  0.28    S(68)= 18.00    A(70)=  0.28    S(70)= 18.00

        CUTOFF WALL STEEL               TOEWALL STEEL
        A(71)=  1.09    S(71)=  9.19    A(72)=  0.24    S(72)= 18.00

        HEADWALL EXTENSION STUB STEEL
        A(73)=  0.50    S(73)= 17.91    A(74)=  0.76    S(74)= 12.11

0       TRANSVERSE SILL STEEL
        A(81)=  0.0                     A(83)=  2.25
        A(82)=  2.28                    A(84)=  0.0
        P(81)=  0.0
        P(82)=  2.12
        AV/S =  0.0
         V/VC=  0.92

        APPROXIMATION OF LIMITING STEEL AREA VALUES AT JUNCTION OF HEADWALL, SIDEWALL, AND HEADWALL EXTENSION
        ** LIMITING VALUES NOTWITHSTANDING - NO STEEL AREA SHOULD BE TAKEN LESS THAN PREVIOUSLY DETERMINED **
              ABOVE WEIR - IN SIDEWALL - TO BALANCE HEADWALL EXTENSION STUB STEEL
                    A(73)BHE=  0.37
              BELOW WEIR - IN SIDEWALL
                    A(23)LSA=  0.90      AT Y=  7.92
                    A(26)LSA=  0.90      AT Y=  7.92
                    A(29)LSA=  0.85      AT Y=  4.54
              BELOW WEIR - IN HEADWALL
                    A( 5)LSA=  0.72      AT SIDEWALL SUPPORT
                    A( 6)LSA=  0.28      AT MIDSPAN(S)
                    A( 7)LSA=  0.40      AT SIDEWALL SUPPORT
                    A( 8)LSA=  0.61      AT MIDSPAN(S)

                WINGWALL DESIGN - TRIAL VALUES                                              QUANT=  10.07
   TWW= 10.00  TWF= 10.00  BUP=  8.50  BDN=  1.50  LEVEL= 11.21  WPROJ=  8.03  WWLB= 11.95  VWING=  11.72

                WINGWALL DESIGN - DETAIL DESIGN                                             QUANT=  10.07
   TWW= 10.00  TWF= 10.00  BUP=  8.50  BDN=  1.50  LEVEL= 11.21  WPROJ=  8.03  WWLB= 11.95  VWING=  11.72

                STEEL REQUIREMENTS
        AREA OF TIE=  0.34

        SECTION AT ARTICULATION JOINT
        A( 1)=  0.24    S( 1)= 18.00                                    A( 4)=  0.18    S( 4)= 18.00
        A( 2)=  0.12    S( 2)= 18.00                                    A( 5)=  0.54    S( 5)= 18.00
        A( 3)=  0.18    S( 3)= 18.00                                    A( 6)=  0.71    S( 6)= 18.00

        SECTION AT UPPER THIRD POINT
        A( 7)=  0.24    S( 7)= 18.00                                    A(10)=  0.12    S(10)= 18.00
        A( 8)=  0.12    S( 8)= 18.00                                    A(11)=  0.22    S(11)= 18.00
        A( 9)=  0.12    S( 9)= 18.00                                    A(12)=  0.30    S(12)= 18.00

        SECTION AT LOWER THIRD POINT
        A(13)=  0.24    S(13)= 18.00                                    A(16)=  0.12    S(16)= 18.00
        A(14)=  0.12    S(14)= 18.00                                    A(17)=  0.12    S(17)= 18.00
        A(15)=  0.12    S(15)= 18.00                                    A(18)=  0.12    S(18)= 18.00

=======================================END DETAIL DESIGN========================================

      END OF INPUT DATA CARDS, END OF JOB
```

Figure 50. Computer output, detail design, continued.

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                              STRAIGHT DROP SPILLWAY
                                STRUCTURAL  DESIGN
                  ELASTIC ANALYSIS AND WORKING STRESS DESIGN ARE USED

              SPECIAL DESIGN PREPARED BY THE DESIGN UNIT AT HYATTSVILLE, MD.
                                       FOR

              EXAMPLE SPECIAL DESIGNS FOR DROP SPILLWAY TECHNICAL RELEASE
              JOAN FOR ESA - - - - - - - - - - - - - - - - - - - - - 10/2/76

            DESIGN PARAMETERS
H=  4.80  J = 13.28  CREEPR= 4.00  HH  =  5.34  GMH= 120.  GSH= 136.  KOH= 0.67               BAT = 0.0
F= 12.00  L = 32.40  FLOATR= 1.33  ZPS =  2.00  GMW= 120.  GSW= 136.  KOW= 0.67  KPW= 2.00  CFSS= 0.550
S=  1.28  LB= 21.60  SLIDER= 1.00  HTOE=  4.00  GMF= 120.  GSF= 136.  KOF= 0.67  KPF= 2.00  CFSC= 0.350
                     SWLDRN= 0.0   TTOE= 10.00

          DW2= 4.80  HEAD2= 9.90  TAIL2= 5.62  HEAD1= 4.28

          M     DW         HEAD        TAIL+S
          1    4.80        9.90         6.90
          2    4.13        9.11         6.11
          3    3.41        8.27         5.27
          4    2.59        7.31         4.31
          5    1.63        6.19         3.19
          6    0.0         4.28        -1.00
          7    0.0        -1.00        -1.00

            MONOLITHIC DESIGN = 100. FOLLOWS

            TRIAL VALUES
TRANSVERSE APRON SLABS = 2                 C-HS-FA-SC-A= 6-15-15-11-1                            QUANT= 111.56
THW= 12.00  TSW= 12.00  TCUT= 15.00  HCUT=  4.99  HWFTG= 3.00  BUTT=  6.00  TBLS= 12.00  HESTUB=    5.00
TAP= 17.00  TSB= 12.00  TTOE= 12.00  HTOE=  4.50  SWFTG= 0.0                BOTT=  0.0     HBAT=    6.64

            DETAIL DESIGN
TRANSVERSE APRON SLABS = 2                                                                       QUANT= 111.56
THW= 12.00  TSW= 12.00  TCUT= 15.00  HCUT=  4.99  HWFTG= 3.00  BUTT=  6.00  TBLS= 12.00  HESTUB=    5.00
TAP= 17.00  TSB= 12.00  TTOE= 12.00  HTOE=  4.50  SWFTG= 0.0                BOTT=  0.0     HBAT=    6.64

            STEEL REQUIREMENTS
0      HEADWALL STEEL
       ATSNX=  0.14                 ATSX =  0.29

       A( 1)=  0.14                 A( 5)=  0.88
       A( 2)=  0.14                 A( 6)=  0.40
       A( 3)=  0.38                 A( 7)=  0.88
       A( 4)=  1.16                 A( 8)=  0.40
       S( 4)= 10.94                 S( 5)= 18.00                 S( 7)= 16.72

0      SIDEWALL STEEL
       ATSNX=  0.14                 ATSX =  0.29
       ATSNX=  0.14                 ATSX =  0.29

          X=  9.04                     X= 15.07                     X= 21.10
       A( 9)=  0.29                 A(13)=  0.29                 A(17)=  0.29
       A(10)=  0.14                 A(14)=  0.29                 A(18)=  0.29
       A(11)=  0.18                 A(15)=  0.14                 A(19)=  0.14
       A(12)=  1.45                 A(16)=  0.52                 A(20)=  0.14
       S(12)= 17.56                 S(16)= 18.00                 S(20)= 18.00

          Y= 13.28                     Y=  8.28                     Y=  4.14
       A(21)=  0.29                 A(24)=  0.14                 A(27)=  0.14
       A(22)=  0.29                 A(25)=  0.21                 A(28)=  0.14
       A(23)=  0.49                 A(26)=  1.05                 A(29)=  0.55
       S(23)= 18.00                 S(26)= 18.00                 S(29)= 18.00

       A(30)=  0.42
```

Figure 51. Computer output, detail design, one longitudinal sill.

```
0       APRON LONGITUDINAL STEEL
        A(31)=  0.41    A(33)=  0.41    A(35)=  0.41    A(37)=  0.41    A(39)=  0.41
        A(32)=  1.63    A(34)=  0.81    A(36)=  0.20    A(38)=  0.20    A(40)=  0.28
        S(31)= 18.00                                                    S(39)= 18.00
        S(32)= 18.00                                                    S(40)= 18.00

        APRON TRANSVERSE STEEL
        A(41)=  0.41    A(43)=  0.41    A(45)=  0.41
        A(42)=  0.83    A(44)=  0.20    A(46)=  0.56
        S(41)= 18.00                    S(45)= 18.00
        S(42)= 18.00                    S(46)= 18.00

        A(49)=  0.41    A(51)=  0.41    A(53)=  0.41
        A(50)=  0.86    A(52)=  0.20    A(54)=  0.28
        S(49)= 18.00                    S(53)= 18.00
        S(50)= 18.00                    S(54)= 18.00

        A(57)=  0.41    A(59)=  0.41    A(61)=  0.41
        A(58)=  0.36    A(60)=  0.20    A(62)=  0.24
        S(57)= 18.00                    S(61)= 18.00
        S(58)= 18.00                    S(62)= 18.00

0       HEADWALL FOOTING STEEL
        A(65)=  0.22    S(65)= 18.00    A(66)=  0.20    S(66)= 18.00


        CUTOFF WALL STEEL               TOEWALL STEEL
        A(71)=  1.35    S(71)=  8.84    A(72)=  0.37    S(72)= 18.00

        HEADWALL EXTENSION STUB STEEL
        A(73)=  0.37    S(73)= 18.00    A(74)=  1.24    S(74)= 10.95

0       HEADWALL BUTTRESS STEEL
        ABUTT=  4.87    PBUTT=  4.40    AV/S =  0.0193   V/VC=  1.46

0       LONGITUDINAL SILL STEEL
        A(75)=  0.0     A(77)=  1.46    A(79)=  0.0
        A(76)=  3.23    A(78)=  0.0     A(80)=  1.99
        P(75)=  0.0                     P(79)=  3.92
        P(76)=  5.67                    P(80)=  3.90
        AV/S =  0.0441                  AV/S =  0.0172
         V/VC=  2.05                     V/VC=  1.41

0       TRANSVERSE SILL STEEL
        A(81)=  0.0                     A(83)=  3.28
        A(82)=  2.62                    A(84)=  0.0
        P(81)=  0.0
        P(82)=  1.98
        AV/S =  0.0
         V/VC=  0.72

        APPROXIMATION OF LIMITING STEEL AREA VALUES AT JUNCTION OF HEADWALL, SIDEWALL, AND HEADWALL EXTENSION
        ** LIMITING VALUES NOTWITHSTANDING - NO STEEL AREA SHOULD BE TAKEN LESS THAN PREVIOUSLY DETERMINED **
               ABOVE WEIR - IN SIDEWALL - TO BALANCE HEADWALL EXTENSION STUB STEEL
                     A(73)BHE=  0.37
               BELOW WEIR - IN SIDEWALL
                     A(23)LSA=  0.87      AT Y= 13.28
                     A(26)LSA=  1.38      AT Y=  8.28
                     A(29)LSA=  1.30      AT Y=  4.14
               BELOW WEIR - IN HEADWALL
                     A( 5)LSA=  0.50      AT SIDEWALL SUPPORT
                     A( 5)LSA=  1.07      AT INTERIOR SUPPORT(S)
                     A( 6)LSA=  0.50      AT MIDSPAN(S)
                     A( 7)LSA=  0.14      AT SIDEWALL SUPPORT
                     A( 7)LSA=  1.25      AT INTERIOR SUPPORT(S)
                     A( 8)LSA=  0.59      AT MIDSPAN(S)

                  WINGWALL DESIGN - TRIAL VALUES                                              QUANT=  16.81
   TWW= 10.00  TWF= 10.00  BUP=  9.00  BDN=  3.00  LEVEL=  8.49  WPROJ= 10.89  WWLB= 12.91    VWING=  16.81

                  WINGWALL DESIGN - DETAIL DESIGN                                             QUANT=  16.81
   TWW= 10.00  TWF= 10.00  BUP=  9.00  BDN=  3.00  LEVEL=  8.49  WPROJ= 10.89  WWLB= 12.91    VWING=  16.81

                  STEEL REQUIREMENTS
        AREA OF TIE=  0.59

        SECTION AT ARTICULATION JOINT
        A( 1)=  0.24    S( 1)= 18.00                                    A( 4)=  0.26    S( 4)= 18.00
        A( 2)=  0.12    S( 2)= 18.00                                    A( 5)=  0.78    S( 5)= 18.00
        A( 3)=  0.13    S( 3)= 18.00                                    A( 6)=  1.03    S( 6)= 18.00

        SECTION AT UPPER THIRD POINT
        A( 7)=  0.24    S( 7)= 18.00                                    A(10)=  0.12    S(10)= 18.00
        A( 8)=  0.12    S( 8)= 18.00                                    A(11)=  0.34    S(11)= 18.00
        A( 9)=  0.12    S( 9)= 18.00                                    A(12)=  0.45    S(12)= 18.00

        SECTION AT LOWER THIRD POINT
        A(13)=  0.24    S(13)= 18.00                                    A(16)=  0.12    S(16)= 18.00
        A(14)=  0.12    S(14)= 18.00                                    A(17)=  0.12    S(17)= 18.00
        A(15)=  0.12    S(15)= 18.00                                    A(18)=  0.15    S(18)= 18.00

==============================================END DETAIL DESIGN=============================================
```

Figure 51. Computer output, detail design, continued.

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                                         STRAIGHT DROP SPILLWAY
                                           STRUCTURAL  DESIGN
                              ELASTIC ANALYSIS AND WORKING STRESS DESIGN ARE USED

                         SPECIAL DESIGN PREPARED BY THE DESIGN UNIT AT HYATTSVILLE, MD.
                                                 FOR

                          EXAMPLE SPECIAL DESIGNS FOR DROP SPILLWAY TECHNICAL RELEASE
                          JOAN FOR ESA - - - - - - - - - - - - - - - - - - - -  10/2/76

                   DESIGN PARAMETERS
 H=  2.40  J =  8.26  CREEPR= 4.00  HB  =  3.57  GMH= 120.  GSH= 136.  KOH= 0.67              BAT = 0.0
 F=  6.00  L = 47.70  FLOATR= 1.33  ZPS =  2.00  GMW= 120.  GSW= 136.  KOW= 0.67  KPW= 2.00  CFSS= 0.550
 S=  0.64  LB= 31.80  SLIDER= 1.00  HTOE=  4.00  GMF= 120.  GSF= 136.  KOF= 0.67  KPF= 2.00  CFSC= 0.350
                      SWLDRN= 0.0   TTOE= 10.00

                DW2= 2.40  HEAD2= 7.12  TAIL2= 7.12  HEAD1= 2.14
                SUBMERGED FLOW OCCURS FOR SOME DISCHARGES
                SOME HEADWALL HEADS HAVE BEEN ADJUSTED

                M     DW          HEAD          TAIL+S
                1     2.40        9.04           7.76
                2     2.06        8.70           6.76
                3     1.70        5.70           5.70
                4     1.30        4.83           4.48
                5     0.82        3.83           3.06
                6     0.0         2.14          -1.00
                7     0.0        -1.00          -1.00

                   MONOLITHIC DESIGN = 103. FOLLOWS

                   TRIAL VALUES
 TRANSVERSE APRON SLABS = 3                        C-HS-FA-SC-A= 6-46-17-20-0                   QUANT= 124.90
 THW= 10.00  TSW= 10.00  TCUT= 16.00  HCUT=  5.50  HWFTG= 3.17  BUTT=  3.00  TBLS= 12.00  HESTUB=   4.00
 TAP= 11.00  TSB= 10.00  TTOE= 16.00  HTOE=  5.50  SWFTG= 1.00               BOTT=  2.25    HBAT=   4.13

                   DETAIL DESIGN
 TRANSVERSE APRON SLABS = 3                                                                     QUANT= 124.90
 THW= 10.00  TSW= 10.00  TCUT= 16.00  HCUT=  5.50  HWFTG= 3.17  BUTT=  3.00  TBLS= 12.00  HESTUB=   4.00
 TAP= 11.00  TSB= 10.00  TTOE= 16.00  HTOE=  5.50  SWFTG= 1.00               BOTT=  2.25    HBAT=   4.13

                   STEEL REQUIREMENTS
0        HEADWALL STEEL
         ATSNX=  0.12                ATSX =  0.24

         A( 1)=  0.12                A( 5)=  0.24
         A( 2)=  0.12                A( 6)=  0.24
         A( 3)=  0.12                A( 7)=  0.24
         A( 4)=  0.25                A( 8)=  0.24
         S( 4)= 18.00                S( 5)= 18.00                S( 7)= 18.00

0        SIDEWALL STEEL
         ATSNX=  0.12                ATSX =  0.24
         ATSNX=  0.12                ATSX =  0.24

             X=  4.52                    X= 17.91                    X= 31.30
         A( 9)=  0.24                A(13)=  0.24                A(17)=  0.24
         A(10)=  0.12                A(14)=  0.12                A(18)=  0.24
         A(11)=  0.22                A(15)=  0.19                A(19)=  0.12
         A(12)=  0.82                A(16)=  0.72                A(20)=  0.12
         S(12)= 18.00                S(16)= 18.00                S(20)= 18.00

             Y=  6.64                    Y=  6.14                    Y=  3.07
         A(21)=  0.12                A(24)=  0.12                A(27)=  0.12
         A(22)=  0.12                A(25)=  0.12                A(28)=  0.12
         A(23)=  0.43                A(26)=  0.44                A(29)=  0.21
         S(23)= 18.00                S(26)= 18.00                S(29)= 18.00

         A(30)=  0.24
```

Figure 52. Computer output, detail design, two longitudinal sills.

```
0       APRON LONGITUDINAL STEEL
        A(31)=  0.26    A(33)=  0.26    A(35)=  0.26    A(37)=  0.55    A(39)=  1.10
        A(32)=  0.48    A(34)=  0.24    A(36)=  0.13    A(38)=  0.13    A(40)=  0.20
        S(31)= 18.00                                                    S(39)= 18.00
        S(32)= 18.00                                                    S(40)= 18.00

        APRON TRANSVERSE STEEL
        A(41)=  0.26    A(43)=  0.26    A(45)=  0.26    A(47)=  0.26
        A(42)=  0.77    A(44)=  0.13    A(46)=  0.41    A(48)=  0.13
        S(41)= 18.00                    S(45)= 18.00
        S(42)=  3.01                    S(46)= 18.00

        A(49)=  0.26    A(51)=  0.26    A(53)=  0.26    A(55)=  0.26
        A(50)=  0.70    A(52)=  0.13    A(54)=  0.23    A(56)=  0.13
        S(49)= 18.00                    S(53)= 18.00
        S(50)=  3.14                    S(54)= 18.00

        A(57)=  0.26    A(59)=  0.26    A(61)=  0.26    A(63)=  0.26
        A(58)=  0.54    A(60)=  0.13    A(62)=  0.19    A(64)=  0.13
        S(57)= 18.00                    S(61)= 18.00
        S(58)=  3.61                    S(62)= 18.00

0       HEADWALL FOOTING STEEL
        A(65)=  0.18    S(65)= 18.00    A(66)=  0.13    S(66)= 18.00

0       SIDEWALL FOOTING STEEL
        A(67)=  0.13    S(67)= 18.00    A(69)=  0.13    S(69)= 18.00
        A(68)=  0.13    S(68)= 18.00    A(70)=  0.13    S(70)= 18.00

        CUTOFF WALL STEEL               TOEWALL STEEL
        A(71)=  0.47    S(71)= 18.00    A(72)=  0.56    S(72)= 18.00

        HEADWALL EXTENSION STUB STEEL
        A(73)=  0.25    S(73)= 18.00    A(74)=  0.50    S(74)= 18.00

0       HEADWALL BUTTRESS STEEL
        ABUTT=  0.74    PBUTT=  1.28    AV/S =  0.0      V/VC=  0.44

0       LONGITUDINAL SILL STEEL
        A(75)=  0.0     A(77)=  1.40    A(79)=  0.0
        A(76)=  3.54    A(78)=  0.0     A(80)=  1.57
        P(75)=  0.0                     P(79)=  1.23
        P(76)=  4.05                    P(80)=  1.22
        AV/S =  0.0195                  AV/S =  0.0
         V/VC=  1.46                     V/VC=  0.44

0       TRANSVERSE SILL STEEL
        A(81)=  0.79                    A(83)=  0.24
        A(82)=  0.24                    A(84)=  0.80
        P(81)=  0.87
        P(82)=  0.17
        AV/S =  0.0
         V/VC=  0.17

        APPROXIMATION OF LIMITING STEEL AREA VALUES AT JUNCTION OF HEADWALL, SIDEWALL, AND HEADWALL EXTENSION
        ** LIMITING VALUES NOTWITHSTANDING - NO STEEL AREA SHOULD BE TAKEN LESS THAN PREVIOUSLY DETERMINED **
              ABOVE WEIR - IN SIDEWALL - TO BALANCE HEADWALL EXTENSION STUB STEEL
                    A(73)BHE=  0.25
              BELOW WEIR - IN SIDEWALL
                    A(23)LSA=  0.46      AT Y=  6.64
                    A(26)LSA=  0.48      AT Y=  6.14
                    A(29)LSA=  0.47      AT Y=  3.07
              BELOW WEIR - IN HEADWALL
                    A( 5)LSA=  0.21      AT SIDEWALL SUPPORT
                    A( 5)LSA=  0.25      AT INTERIOR SUPPORT(S)
                    A( 6)LSA=  0.25      AT MIDSPANS OF EXTERIOR SPANS
                    A( 6)LSA=  0.23      AT MIDSPAN OF INTERIOR SPAN
                    A( 7)LSA=  0.02      AT SIDEWALL SUPPORT
                    A( 7)LSA=  0.29      AT INTERIOR SUPPORT(S)
                    A( 8)LSA=  0.33      AT MIDSPANS OF EXTERIOR SPANS
                    A( 8)LSA=  0.20      AT MIDSPAN OF INTERIOR SPAN

                 WINGWALL DESIGN - TRIAL VALUES                                                    QUANT=    8.41
   TWW= 10.00  TWF= 10.00  BUP=  5.50  BDN=  2.50  LEVEL=  7.07  WPROJ=  7.07  WWLB=  8.12  VWING=    8.85

                 WINGWALL DESIGN - DETAIL DESIGN                                                   QUANT=    8.41
   TWW= 10.00  TWF= 10.00  BUP=  5.50  BDN=  2.50  LEVEL=  7.07  WPROJ=  7.07  WWLB=  8.12  VWING=    8.85

                 STEEL REQUIREMENTS
        AREA OF TIE=  0.19

        SECTION AT ARTICULATION JOINT
        A( 1)=  0.24    S( 1)= 18.00                                    A( 4)=  0.12    S( 4)= 18.00
        A( 2)=  0.12    S( 2)= 18.00                                    A( 5)=  0.17    S( 5)= 18.00
        A( 3)=  0.12    S( 3)= 18.00                                    A( 6)=  0.23    S( 6)= 18.00

        SECTION AT UPPER THIRD POINT
        A( 7)=  0.24    S( 7)= 18.00                                    A(10)=  0.12    S(10)= 18.00
        A( 8)=  0.12    S( 8)= 18.00                                    A(11)=  0.12    S(11)= 18.00
        A( 9)=  0.12    S( 9)= 18.00                                    A(12)=  0.14    S(12)= 18.00

        SECTION AT LOWER THIRD POINT
        A(13)=  0.24    S(13)= 18.00                                    A(16)=  0.12    S(16)= 18.00
        A(14)=  0.12    S(14)= 18.00                                    A(17)=  0.12    S(17)= 18.00
        A(15)=  0.12    S(15)= 18.00                                    A(18)=  0.12    S(18)= 18.00
=======================================END DETAIL DESIGN==========================================

      END OF INPUT DATA CARDS, END OF JOB
```

Figure 52. Computer output, detail design, continued.