4.0 LOADS

4.1 General

The loads and load combinations described in this Section are primarily those related to the stability analysis and structural design of water control structures or components thereof.

Loads and load combinations for mechanical equipment such as gates and operators are discussed in Section 19.0.

4.2 Dead Loads

Dead loads (D) consist of permanent structure and equipment loads.

Typical unit weights are 23.5 kN/m^3 for normal density concrete and 77 kN/m^3 for steel.

4.3 Earth Loads

Earth loads (E) include vertical and horizontal loading due to earth backfill or silt deposition.

Usually, unit weights of the various compacted earth materials that will be incorporated at a particular installation will be determined as part of the geotechnical program. However, the following typical saturated unit weights can generally be assumed:

- Impervious Fill 21 kN/m^3
- Granular Fill 21 kN/m^3
- Rockfill 22 kN/m^3

In general, the design of water control structures normally incorporates conservative criteria that tend to produce relatively stiff and rigid wall components that may allow lateral earth pressures greater than active pressures to develop. Under these conditions, the at-rest lateral earth pressure can be assumed to act on the driving side (side on which the horizontal force causes instability).

Usually, lateral earth pressure coefficients for a particular installation will be confirmed as part of the geotechnical program. In the absence of such a program, the following at-rest lateral earth pressure coefficients can generally be conservatively assumed for normal backfill situations behind walls:

- 1.0 for impervious fill consisting of low to medium plasticity clay placed against a restrained wall or where no wall movement can be tolerated.

- 0.6 to 0.8 for impervious fill consisting of low to medium plasticity clay placed against an unrestrained wall or where some wall movement can be tolerated.

- 0.5 for a granular backfill zone that extends for an area equivalent to the active earth pressure wedge.

The above coefficients require that appropriate restrictions on the size and operation of compaction equipment immediately adjacent the walls are specified as outlined in Section 24.2. Similar restrictions are also important where backfill is being placed within a relatively confined area between a rigid structure and a rock face.

For impervious fill comprised of high plasticity clay, the potential to develop higher lateral at-rest earth pressure coefficients than those noted above should be examined.

Where permanent backfill (material that will not be displaced, eroded or excavated during the life of the structure) is placed on the resisting side of the wall (side on which the horizontal force maintains stability), it may be overly conservative to assume at-rest lateral earth pressures for the design. While permissible wall movements are often too small to produce full passive pressures, modest movements can produce partial passive pressures, for example, which can often be relied upon. Judgment is required in determining acceptable passive pressures that will be appropriate for a particular installation. One approach suggested in USACE EM 1110-2-2502 (1989) for the design of retaining and flood walls where the at-rest pressure on the driving side of the wall exceeds the at-rest pressure on the resisting side plus the base friction force, is to assume that the additional resisting force is obtained by mobilizing a portion of the passive pressure up to a maximum of one-half of the available passive force.

In cases where silt may be naturally deposited against a structure, the resulting loads should be considered. In general, where actual data is not available, the horizontal component of the silt load (S) may be assumed as being equivalent to that of a fluid weighing 13 kN/m^3, and the vertical component based on a weight of 19 kN/m^3, USBR (1987).

4.4 Hydrostatic Loads

4.4.1 General

In general, the hydrostatic load referred to herein comprises the static load caused by water bearing against a structure or component thereof. The hydrostatic load may be due to water impounded against a structure, or contained or flowing through a structure, or due to groundwater acting against a structure. The magnitude of the hydrostatic load may also vary due to fluctuating water levels caused by drawdown or flood conditions. Careful attention is required in determining the hydrostatic load for a particular loading condition that may act on a structure or component thereof.

Some examples of loading conditions and associated hydrostatic loads that may apply for a typical service spillway structure are provided below. In general, service spillways are ordinarily designed to pass floods up to the chosen SDF with a limited amount of surcharge in the reservoir above its full supply level (FSL). The service spillway, possibly in conjunction with an auxiliary spillway, is usually designed to pass more extreme floods up to and including the IDF without overtopping the dam. Examples of hydrostatic loading conditions that may apply for a typical chute-type service spillway structure with a controlled (gated) crest section are as follows:

- H_c - maximum phreatic level during construction.
- H_{FSL} - FSL with the structure not in operation (gates closed), and with the most critical concurrent phreatic or tailwater level.
- H_{SDF} - maximum flood headwater level or depth of flow due to the SDF with the structure in operation (gates open), and with the most critical concurrent phreatic or tailwater level.
- H_{IDF} - maximum flood headwater level or depth of flow due to the IDF with the structure in operation (gates open), and with the most critical concurrent phreatic or tailwater level.
- H_{DFSL} - maximum normal rapid drawdown conditions wherein the phreatic level in the surrounding fill remains at the same level as that for condition H_{FSL} described above, while the headwater level is lowered to its minimum drawdown level.
- H_{DSDF} - maximum flood rapid drawdown conditions wherein the phreatic level in the surrounding fill remains at the same level as that for condition H_{SDF} described above, while the headwater level is lowered to the H_{FSL} level.
- H_{DIDF} - maximum flood rapid drawdown conditions wherein the phreatic level in the surrounding fill remains at the same level as that for condition H_{IDF} described above, while the headwater level is lowered to the H_{FSL} level.

Hydrodynamic loads resulting from wave action, moving water, and an earthquake are discussed in Section 4.7.

4.4.2 Hydrostatic Uplift Loads

In general, hydrostatic uplift loads consist of the upward water pressure acting against the base of the structure or component thereof.

The magnitude of the uplift load may vary due to fluctuating water levels caused by drawdown or flood conditions, and by the effectiveness of any drains that may have been provided. As a result, careful attention is required in determining the hydrostatic uplift load for the various loading conditions that may act on a structure or component thereof.

Some examples of loading conditions and associated uplift loads that may apply for a typical chute-type service spillway structure with a controlled (gated) crest section are provided below. Hydrostatic loading conditions, H_{FSL}, H_{SDF}, and H_{IDF}, are described in Section 4.4.1.

- U_c - uplift load due to the maximum phreatic level during construction

- U_{OFSL} - uplift load due to H_{FSL} combined with the most critical concurrent phreatic or tailwater level with the drains operational.

- U_{PFSL} - uplift load due to the maximum normal headwater level H_{FSL} combined with the most critical concurrent phreatic or tailwater level with the drains plugged.

- U_{OSDF} - uplift load due to the maximum flood headwater level H_{SDF} and corresponding phreatic or tailwater level with the drains operational.

- U_{PSDF} - uplift load due to the maximum flood headwater level H_{SDF} and corresponding phreatic or tailwater level with the drains plugged or where drains are not needed (e.g. stilling basin designed to resist the full uplift load).

- U_{OIDF} - uplift load due to the maximum flood headwater level H_{IDF} and corresponding phreatic or tailwater level with the drains operational.

- U_{PIDF} - uplift load due to the maximum flood headwater level H_{IDF} and corresponding phreatic or tailwater level with the drains plugged or where drains are not needed (e.g. stilling basin designed to resist the full uplift load).

For the chute section, it is usually assumed that a plugged drain condition cannot occur since the underslab drainage system that is normally provided tends to have significantly greater drainage capacity than required as well as a large degree of redundancy against plugging (i.e. many drainpipes).

4.5 Ice Loads

4.5.1 Horizontal Ice Loads

4.5.1.1 Static Ice Loads

Traditional values generally considered suitable for static ice loads (I_s) acting on a unit width of a dam or related structure are as follows: 150 kN/m (10 kips/ft) for concrete dams and structures, 75 kN/m (5 kips/ft) for steel gates, and 30 kN/m (2 kips/ft) for timber stop logs. The ice thickness is normally considered to be 0.6 m thick with the ice load acting at 0.3 m below the water level.

Recent studies on static ice loads for dams, sponsored by the Canadian Electricity Association (CEA) and conducted by Comfort et al. (2003), suggest that much higher ice loads than traditional values can occur when significant, but not excessive, changes in reservoir levels are expected. CEA is in the process of producing a design guide that should be available in the near future.

Provisions for preventing ice formation against gates are discussed in Section 19.1.9.

4.5.1.2 Dynamic Ice Loads

Dynamic ice loads (I_D) occur when a moving ice floe impacts a structure element. The load imposed on the structure depends on the size of the floe, the strength and thickness of the ice, and the geometry of the structure element. Some information on dynamic ice loads can be obtained from CAN/CSA-S6-00 and the Commentary.

Wherever possible, structures, and in particular gates, should be sheltered to prevent moving ice floes from impacting them.

4.5.2 Vertical Ice Loads

Vertical ice loads may occur particularly where the water level initially remains at one level long enough for the ice to freeze to the structure, and then changes. As the water level changes, either the adfreezing bond developed between the ice sheet and the structure or the bending strength of the ice must be exceeded in order for the ice to move. Some information on vertical ice loads can be obtained from CAN/CSA-S6-00 and the Commentary.

4.6 Earthquake Loads

For water control structures at dams, the appropriate Maximum Design Earthquake (Q_{MDE}) can be established based on the consequence of failure of the structure as described in the CDA Dam Safety Guidelines (1999).

In other instances, the minimum earthquake force (Q) is usually determined based on ground acceleration and ground velocity that have a 10% probability of exceedance in 50 years (annual probability of exceedance of 1 in 475). Design values for ground acceleration and velocity for various municipalities are provided in the Alberta Building Code. The Alberta Building Code requirement could exceed the criteria outlined in CDA (1999) for low or very low consequence situations.

In general, water control structures should be constructed on foundations that have no potential for liquefaction. Therefore, pseudostatic methods, wherein the earthquake load is determined as the product of the structure mass and a seismic coefficient, may be used in the design.

During an earthquake, the hydrodynamic pressure distribution that will act on a structure may be estimated using the following Westergaard equation. This hydrodynamic pressure will be in addition to the full hydrostatic pressure.

$$p = 0.875 \left[\gamma_w \, a_c \, (H\,y)^{0.5} \right]$$ USACE EM 1110-2-2702 (2000).

where: p = lateral hydrodynamic pressure at a vertical distance y below the pool surface,
γ_w = unit weight of water,
a_c = maximum base acceleration due to the earthquake (expressed as a fraction of

gravitational acceleration),
H = total depth of water acting on the structure, and
y = vertical distance below the pool surface.

4.7 Hydrodynamic Loads

Hydrodynamic loads discussed in this Section include those resulting from wave action, moving water, and an earthquake.

Wave loads may be determined from the guidelines contained within the USACE Shore Protection Manual (1984).

For moving water, the forces caused by changes in flow direction or flow area, drag, or impact may be determined using established impulse-momentum principles or equations.

For hydraulic jump stilling basins, the sidewalls will be subjected to a dynamic load as a result of the turbulence created by the hydraulic jump as noted in USACE EM 1110-2-1603 (1990). The drag force created by the chute blocks, basin blocks and end sill provided within the stilling basin may be determined using USACE EM 1110-2-1603 (1990), USBR (1987), and Smith (1995).

For flip buckets provided at the end of chutes (rectangular) and tunnels, the maximum dynamic pressure acting on the slab and walls due to the curvilinear flow can be estimated from the equations derived by Balloffet (1961) and Mason (1993). For flip buckets on major spillway structures, pressures should be obtained from hydraulic model testing.

Hydrodynamic pressures due to an earthquake are discussed in Section 4.6.

4.8 Occupancy Loads

Occupancy loads (O) should be in accordance with the requirements of the Alberta Building Code.

4.9 Vehicle Loads

Vehicle loads (V) should be in accordance with CAN/CSA-S6-00. The applicable design truck for a particular facility will generally be established by the Province.

For vehicle access decks located on the crest section of chute spillways, possible crane loads during installation and future maintenance of the gates or other equipment should also be considered.

Where light vehicles or equipment will operate in close proximity to a wall, a uniform lateral load assuming a surcharge load of at least 0.3 m of backfill should be considered. However for heavy vehicles or equipment or for a high wall, a more detailed analysis using the equations and lateral pressure diagrams outlined in USACE EM 1110-2-2502 (1989) may be appropriate.

4.10 Wind and Snow Loads

Wind loads (W) and snow loads (S) for various municipalities should normally be in accordance with the Alberta Building Code.

4.11 Temperature Loads

The effects of temperature variations, temperature-induced loads (T), on the movements and overall behaviour of structure components, and the performance of equipment should be considered.

Design atmospheric temperatures for various municipalities are provided in the Alberta Building Code.

4.12 Load Combinations

Load combinations for water control structures can be more complex than those for other structures such as buildings. This is due, not only to the wider variation in load types, but also to the fact that water control structures may be required to resist loads resulting from various conditions including Construction Conditions, Usual Conditions during normal operation, Unusual Conditions due to infrequent events, and Extreme Conditions due to rare events.

Examples of these load variations include the normal maximum operating water level; maximum flood levels and discharges due to the IDF; uplift pressures with pressure relief drains operating and plugged; and the operating design earthquake and the maximum design earthquake.

As a result, possible load combinations that may occur under the Construction, Usual, Unusual, and Extreme Conditions of loading must be carefully identified and considered in the design of the water control structure or elements thereof.

In general, ice loads are usually not considered simultaneously with flood conditions or earthquake conditions, nor are the IDF and MDE considered simultaneously.

Examples of possible loading combinations for the different conditions that may apply for a specific water control structure or elements thereof are provided in later sections.