

12.0 CHUTE SPILLWAYS

12.1 General

Reinforced concrete chute spillways are the most frequently used type of service spillway for providing flood protection for earthfill dams, particularly those with larger catchment areas and higher consequence of failure classifications. In general, the service spillway is designed to pass floods up to a chosen SDF under a specific amount of surcharge above the reservoir FSL. Under more extreme flood events up to and including the IDF, the service spillway by itself or possibly in conjunction with an auxiliary spillway, is designed to pass these floods without overtopping the dam.

Guidelines for establishing the IDF are usually based on the consequences of failure of the dam as discussed in the CDA (1999). For smaller, low consequence dams, further guidelines for establishing the IDF are provided in Section 13.1.

12.2 General Arrangement

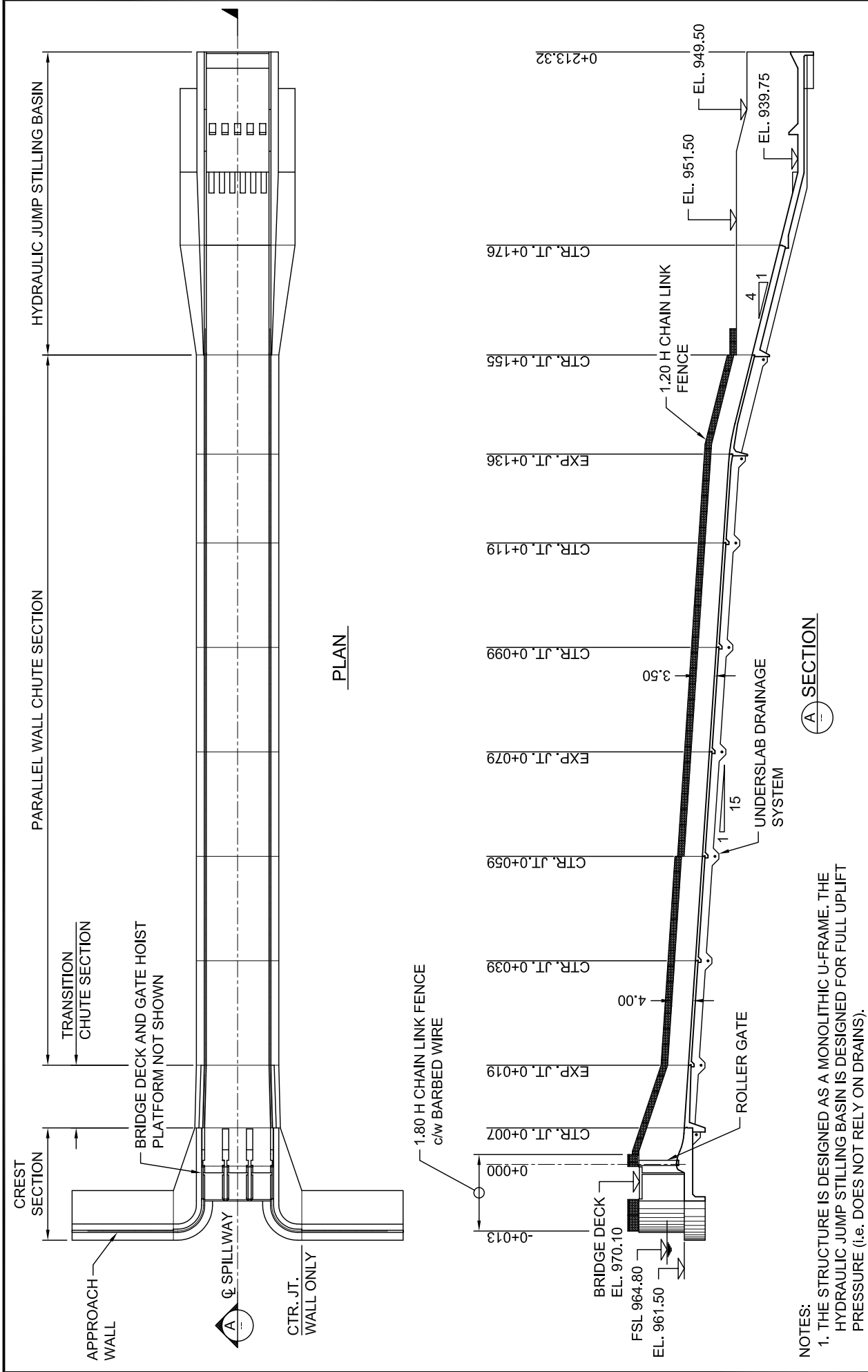
A typical arrangement for a chute spillway includes an entrance channel, a control section, a chute section, a terminal structure, and an outlet channel. General arrangements of the chute spillway at the Little Bow River Dam, and the larger spillway at St. Mary Dam are shown on Figures 12-1 and 12.2, respectively.

12.3 Location and Layout

Normally, the chute spillway will have a straight centreline and will be located at one of the abutments of the dam. The preference is to align the terminal structure of the chute spillway with the outlet channel such that a horizontal curve is not required within the channel until well downstream. An entrance channel is normally required to connect the structure with the reservoir. Horizontal curves are often needed in the entrance channel, and are acceptable provided that the flow velocities are low and the curves do not cause adverse flow conditions (flow separation, back eddies, flow disturbance) at the control section.

Normally, the control section of the spillway structure is placed in line with, or upstream of the dam centreline. Downstream of the control section, the vertical profile of the chute section is designed to accommodate the change in elevation between the dam abutment and outlet channel. Depending on the natural topography, foundation conditions, and the extent of excavation required, the chute profile could consist of either a mildly sloping upstream section with a vertical curve and a steep downstream section, or a steep section with a constant slope. As noted in Section 5.3.1, the chute profile should preferably follow the natural topography as closely as possible to minimize rebound due to deep excavations.

The chute slope should normally not be steeper than 3H:1V since steeper slopes make construction difficult and more expensive, increase the potential for downhill creep to occur, and result in difficult access for inspection and maintenance.



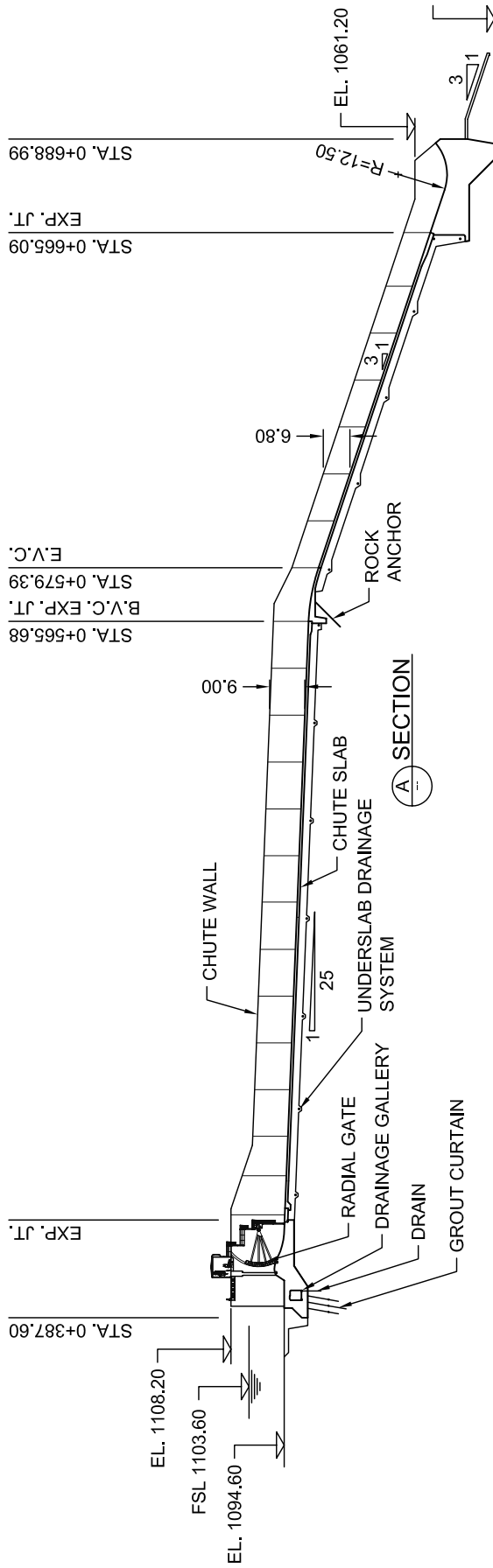
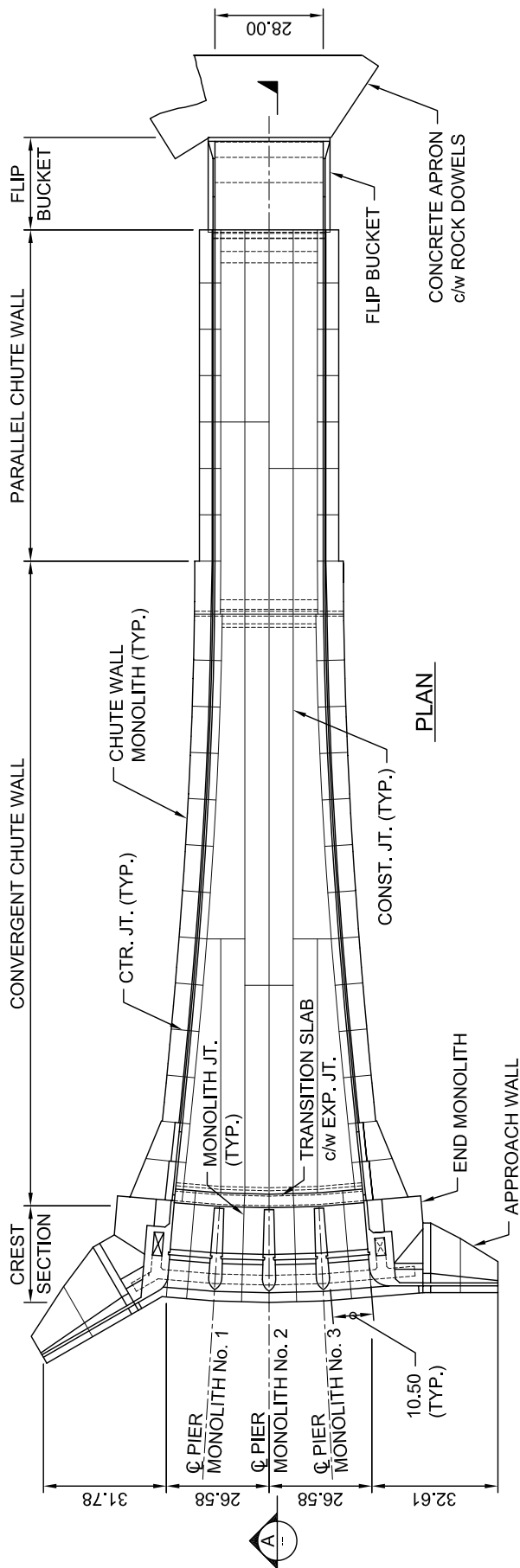
NOTES:

1. THE STRUCTURE IS DESIGNED AS A MONOLITHIC U-FRAME. THE HYDRAULIC JUMP STILLING BASIN IS DESIGNED FOR FULL UPLIFT PRESSURE (i.e. DOES NOT RELY ON DRAINS).

2. STATIONS, ELEVATIONS AND DIMENSIONS ARE IN METRES.

ALBERTA TRANSPORTATION CIVIL PROJECTS BRANCH		WATER CONTROL STRUCTURES - SELECTED DESIGN GUIDELINES	
GENERAL ARRANGEMENT OF THE CHUTE SPILLWAY AT THE LITTLE BOW RIVER DAM		FIGURE No.: 12-1	
ALBERTA ENVIRONMENT WATER MANAGEMENT OPERATIONS		CAD FILE: 99008A12-1.dwg	
DATE: November 2004			

SOURCE: ALBERTA TRANSPORTATION, CIVIL PROJECTS BRANCH, 2001



NOTE:

1. STATIONS, ELEVATIONS AND DIMENSIONS ARE IN METRES.

ALBERTA TRANSPORTATION
CIVIL PROJECTS BRANCH

ALBERTA ENVIRONMENT
WATER MANAGEMENT OPERATIONS

WATER CONTROL STRUCTURES - SELECTED DESIGN GUIDELINES

**GENERAL ARRANGEMENT OF THE CHUTE
SPILLWAY AT THE ST. MARY DAM**

SOURCE: ALBERTA INFRASTRUCTURE, CIVIL PROJECTS DIVISION, 1997.

DATE: November 2004
CAD FILE: 99008A12-2.dwg

FIGURE No.: 12-2

12.4 Seepage and Drainage Measures

12.4.1 Control Section

The main seepage reduction zone for a chute spillway is typically located at the control section area.

Depending on the foundation conditions, seepage reduction measures may typically include one or more of the following: an impervious blanket constructed within the cross section of the approach channel; an impervious embankment constructed behind the abutment walls; a cutoff wall; and a grout curtain which extends beneath the control structure.

In some instances, primarily for very large spillways, drainage measures may be required beneath the control section to reduce hydrostatic pressures. These measures may consist of pressure relief drains that discharge into a drainage gallery located within the crest section or into a separate drainage tunnel constructed well beneath the control section.

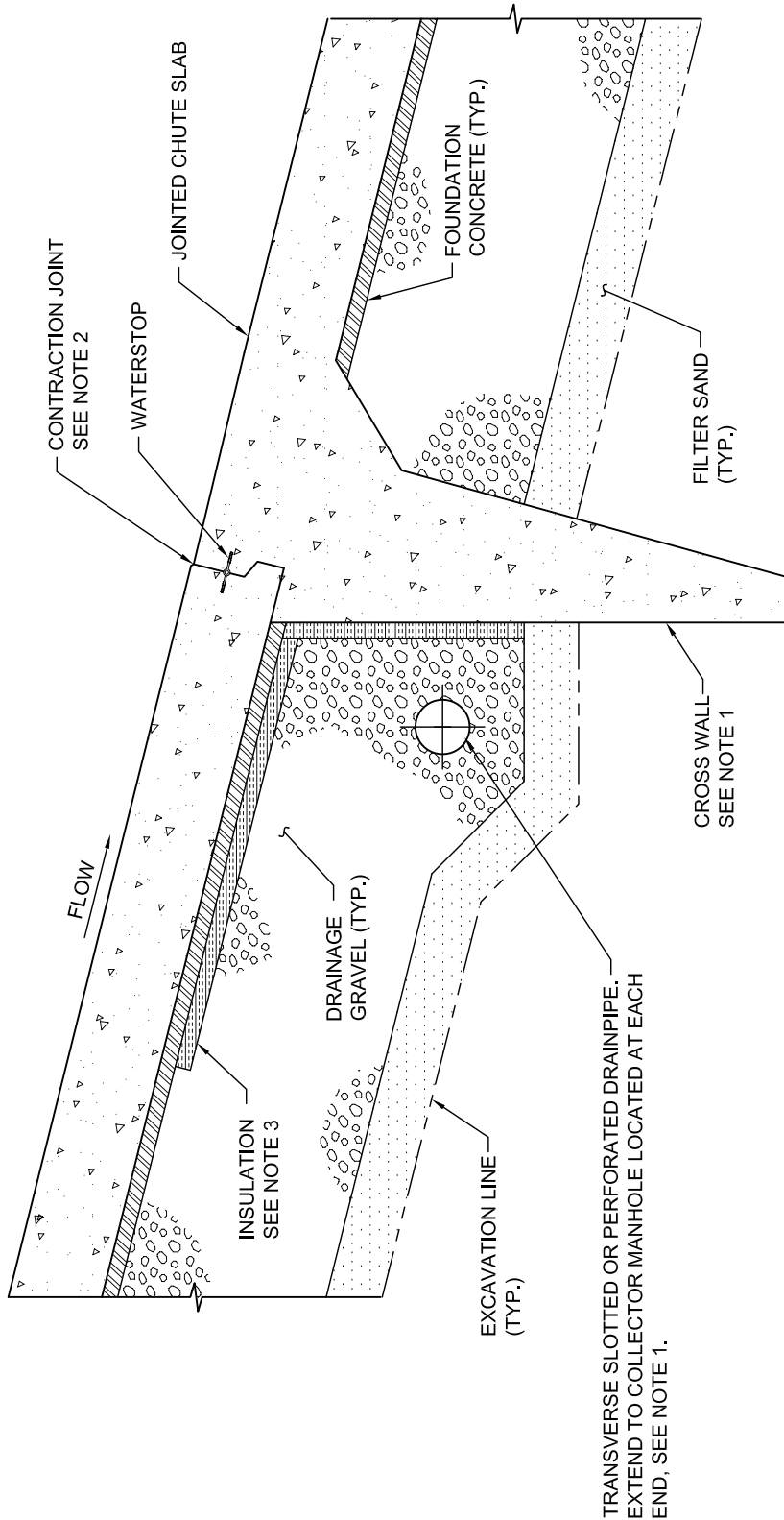
12.4.2 Chute Section

Beneath the chute, a drainage blanket typically consisting of drainage gravel and filter sand is usually provided to control seepage and relieve any hydrostatic pressures. The total thickness of the blanket is usually determined based on flow capacity and frost protection requirements. Typically, the thickness of the filter sand for the blanket is not less than 300 mm.

Within the blanket, transverse perforated drainpipes are typically provided at regular intervals to intercept and convey seepage flows into a collection manhole located at either end of each of the drainpipes. The drainpipes should preferably be located below the frost line; however in cases where this is not feasible, insulation may be used as shown on Figures 12-3 and 12-5. A longitudinal pipe is used to connect the manholes and convey seepage water into the outlet channel. The pipe should be located below the frost line and its termination point (i.e. daylight) should typically be located within the riprap zone placed immediately downstream of the structure. In some instances, the termination point may be turned into a hydraulic jump stilling basin (i.e. near the start of the hydraulic jump) to take advantage of the lowered water level.

Weep drains that daylight through the chute slab should normally not be used because they can cause ice build-up on the slab, which makes access for inspection and maintenance difficult, and are prone to freezing.

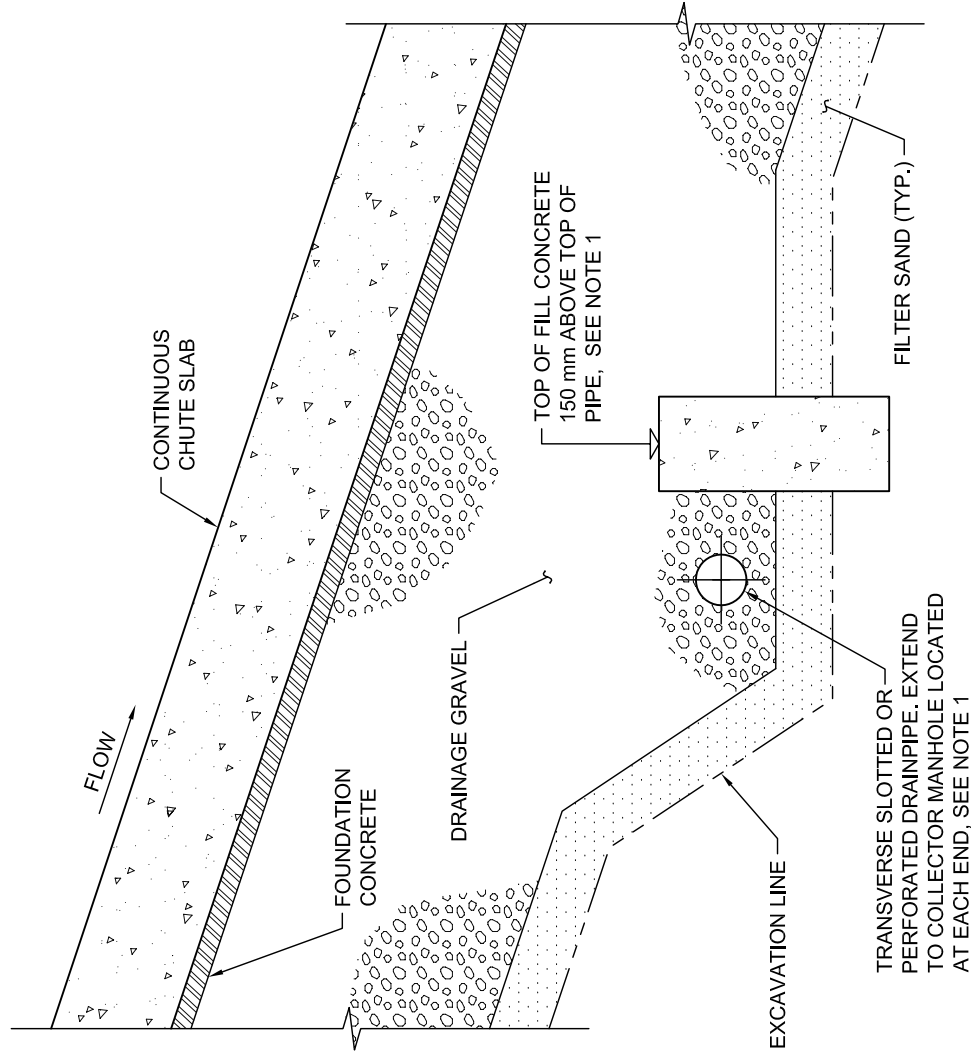
At transverse drain locations on steeper portions of the chute and other important locations, an impervious barrier should be constructed to prevent water from flowing past and possibly lifting the slab or surcharging the other downstream transverse drains. For a jointed slab configuration, cross walls as described in Section 12.6.5 and shown on Figure 12-3 are typically used as the impervious barrier, whereas for a continuous slab configuration, a concrete wall that extends from the foundation surface to 0.15 m above the top of the drainpipe has been used as shown on Figure 12-



- NOTES:
1. DESIGN THE BACKFILL DETAILS AT THE ENDS OF THE CROSS WALL AND AT THE COLLECTOR MANHOLES TO INCLUDE AN IMPERVIOUS BLOCK SO THAT WATER DOES NOT NORMALLY FLOW AROUND THE CROSS WALL OR MANHOLES, BUT IS FORCED INTO THE DRAINPIPE.
 2. FOR THINNER SLABS, TO AVOID THE RISK OF FAILING THE CONCRETE INTO THE WATERSTOP, CONSIDERATION SHOULD BE GIVEN TO EITHER
a) OMISSION OF THE SHEAR KEY OR b) LOCAL THICKENING OF THE SLAB.
 3. LIMIT EXTENT OF INSULATION TO THE MINIMUM NEEDED TO PROTECT THE TRANSVERSE DRAINPIPE FROM FREEZING.

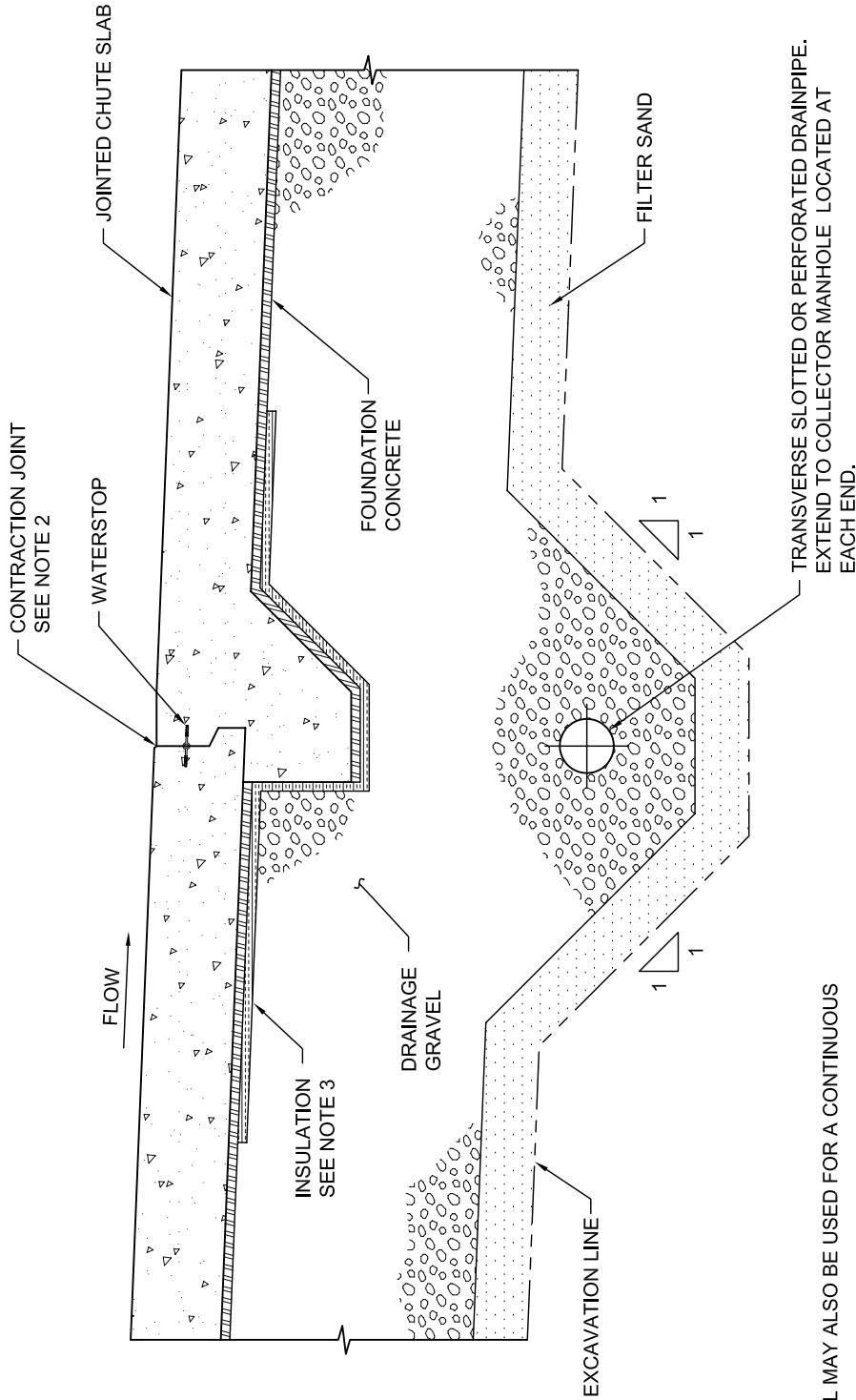
ALBERTA TRANSPORTATION
CIVIL PROJECTS BRANCH
ALBERTA ENVIRONMENT
WATER MANAGEMENT OPERATIONS

WATER CONTROL STRUCTURES - SELECTED DESIGN GUIDELINES
CROSS WALL ARRANGEMENT
DATE: November 2004
CAD FILE: 99008A12-3.dwg
FIGURE No.: 12-3



NOTES:

1. EXTEND THE FILL CONCRETE WALL AND DESIGN THE BACKFILL DETAILS AT THE ENDS OF THE WALL AND AT THE COLLECTOR MANHOLES SO THAT WATER DOES NOT NORMALLY FLOW AROUND THE WALL OR MANHOLES, BUT IS FORCED INTO THE DRAINPIPE.



NOTES:

1. SIMILAR TRENCH DETAIL MAY ALSO BE USED FOR A CONTINUOUS CHUTE SLAB.
2. FOR THINNER SLABS, TO AVOID THE RISK OF FAILING THE CONCRETE INTO THE WATERSTOP, CONSIDERATION SHOULD BE GIVEN TO EITHER
a) OMISSION OF THE SHEAR KEY OR b) LOCAL THICKENING OF THE SLAB.
3. LIMIT EXTENT OF INSULATION TO THE MINIMUM NEEDED TO PROTECT THE TRANSVERSE DRAINPIPE FROM FREEZING.
4. DOWNSTREAM OF THE TRENCH, AN EARTH OR CONCRETE BARRIER MAY BE REQUIRED TO FORCE WATER INTO THE DRAINPIPE, DEPENDING ON CONDITIONS AND DESIGN FACTORS.

ALBERTA TRANSPORTATION
CIVIL PROJECTS BRANCH
ALBERTA ENVIRONMENT
WATER MANAGEMENT OPERATIONS

WATER CONTROL STRUCTURES - SELECTED DESIGN GUIDELINES

TRENCH DETAIL FOR TRANSVERSE DRAINPIPE

CAD FILE: 99008A12-5.dwg
DATE: November 2004
FIGURE No.: 12-5

4. At locations where the chute is very flat, consideration can be given to directing water into the transverse drains by setting the drain within a trench as shown on Figure 12-5. In some cases, an impervious barrier located downstream of the trench may be required depending on conditions and design factors.

In general, the perforated transverse drainpipes should not be less than 200 mm in diameter in order to minimize the chance of plugging and to facilitate inspection and maintenance. At critical locations, the use of larger diameter transverse perforated pipes should be considered.

In cases where there is significant concern about the increased loading on the chute sidewalls that may occur due to water and/or frost action, a zone of pervious material can be placed immediately adjacent to the chute sidewalls to prevent such loads from developing. Generally, a single zone blanket ordinarily consisting of clean pitrun gravel or a two zone vertical blanket consisting of drainage gravel and filter sand combination, may be used. Factors such as drainage capacity, frost penetration, and construction considerations should be assessed to establish the appropriate width of the vertical blanket for a particular structure. However, it is suggested that a minimum width of 1.5 m should be used. Normally, this pervious zone is tied into the underslab drainage blanket. A layer of impervious material is also used to cap the pervious backfill placed adjacent the walls to prevent surface runoff water from infiltrating and surcharging the underslab drainage system.

12.4.3 Terminal Structure

For the terminal structure, the drainage measures that may be required will depend on the type of structure being provided.

As noted in Section 12.7.1, the first preference is to provide a narrow basin, where feasible, that can be designed to resist the entire uplift pressure that occurs due to the hydraulic jump by weight rather than relying on drains. However for wide basins where it is determined that the drainage system is viable and cost effective, a separate (i.e. isolated from the chute underslab drainage system in order to prevent surcharging) underslab drainage system consisting of a drainage blanket (filter sand and/or drainage gravel) and perforated drainpipes may be incorporated. Water collected by the transverse drainpipes would normally be conveyed into manholes that would also permit access for inspection and maintenance. Although the underslab drainage system will ordinarily be below the bed of the outlet channel, it may be possible in some cases to discharge some of the water by gravity from the manholes back into the stilling basin near the start of the jump. In other instances, pumping may be required. Careful attention to backfill provisions and cutoff requirements particularly along the sides and downstream end of the basin slab is required to prevent tailwater from entering and surcharging the underslab drainage system (i.e. becomes a pump).

For a gravity-designed flip bucket structure, a separate (i.e. isolated from the chute underslab drainage system in order to prevent surcharging) vertical drainage blanket consisting of drainage gravel and filter sand is typically provided on the upstream side of the flip bucket. A perforated drainpipe is located at the low point of the vertical drainage blanket, which generally corresponds to

the normal tailwater level of the outlet channel, to collect and convey any seepage water into the downstream outlet channel. This drainage system is used to improve the stability of the structure by relieving any hydrostatic pressures at the upstream side of the flip bucket.

12.5 Control Section

The control section typically consists of abutment walls, the crest section and, if required, a vehicle access deck and operating deck.

12.5.1 Abutment Walls

Abutment walls are provided to accommodate the transition from the trapezoidal entrance channel to the rectangular crest section. These walls also serve as a watertight barrier between the reservoir and the impervious backfill zone placed at the control section as part of the seepage control zone.

Typically, the top of wall elevation corresponds to the top of dam elevation, whereas the base elevation may vary based on the depth below grade needed for frost protection and for seepage cutoff purposes. Similarly, the walls should extend far enough into the abutment to minimize seepage around the ends.

Depending on the height, the walls may be designed as cantilever or semi-gravity retaining walls. Vertical contraction joints may be required to divide the wall into shorter segments in order to reduce cracking.

Examples of conditions and loading combinations that may apply in the design of a typical abutment wall are similar to those described in Section 12.5.2.

12.5.2 Crest Section

The crest section of a chute spillway can be either a controlled crest equipped with gates or an uncontrolled free overflow crest. A controlled crest is more frequently used where large spill capacities are needed since it normally results in a much narrower crest section and/or reduces the reservoir surcharge required for the same spillway discharge. It may also be used where operational flexibility is needed in order to manage reservoir levels and releases to meet multi-use objectives. Where appropriate, the uncontrolled crest option is preferred because it functions without an operator and requires less maintenance.

Freeboard requirements at the crest section are discussed in Section 7.0.

For a narrow crest, it may be possible to design the structure as a single monolith, however for a wide structure, it may be necessary to divide the crest section into individual structure monoliths by introducing vertical expansion/contraction joints in order to accommodate differential foundation deformations and thermal effects.

In the case of a wide controlled crest section, one of the jointing configurations described below has typically been used depending primarily on the amount of differential foundation movement that is expected across the crest section.

- Where only minor differential foundation movements are expected, the vertical expansion/contraction joint may be located at the centreline of each gate bay; however, the potential impact of differential foundation movements on the installation and long-term performance of the gates should be carefully examined. Dual waterstops (twin lines) are normally installed within the joint.
- Where large differential foundation movements are expected, the vertical expansion-contraction joint is normally located at the centreline of each pier (split pier). This arrangement provides for proper operation of the gate since there is no risk of differential movement occurring within the gate bay. The disadvantages of this arrangement include increased pier thickness, wider chute immediately downstream of the crest section, and the requirement for a more complex expansion/contraction joint located within the pier. Within the expansion/contraction joint, a back-up system to the twin lines of waterstops may be required to ensure that the joint remains watertight after the movements have occurred. Examples of back-up systems used include grout pipes, and a split pipe filled with asphalt and equipped with steam pipes; however, the latter system has generally not been very effective.

In order to reduce the differential foundation movements, the construction sequence and the schedule for backfilling the crest section should be considered.

For a specific installation, special care is required in identifying all of the conditions and loading combinations that can occur. Some examples of conditions and loading combinations that may be applicable in the design of a typical crest section are described below. The load symbols are defined in Section 4.0, and load factors are discussed in Section 9.0.

Construction Conditions

- $D+E+V$ (surcharge)+ H_C+U_C (construction)

Usual Conditions

- $D+E+H_{FSL}+U_{OFSL}+I_S$
- $D+E+H_{SDF}+U_{OSDF}$

Unusual Conditions

- $D+E+H_{FSL}+U_{PFSL}+I_S$
- $D+E+H_{FSL}+U_{OFSL}+Q$

- $D+E+U_{\text{OFSL}}+H_{\text{DFSL}}$ (rapid drawdown)
- $D+E+H_{\text{FSL}}+U_{\text{OFSL}}+V$ (surcharge)

Extreme Conditions

- $D+E+H_{\text{IDF}}+U_{\text{PIDF}}$
- $D+E+H_{\text{FSL}}+U_{\text{OFSL}}+Q_{\text{MDE}}$

12.5.3 Vehicle Access and Operating Decks

Above each gate bay, an operating deck is typically provided for mounting the hoists and related equipment for the control gate and stop log systems.

In most cases, a concrete operating deck is preferred over one fabricated from steel since the former generally provides a better working surface, requires less maintenance, and is aesthetically compatible with the rest of the structure. Either a cast-in-place or precast concrete deck can be incorporated, although the cast-in-place option allows the electrical and control conduits to be readily embedded and protected within the concrete rather than being installed on exposed cable trays or on the underside of the precast concrete elements. In no case should the conduits be mounted directly on the topside of the deck. In addition, equipment base plates should be set on concrete housekeeping pads at least 100 mm high. All service conduits should also come up within the housekeeping pad.

Generally for major spillways equipped with large gate and stop log systems, related mechanical, electrical, and controls equipment are housed within an enclosure to protect them from the elements and to improve working conditions for carrying out operations and maintenance activities. In designing such enclosures, the potential for birds to construct nests and become a maintenance problem should be considered.

Depending on project specific requirements, vehicle access across the crest section can either consist of a combined access bridge and operating deck (e.g. as provided at the Oldman and Dickson Dam spillways) or a separate access bridge deck (e.g. as provided at the St. Mary Dam spillway). As noted in Section 4.9, the bridge deck design should consider not only anticipated highway vehicle loads, but also crane loads that may occur during installation or future maintenance of the gates, stop logs or other facilities.

The arrangement and configuration of the decks should be compatible with that of the crest section, and should be capable of accommodating the differential foundation movements that may occur across the crest section.

12.6 Chute Section

12.6.1 General

The chute section extends from the crest section to the terminal structure, and consists of sidewalls, slab, and cross walls.

Depending on topography, foundation conditions, and extent of excavation required, the slope of the chute may vary along its length. For longer spillways, the chute width may also be varied to provide satisfactory hydraulic performance with a more economical configuration, considering the above and other factors.

The cross section of the chute section will generally consist of one of the following types:

- Type 1: The sidewalls and chute slab are designed as a monolith U-shaped frame. This cross section is typically used in narrow chutes (i.e. spans of less than 20 m between the side walls).
- Type 2: The sidewalls are designed as semi-gravity or cantilever retaining walls supported by footings. The chute slab extends between the footings and the joint between the footing and the slab is designed to transmit shear (so that no differential movement occurs at the joint).
- Type 3: The sidewalls are designed as semi-gravity or cantilever retaining walls complete with footings. The chute slab extends between the footings and is independent of the footing.

In the longitudinal direction, the chute section should consist of either a jointed slab or continuous slab configuration as described in Sections 12.6.3 and 12.6.4, respectively. Regardless of which configuration is used, transverse expansion joints should be provided at strategic locations along the chute to accommodate possible differential movements between the spillway structure components, and slope changes. Such locations include:

- The interface between the crest section and the chute section.
- Upstream of, or at the start of, the vertical curve used to transition between different chute slopes.
- The interface between the chute section and the terminal structure. For continuous slab chutes where large differential movements are anticipated at the interface with the terminal structure, the use of a closure section as discussed in Section 12.6.4 may be considered.

In addition to the locations identified above, expansion joints may also be required where ongoing differential foundation movements are expected to occur between foundation layers that are

intercepted by the chute.

12.6.2 Sidewalls

The height of the chute sidewalls will ordinarily be governed by the depth of flow that will occur during the IDF event, plus a freeboard allowance to account for waves, air bulking, splash and spray. Relevant information for estimating freeboard is provided in Smith (1995), Zipparo et al. (1993), Slopek et al. (1989), and USBR (1987). Where physical model testing has been undertaken to determine the water surface profile and wave formation, freeboard will be required to account for the accuracy of the model, scale effects, bulking, spray and splash.

The height of backfill behind the sidewalls is normally kept to the minimum level needed to provide adequate factors of safety against sliding under spillway operating conditions. This offers the following advantages: reduces the lateral earth pressure and vertical earth load; lowers the quantity of backfill required; and the sidewall projection above the fill serves as a safety barrier. In some instances, pervious materials as described in Section 12.4.2 can be placed adjacent the walls to prevent the build-up of hydrostatic pressures and development of loads due to frost action.

The sidewall is generally designed as either a cantilever or semi-gravity retaining wall depending on its height. Vertical contraction joints are normally provided at regular intervals to reduce cracking and facilitate construction.

For a particular installation, special care is required in identifying all of the conditions and loading combinations that can occur. Some examples of conditions and loading combinations that may apply in the design of the chute sidewalls are provided below. The load symbols are defined in Section 4.0, and load factors are discussed in Section 9.0.

Construction Condition

- $D+E+V$ (surcharge)

Usual Condition

- $D+E$
- $D+E+H_{SDF}+U_{OSDF}$

Unusual Condition

- $D+E+Q$

Extreme Condition

- $D+E+Q_{MDE}$
- $D+E+H_{IDF}+U_{OIDF}$

12.6.3 Jointed Slab

Jointed slab refers to the use of transverse contraction joints across the entire width of the slab at regular intervals along the chute, and expansion joints at strategic locations as discussed in Section 12.6.1. The slab joints are normally set in line with those in the sidewalls.

The jointed slab configuration results in an articulated chute that will have fewer cracks, more flexibility to accommodate differential movements, and where cross walls are used, added resistance to possible downhill creep movements. Disadvantages are related to the added construction complexity and costs caused by the additional joints and cross walls, the need to limit abrupt offsets from occurring at the joints, and the costs and risks associated with keeping the joints watertight.

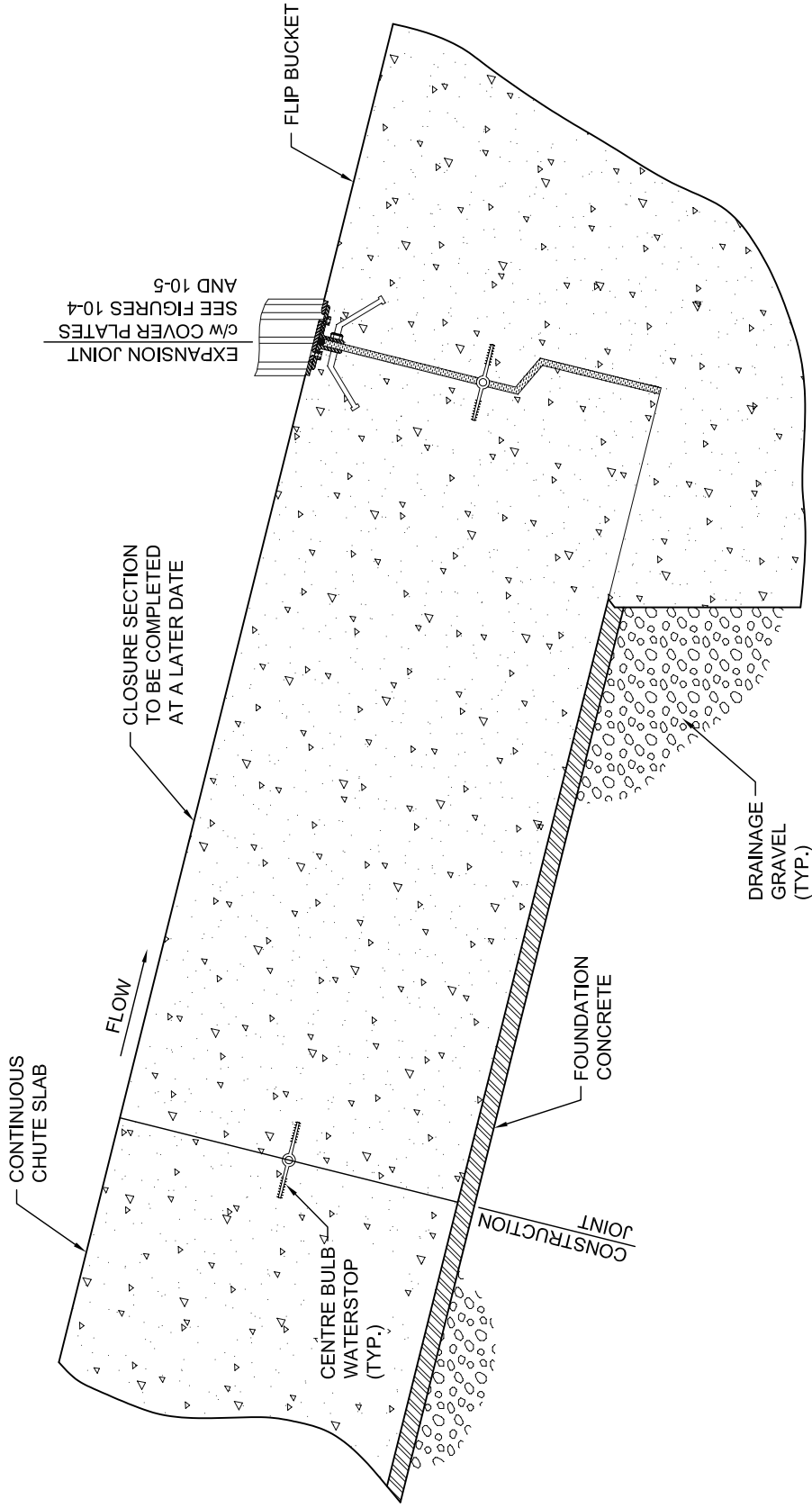
12.6.4 Continuous Slab

The continuous slab design approach generally consists of providing a chute slab that has a minimal number of expansion joints. With this approach, expansion joints may only be required at strategic locations as discussed in Section 12.6.1. Where large differential movements are anticipated at the interface between the chute and the terminal structure, the use of a closure section may be considered. An example of such a closure section is shown on Figure 12.6.

Depending on the profile and slope of the chute, the use of anchor blocks to reduce the potential for downhill creep, caused by contraction and expansion due to temperature change, may be considered. Downhill creep may result in the opening of upstream expansion joints, and closing of downstream expansion joints. Joint closing can lead to spalling of concrete and instability of the structure components. In general, the anchor force required to resist downhill creep is equal to the downslope weight component of the slab plus the friction force at the base of the slab which arises due to thermal contraction of the slab (Smith 1995). This resistance force may be obtained by keying the anchor block into the insitu foundation. Further resistance at the anchor block can be obtained by installing soil or rock anchors.

Sufficient reinforcement is required to limit the width and spacing of cracks that will form in the slab. The amount of reinforcement should normally be such that the elastic limit of the reinforcement would not be reached under forces required to cause tensile cracking of the concrete. However, the minimum reinforcement to gross concrete area ratio should ordinarily not be less than 0.005. For a steep continuous slab section where an uphill anchor block is used to provide resistance against downhill creep, more reinforcement may be required to resist the additional longitudinal tensile stresses that may result.

In addition to expansion joints, longitudinal and transverse construction joints are normally incorporated to divide the slab into appropriately sized blocks to permit the use of conventional concrete paving equipment, and to provide convenient stopping points during construction.



NOTES:

1. CLOSURE SECTION EXTENDS ACROSS FULL CROSS-SECTION OF THE CHUTE (i.e. SLAB AND WALLS).
2. CHUTE REINFORCING STEEL EXTENDS DOWNSTREAM BEYOND CONSTRUCTION JOINT AS REQUIRED TO FACILITATE SPLICING WITH CLOSURE SECTION REINFORCEMENT.

12.6.5 Cross Walls

Cross walls are used as a seepage cutoff between specific transverse drains, and to provide increased resistance to possible downhill creep movements, particularly where the chute slab is jointed.

Typically, the cross wall is attached to the base of the downstream slab and extends upstream to form a ledge that supports the upstream slab. The arrangement as shown on Figure 12-4, with or without the key, will prevent the slab on the downstream side of the joint from projecting into the flow.

Resistance to downhill creep is obtained by keying the cross wall into the insitu foundation. Additional resistance may be provided through the use of soil or rock anchors.

12.7 Terminal Structure

Terminal structures are used to safely deliver the spillway discharge into the outlet channel. Typical types used in chute spillways include a hydraulic jump stilling basin or a flip bucket and plunge pool combination.

Downstream of the terminal structure, riprap is ordinarily provided to protect the outlet channel from erosion.

12.7.1 Hydraulic Jump Stilling Basin

In general, a hydraulic jump stilling basin is used where the energy of the flow must be dissipated prior to returning the discharge into the downstream channel.

Typically, a hydraulic jump stilling basin designed with sidewalls that terminate at the end of the basin is used. However, wingwalls may be provided at the downstream end of the basin in order to avoid having to lengthen the basin in order to accommodate the earthworks needed to transition between the structure and the outlet channel. If downstream wingwalls are proposed, adequate means to drain water, due to splash or runoff, from the adjacent backfilled areas are required. The sequent or tailwater depth for the IDF event plus freeboard will generally govern the height of the basin sidewalls. Freeboard is normally provided to exclude tailwater from entering the jump area and affecting the jump action, and to prevent overtopping due to surge, splash, spray and wave actions. In establishing the height of the sidewalls near the chute to basin interface area, the position of the hydraulic jump under various discharge conditions should be considered. Relevant information for estimating freeboard for the basin sidewalls is provided in Smith (1995) and USBR (1987). Where physical model testing has been employed to determine the water surface profile and wave formation, increments of freeboard will also be required to account for the accuracy of the model, scale effects, bulking, spray and splash.

With the formation of the hydraulic jump, the stilling basin can potentially be subjected to uplift

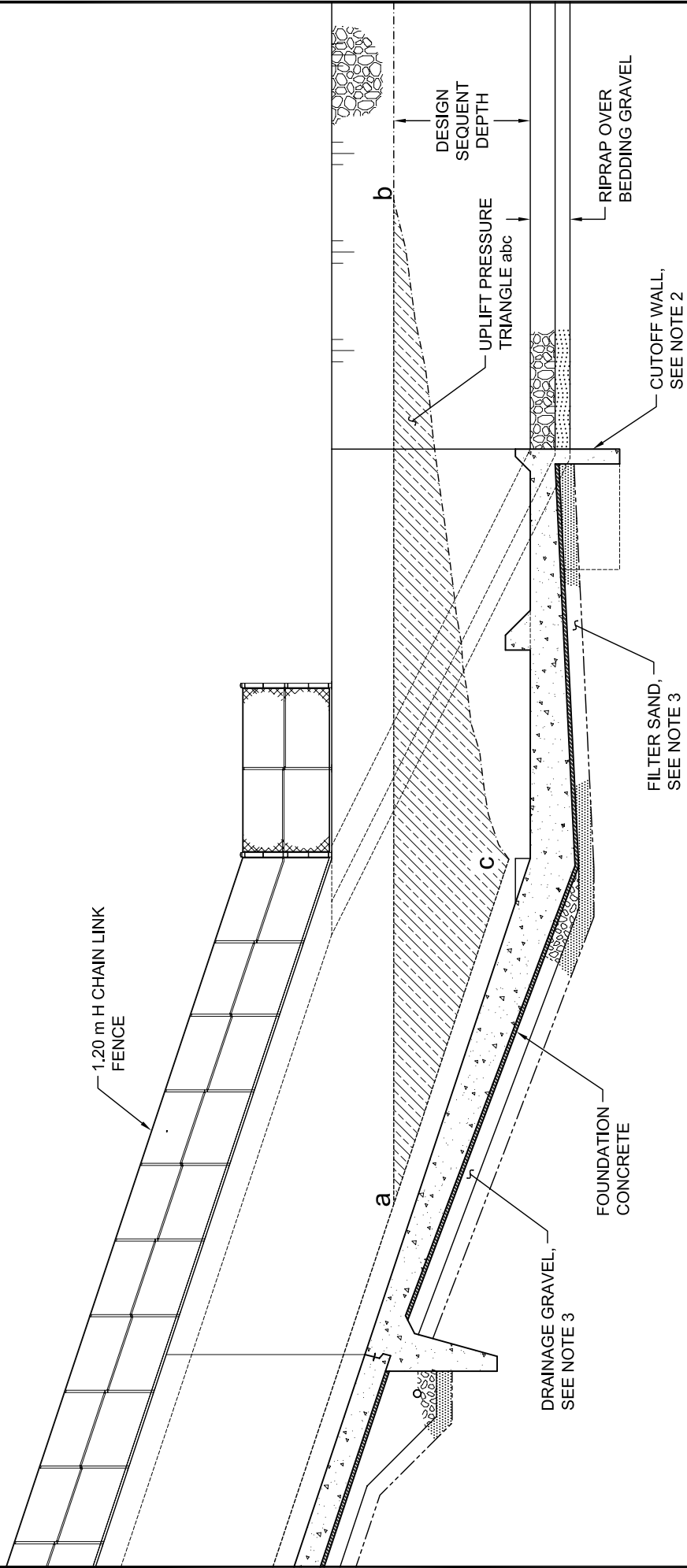
pressures represented by the area contained between the tailwater level and the flow profile, as schematically shown on Figure 12-7, and the sidewalls to a dynamic load as noted in USACE EM 1110-2-1603 (1990). An underslab drainage system, as described in Section 12.4.3, may help to relieve and thereby reduce the uplift pressure and construction costs. However, the position of the drainpipes (i.e. beneath the basin slab and the bed of the outlet channel) inherently means that they are located below the groundwater or outlet channel water level. Therefore, the drainpipes are prone to plugging due to ice formation (i.e. become frozen) or with sediment, and are generally difficult to inspect and maintain. Consequently, it is preferred that, whenever viable and cost effective, the stilling basin be designed to resist the uplift pressures rather than installing drainpipes as discussed below.

For the design of a hydraulic jump stilling basin, it is preferred that, whenever viable and cost effective, the following approaches, listed in order of preference, be implemented.

- .1 Hydraulically size the stilling basin so that it results in a narrow basin (Type 1 cross section as described in Section 12.6.1) that can be designed as a monolith U-shaped frame. With this configuration, the weight required to resist the entire uplift pressure can be obtained by extending the slab beyond the sidewalls in order to mobilize the buoyant weight of the backfill, and/or by increasing the slab thickness.
- .2 Where a wide stilling basin is needed, the entire uplift pressure can be resisted by increasing the slab thickness and/or by providing rock or soil anchors or piles. However, the effectiveness, benefits and cost of providing an underslab drainage system, as noted in the third approach outlined below, should be evaluated.
- .3 Where a wide stilling basin is needed, an underslab drainage system with drainpipes can be incorporated to reduce the uplift pressure. The reduced uplift pressure can be resisted by increasing the slab thickness and/or by providing rock or soil anchors or piles. The long-term performance (i.e. drainage efficiency) of the drains must be closely examined, and should consider factors such as access for inspection and maintenance and provisions for monitoring their performance (i.e. instruments). In addition, the effects of plugged drains on the structure performance should be reviewed.

With approaches 1 and 2 above where drainpipes are not required, measures to exclude tailwater from the underside of the basin slab, such as installing impervious fill zones and providing extensive side and downstream cutoff walls, are not needed. Instead, granular backfill materials including a vertical drainage blanket as discussed in Section 12.3.2 are used adjacent to the walls, and the cutoff walls actually act to prevent scour rather than seepage. Drainage materials are still provided beneath the slab to intercept and drain seepage water out toward the granular backfill materials adjacent the walls. An example of these measures is shown on Figure 17-9.

With approach 3 above, careful attention to backfill requirements and the extent of cutoff walls is required to ensure that tailwater is prevented from entering and surcharging the underslab drainage system (i.e. becomes a pump). In some cases, the side cutoff walls should be extended up the



NOTES:

1. THE STILLING BASIN SHOWN IS DESIGNED TO RESIST THE FULL UPLIFT PRESSURE DUE TO THE HYDRAULIC JUMP (i.e. DOES NOT RELY ON DRAINS).
2. EXTEND THE CUTOFF WALL BACK SOME LENGTH ALONG THE HEEL OF THE WALL FOOTING WHERE TOE EROSION AND POTENTIAL UNDERCUTTING BENEATH THE SLAB IS A CONCERN.
3. EXTEND FILTER SAND/DRAINAGE GRAVEL OUT PAST SLAB AND PROPERLY TIE-IN WITH DRAINAGE MATERIALS ADJACENT TO THE WALLS AND THE BEDDING GRAVEL.

chute slope to the appropriate design tailwater level.

Also, if the foundation beneath the basin is susceptible to frost action, appropriate measures will have to be provided. This is particularly important where anchors or piles are proposed.

For a particular installation, special care is required in identifying all of the conditions and loading combinations that can occur. Some examples of conditions and loading combinations that may apply in the design of a typical stilling basin designed using approach 1 are provided below. The load symbols are defined in Section 4.0, and load factors are discussed in Section 9.0.

Construction Condition

- $D+E+V$ (surcharge)+ H_C+U_C

Usual Condition

- $D+E$
- $D+E+H_{SDF}+U_{OSDF}$

Unusual Condition

- $D+E+Q$
- $D+E+H_{SDF}+U_{PSDF}$ (basin designed to resist full uplift pressure)

Extreme Condition

- $D+E+Q_{MDE}$
- $D+E+H_{IDF}+U_{PIDF}$ (basin designed to resist full uplift pressure)

Where basin blocks have been provided within the stilling basin, the blocks are normally designed to resist the dynamic force due to the jet impinging against their upstream faces as discussed in Section 4.7.

Riprap is normally provided to protect the outlet channel downstream of the stilling basin. Factors that should be considered in determining the extent and sizes of riprap required downstream of a stilling basin include the design condition under consideration, flow velocities and flow depths, wave action, outlet channel parameters including structure/channel transitions, erodibility of the material, slope of banks and channel alignment, and accessibility for conducting repairs.

In general, larger sized riprap is placed on the channel bed and side slopes immediately adjacent the stilling basin because the flow leaving the basin is turbulent. Information for sizing riprap downstream of a stilling basin is available in USACE Hydraulic Design Criteria Sheet 712-1 (1970), USBR Engineering Monograph No. 25 (1978), USACE EM 1110-2-1603 (1990), and Smith (1995). The larger size riprap should ordinarily extend for a minimum distance of $6D_2$, where D_2 is the

sequent depth, downstream of the stilling basin. Beyond this point, smaller sized riprap may be required on the channel side slopes primarily to resist the effects of wave action. In cases where a curve exists just downstream of the basin, the riprap should normally extend far enough to protect the outside bank of the channel. Similarly, where a transition is required to accommodate a change in the section between the structure and the channel, this transition is preferably made within the riprap zone.

Guidelines for establishing the gradation and layer thickness for the riprap and bedding are provided in Section 11.1.

12.7.2 Flip Bucket and Plunge Pool

The flip bucket and plunge pool combination is generally the most economical design for energy dissipation; however, its use is dependent on having favourable geologic and geotechnical conditions (i.e. competent rock).

The bucket flips the flow away from the spillway, the dam, and any other structures, and into the receiving channel. Energy dissipation occurs primarily in the plunge pool. The maximum scour depth within the plunge pool and the lateral extent will depend on the unit discharge, head drop, tailwater depth, and mean particle size of the bed material (Mason, 1993), and should ordinarily be confirmed by hydraulic model testing.

During low spillway flows where the flip does not form, water will cascade over the lip of the flip bucket. Consequently, depending on the geologic conditions, a concrete apron may be required to protect the downstream toe of the flip bucket from scour.

Subject to costs, it is preferred that the flip bucket be designed as a concrete gravity structure rather than one that relies on soil or rock anchors for stability. Stability analyses should consider the effects of the plunge pool that will be formed downstream of the flip bucket.

For a specific installation, special care is required in identifying all of the conditions and loading combinations that can occur. Some examples of conditions and loading combinations that may apply in the design of a typical flip bucket are provided below. The load symbols are defined in Section 4.0, and load factors are discussed in Section 9.0.

Construction Condition

- $D+E+V$ (surcharge)+ H_C+U_C

Usual Condition

- $D+E+H_{FSL}+U_{OFSL}$
- $D+E+H_{SDF}+U_{OSDF}$ +Hydrodynamic Load

Unusual Condition

- $D+E+H_{FSL}+U_{OFSL}+Q$
- $D+E+U_{OSDF}+H_{DSDF}$ (rapid drawdown)

Extreme Condition

- $D+E+H_{IDF}+U_{OIDF}+\text{Hydrodynamic Load}$
- $D+E+H_{FSL}+U_{OFSL}+Q_{MDE}$
- $D+E+U_{OIDF}+H_{DIDF}$ (rapid drawdown)

Also, if the structure foundation is susceptible to frost action, appropriate measures will have to be provided. This is particularly important where rock or soil anchors are proposed instead of structure weight. Where post-tensioned anchors are proposed, the structure design should preferably include provisions for accessing the stressing ends of the anchors (e.g. via a gallery) so that the anchor loads can be checked and re-tensioned, if required.

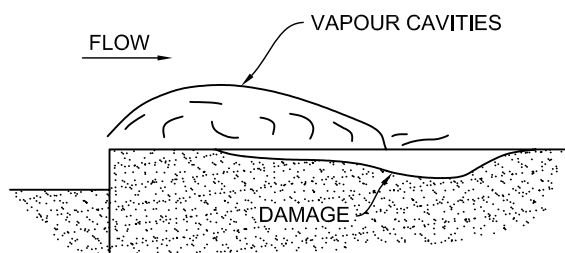
The design of the flip bucket sidewalls should consider the hydrodynamic pressures that will occur during spillway operation as described in Section 4.7.

Unless suitable rock conditions exist, riprap may be required along the banks adjacent to the plunge pool, particularly at the flip bucket and apron, and at any other structure locations. Factors that need to be considered in determining the extent and sizes of riprap required include the design condition under consideration; tailwater depths; extent of plunge pool scour; velocities; wave action; outlet channel parameters including slope of banks, and channel alignment; and accessibility for conducting repairs. Guidelines for establishing the gradation and layer thickness for the riprap and bedding are provided in Section 11.1.

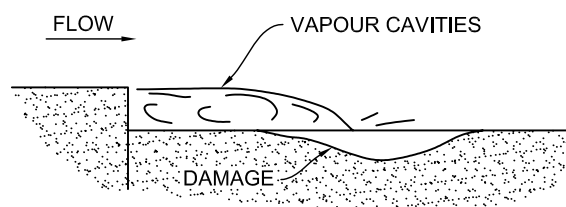
12.8 Cavitation

Cavitation is the formation of bubbles or voids that are filled with water vapour. The voids are formed when the local pressure drops sufficiently to permit water to vaporize. In water control structures, such pressure drops can occur when high velocity water flows past surface irregularities such as an abrupt offset into or away from the flow, an abrupt curvature or slope away from the flow, a void or transverse groove, a protruding joint, or a roughened surface, as shown on Figure 12-8. Cavitation damage is caused by the collapse or implosion of the voids that result in extremely high pressures.

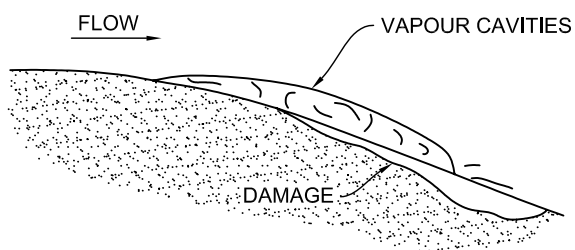
As a rule of thumb, cavitation should be investigated whenever flow velocities exceed 10.7 m/s as noted in USACE EM 1110-2-1603 (1990). However, cavitation is usually not a problem until considerably higher velocities occur. Significant damage has been observed by the USACE on spillway structures where the flow velocity was greater than 30 m/s.



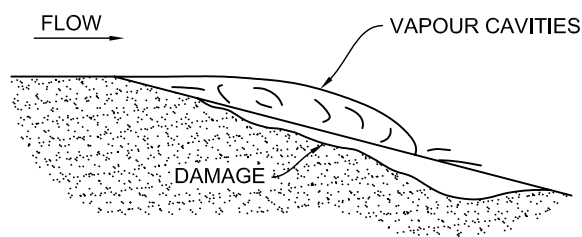
OFFSET INTO FLOW



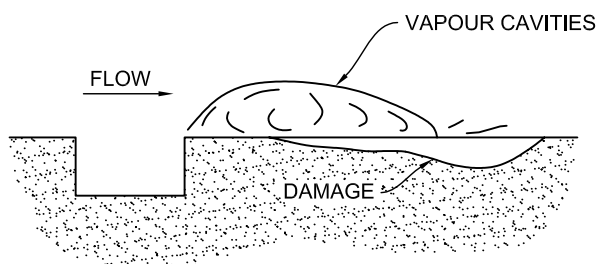
OFFSET AWAY FROM FLOW



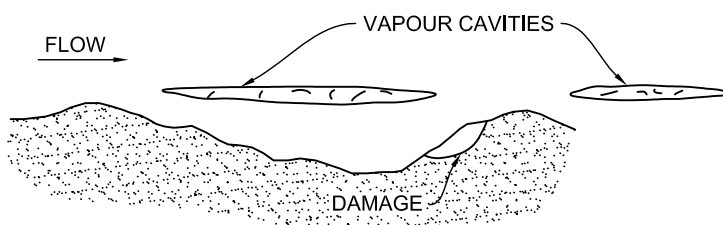
ABRUPT CURVATURE AWAY FROM FLOW



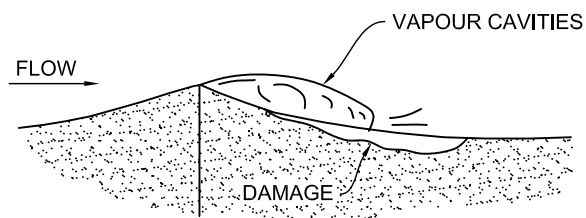
ABRUPT SLOPE AWAY FROM FLOW



VOID OR TRANSVERSE GROOVE



ROUGHENED SURFACE



PROTRUDING JOINT

SOURCE: CAVITATION IN CHUTES AND SPILLWAYS, ENGINEERING MONOGRAPH No. 42, USBR, 1990.

ALBERTA TRANSPORTATION
CIVIL PROJECTS BRANCH

ALBERTA ENVIRONMENT
WATER MANAGEMENT OPERATIONS

WATER CONTROL STRUCTURES - SELECTED DESIGN GUIDELINES

**SURFACE IRREGULARITIES IN
HYDRAULIC STRUCTURES**

DATE: November 2004

CAD FILE: 99008A12-8.dwg

FIGURE No.:

12-8

Flow surfaces should be designed and constructed such that they are free of surface irregularities or abrupt changes in profile that can cause cavitation. Measures that can be employed to reduce surface irregularities or abrupt profile changes include specifying appropriate flow surface tolerances, using proper curves to accommodate changes in the vertical profile, providing a bevel on the downstream corner of slots (e.g. at gates or stop logs), and filling large air pockets (bug holes) on the formed concrete surface. Flow surface tolerances are provided in USACE EM 1110-2-1603 (1990).

At locations where the flow surface tolerances needed to avoid cavitation cannot be achieved, measures for reducing the occurrence of cavitation and for increasing the resistance to cavitation should be considered. These measures may include:

- Providing aerators, deflectors, grooves, offsets or a combination of these, to lift the flow from the surface so that air can be entrained underneath the flow surface.
- Using polymerized concrete, metal plates or other appropriate materials. However, using this approach only increases the resistance for a period of time.