Tampereen ammattikorkeakoulu

Geotechnical and Structural Design Considerations for Fixed Crest Weirs

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Construction Engineering
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ABSTRACT

Tampereen ammattikorkeakoulu
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Geotechnical and Structural Design Considerations for Fixed Crest Weirs

Bachelor's thesis 48 pages, appendices 5 pages
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The objective of this thesis was to identify and outline structural and geotechnical design considerations for fixed crest weirs. Regional geotechnical properties and climate impediments, due to the geographic location of Finland, were also considered. This thesis was commissioned by Ramboll Finland Oy.

The methodology of this thesis was conducted through data collection and data analysis. Thus, resulting in a compendium of theoretical aspects, that are necessary for the design and for the integrity of the structure.

The results show that weirs constructed from soils are affected by frost heave and that impact of the accumulation of ice build-up against the structure requires individual evaluation for each design criteria separately. The design considerations for weirs are sufficient stiffness to prevent settlement, the ground must have adequate capacity to withstand vertical actions, and there must be sufficient sliding resistance to withstand horizontal and inclined actions. The function of the weir is subjected to high destabilising actions, due to the retained water and accumulated sediments, and low stabilising actions, due to the distribution of uplift pressures under the foundations. Therefore, there must be adequate resistance for seepage and hydraulic fracture. Pore water pressure dissipation and creep in soft organic subsoils can cause significant deformations of the weir structure.

Analytical calculations with Eurocode 7 compare the design destabilizing pore water pressure to the design stabilizing vertical stress and compares the design destabilizing seepage force to the design stabilizing submerged weight. Due to local variations in the hydraulic gradient caused by inconsistencies in the soil matrix, relatively high limit states in design are recommended for higher integrity of the weir.

Key words: piping, scour, structural loads, weir

TIIVISTELMÄ

Tampereen ammattikorkeakoulu
Rakennus- ja yhdyskuntatekniikka
Infrarakentaminen

VAN-ECKMANN, AARON:
Kiinteäharjaisten pohjapatojen suunnittelussa huomioitavat geotekniset ja rakenteelliset tekijät

Opinnäytetyö 48 sivua, joista liitteet 5 sivua
Huhtikuu 2022

Opinnäytetyön tavoitteena oli tunnistaa ja jäsentää rakenteellisia ja geoteknisiä suunnitteluseikkoja kiinteäharjaisille pohjapadoille. Vaikuttavina tekijöinä otettiin huomioon myös alueelliset geotekniset ominaisuudet ja Suomen maantieteellisestä sijainnista johtuvat ilmastorajoitteet. Opinnäytetyö tehtiin Ramboll Finland Oy:n toimeksiannosta.

Opinnäytetyö toteutettiin tiedonkeruun ja data-analyysin avulla. Tuloksena on kokoelma teoreettisia soveltamissääntöjä, jotka ovat välttämättömiä patosuunnittelun ja -rakenteen eheyden kannalta.

Tulokset osoittavat, että roudan vaikutus pohjapatoihin käytettyihin maatäyttöihin ja pohjapatoa vasten muodostuvan kiinteän jääpeitteen rakenteisiin kertyvän jään vaikutus vaativat yksilöllisen arvioinnin jokaiselle suunnittelukriteerille erikseen. Pohjapatojen suunnittelussa on otettava huomioon riittävä jäykkyys estämään padon painuminen, maan riittävä pystysuuntaisen kuormituksenkesto ja riittävä liukukestävyys vaaka- ja sivuttaisvoimille. Pohjapatoon kohdistuu suuria epävakauttavia voimia, jotka johtuvat padon pidättämästä vedestä ja kerääntyneistä sedimenteistä, sekä alhaisia vakauttavia voimia, jotka johtuvat perustusten alle jakautuvasta hydraulisesta noususta. Edellä mainittujen voimien vuoksi tulee varmistaa pohjapadon riittävä suoja vuotoja ja hydraulista murtumista vastaan. Huokosveden painehäviö ja viruminen aiheuttavat padon rakenteen muodonmuutoksia.

Eurokoodi 7:n mukaisissa analyyttisissä laskelmissa verrataan huokosvedenpaineiden vakavuusehtoja epävakaisiin kokonaisjännityksiin sekä epävakauttavia suotovirtausvoimia vedenalaisiin painoihin. Maalajien matriisien epäjohdonmukaisuuksista johtuvien hydraulisen gradientin vaihteluiden vuoksi suunnittelussa suositellaan suhteellisen korkeita osavarmuuskertoimia padon paremman eheyden vuoksi.

Avainsanat: kuormat, putkieroosio, pohjapato, sisäinen eroosio

CONTENTS

ABBREVIATIONS AND TERMS

Abutment	The walls that flank the edge of a weir and which support the banks on each side of the weir
Afflux	The maximum increase in water level before and after the construction of weir.
Apron	Scour-resistant component or layer placed next to the weir for protection against erosion.
Head (of water)	Total energy of water denoted in metres of water above a datum e.g., weir crest.
Piping	The movement of soil particles due to water percolating through soil.
Nappe	The jet of water passing over the weir crest.
Scour	Erosion resulting from the shear forces associated with flowing water or wave action.

1 INTRODUCTION

The purpose of this thesis was to identify and to comprehend the process of preliminary weir geometry and contributing factors in hydraulic and foundation sizing. The objective was to outline weir design considerations and apparent factors regarding structural loads and geotechnical aspects for foundations for conditions in Finland. This thesis was commissioned for Ramboll Finland Oy – Geotechnical Services and Construction Management.

Geotechnical and climate constraints must be accounted for when designing weir structures in Finland. Riverbed sediments in Finland are predominantly layers of soft and possibly organic silt and clay that make geotechnical design challenging. Moreover, structures are impacted by frost heave during cold spells and by loads due to build-up of ice sheets in aquatic environments.

Most of the weir structure remains unseen throughout its life. The unseen portion is largely inaccessible and therefore, should be engineered to remain durable with little maintenance. Weir foundations differ from conventional building foundations as they are acted upon by destabilising water forces that are relatively large when compared to the foundation weight. Other processes such as scour and piping i.e., hydraulic fracture make the design of the weir foundations a highly interlinked exercise.

The low head weirs that are typically constructed in Europe can be considered as spread foundations and retaining structures subject to special hydraulic actions because they impound water on one side. The design considerations are that (1) load distribution shall prevent unacceptable settlement, considering ground deformability, (2) the ground-weir system has adequate capacity to withstand vertical actions, and (3) there is sufficient sliding resistance to withstand horizontal and inclined actions.

The considerations in question result from the fact that the weir structure experiences abnormally high destabilising actions due to the retained water and accumulated sediments, and abnormally low stabilising actions due to the distribution of uplift pressures under the foundations. Furthermore, it is crucial to check that there is adequate resistance to seepage and hydraulic fracture i.e., piping.

2 WEIR FUNCTIONS AND GENERAL ASPECTS

Typical functions of a weir are water level management, flow measurement, channel stabilisation, environmental enhancement including mitigation and commercial use.

Weirs control the vagaries of a river through water level management, that regulates the supply of water, thus providing a unique stage-discharge relationship for many flow conditions alternating a desired hydraulic performance. Weirs can contribute to land drainage for flood risk management e.g., to divert water to an offline flood storage by dissipating energy at a defined location, thus allowing the management of water levels for stability of riverside structures and advertently helps to control the entry of sediments into the canal. (Kitchen, Gauldie, Patterson, Bentley, Kirby 2016.)

Weirs can have beneficial outcomes regarding environmental enhancement and mitigation through the diversion of water to wetlands, creating and providing for pondage including the provision of sediment continuity along the watercourse, thus aiding in habitat diversity, and improving water quality and ecology.

Where applicable recreation requirements in the vicinity of a weir must be taken into consideration to allow public access opportunities, angling, boating and safe passage. Weirs may also be used to conserve or restore natural or built heritage. Tending to the landscape for riverside attractiveness in order to preserve and better reveal the historic significance of the location, often within a designed landscape or industrial archaeological context. (Kitchen et al. 2016)

Weirs may be further used for impoundment of water for navigation for abstraction or hydropower. Figure 1. implicates the key issues during the design and implementation of a weir and its adjacent surroundings.

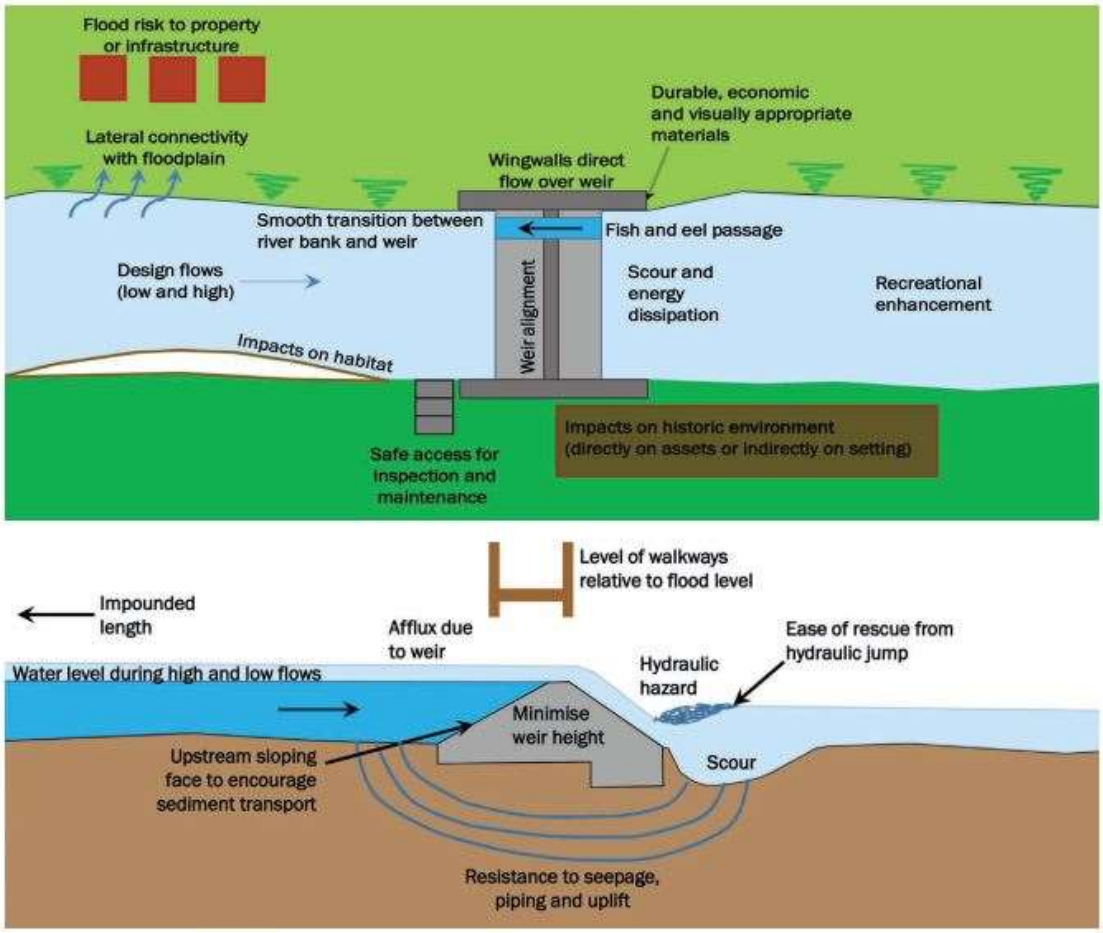

FIGURE 1. Key issues during design and implementation (Kitchen et al. 2016)

Fixed crest weirs have a singular stage discharge relationship that is defined by their geometry. In order to achieve the desired discharge, the crest length of the weir can be increased with the available space in the river and the crest shape can be altered to increase the efficiency of discharge. There are various fixed crest weir arrangement that be used, shown in section 3.2. Figure 2. illustrates the main components of a weir.

Stone and stone-filled gabion cages can be used for small rudimentary weirs and rock ramps, sheet piles are used for simple weir design. However, concrete is usually the primary material selected, due to steel reinforced concrete being versatile, can be shaped into hydraulic efficient forms that are not susceptible to roughening or vegetation growth, and mass concrete is efficient for providing weight for stability. (Kitchen et al. 2016)

Water level regulation can be achieved by using a long weir crest or a labyrinth weir, which allows a long crest to be accommodated into a shorter width of channel. Weir gates allow the regulation of water level and temporary timber weir boards (or 'summer-boards') can improve water level control at different time of the year.

Hydraulic performance can be affected by floating debris accumulation, seasonal vegetation growth, ice accumulation or sedimentation at weir boards or trash screens. The discharge of surplus water can be affected by incompatible combinations of weir length and downstream culvert capacity or downstream river flood (i.e., the weir can pass more flow than can be accepted downstream). (Kitchen et al. 2016)

The design and operation of weirs should consider the safety boaters, and other river and towpath users, as well as those responsible for inspection and maintenance. User safety and access restrictions are particular considerations for weirs located beneath the towpath. Weirs should be resistant to boat and ice impact. Consideration should also be given to the consequences of gate failure in the closed or open position (increase of flood risk). (Kitchen et al. 2016)

Typical issues with weirs include damage and irregular crests, local scour due to hydraulic action or boat movements, scour behind abutments, and maintenance and repair of downstream aprons.

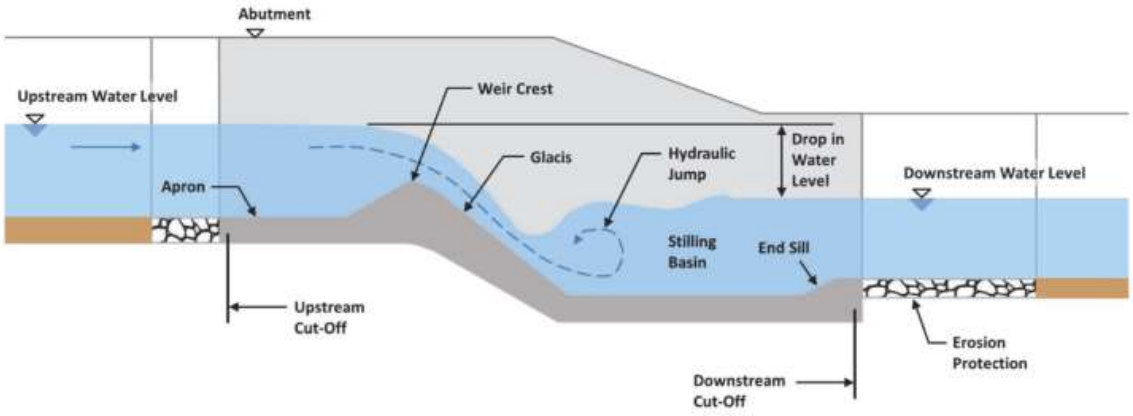

FIGURE 2. Main components of a weir (Kitchen et al. 2016)

The structural integrity of a weir on to maintain a specified condition and to resist imposed loads or actions due to hydrostatic pressure, hydrodynamic forces, and debris impact, throughout its design life, without the need for excessive maintenance by limiting damage during a flood that exceed the design flood. (Kitchen et al. 2016)

3 HYDRAULICS OF WEIR STRUCTURES

3.1 Flow conditions

The shape of the crest has the most influence on the discharge of water.
As the flow over the weir changes, the head (depth of water) over the crest will also change. This results in there being a relationship between the discharge over the weir and the upstream head above the weir crest. It is this principle that allows weirs to be used for discharge measurement. This mathematical link between upstream head and flow remains valid while the downstream water is low enough to have no effect on upstream water level, i.e., while the flow remains 'modular' or free flowing. As flow increases in the river, the downstream water level will naturally increase because the river is carrying extra water. (Kitchen et al. 2016)

In general, modular flow across a weir is described by the weir equation:

$$Q = C \cdot B \cdot H^n \quad (1)$$

Q = flow (m^3/s)
C = discharge coefficient dependent on weir crest shape
B = weir crest length (m)
H = head over weir crest (m)
n = varies with structure (e.g., 3/2 for horizontal weir, 5/2 for v-notch weir)

The value for C is a combination of parameters that includes the gravitational constant, discharge coefficient (dimensionless), and constants based on geometric properties.

Parameters for the equation 1. are illustrated in figure 3.

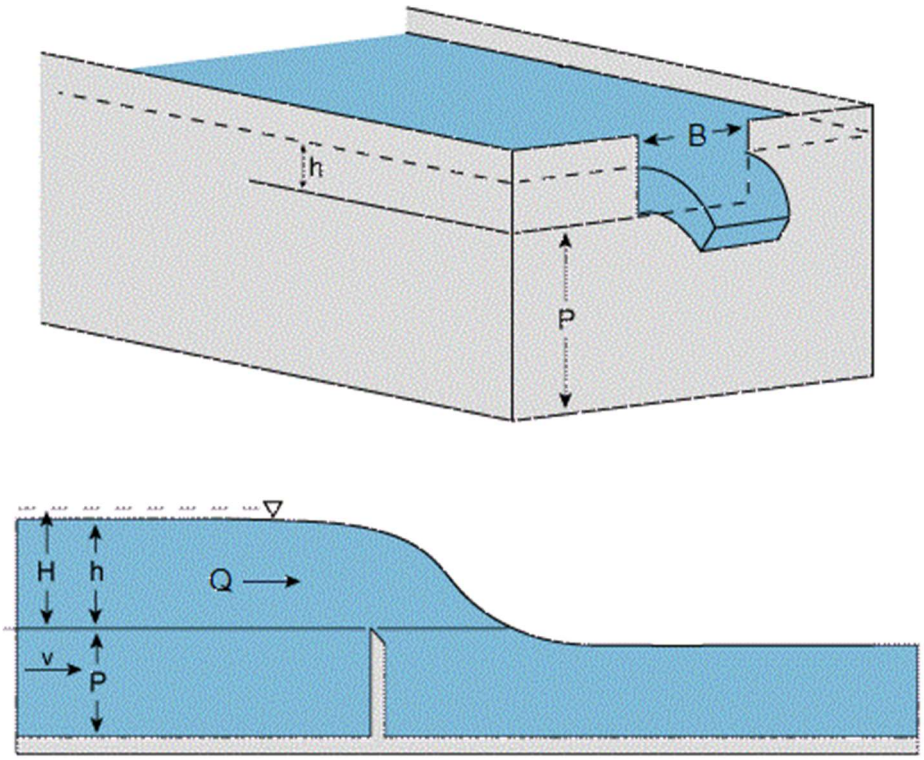

FIGURE 3. Schematic for equation parameters of a weir (adapted from National Weather Service)

In order to achieve the required discharge characteristics, the crest length of the weir can be increased or decreased within the available space in the river, the crest level can be lowered or raised to increase the head driving flow over the crest, or the crest shape can be altered to increase the efficiency of the discharge. (Kitchen et al. 2016)

The hydraulic impact of a weir increases the upstream water level. The water level upstream of the weir is dictated by the head required to drive the flow over the weir. It should be noted that the impact of the weir on the upstream water level is not confined to the immediate vicinity of the weir – there is 'backwater effect' (see figure 4), which extends some way upstream of the weir. (Kitchen et al. 2016)

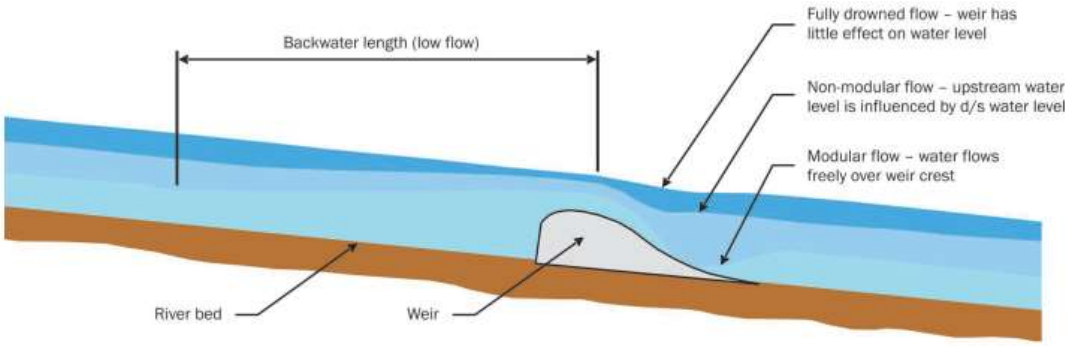

FIGURE 4. Modular, non-modular and drowned flow conditions (Kitchen et al. 2016)

The increase in water level will, for the same flow rate, reduce the average velocity in the upstream reach. This will have an effect of the sediment transporting capacity of the channel. The lower velocities will have knock-on effects in terms of water quality and habitat type. There are downstream issues as well, namely that there is likely to be a localised increase in turbulence and flow velocity immediately downstream of the weir. This has the potential to cause erosion of the riverbed and banks and may result in the creation of a deep pool downstream of the weir, and deposition in the form of a shoal further downstream. (Kitchen et al.2016)

Eventually the water level will increase to a point where water no longer freely discharges over the weir crest, and a situation occurs where a change in downstream water level affects the upstream level. When this occurs, the weir is described as being ‘drowned’, ‘submerged’ or operating under ‘non-modular’ conditions as illustrated in figure 4. Instead of flow being a function of upstream head only, it is now dependent upon both upstream and downstream levels. (Kitchen et al. 2016)

3.2 Weir arrangements

An orthogonal arrangement (shown in figure 5.) spans the river at a right angle to the flow direction. This is the most used weir alignment due to simple approach hydraulics, minimal use of material and ease of design.

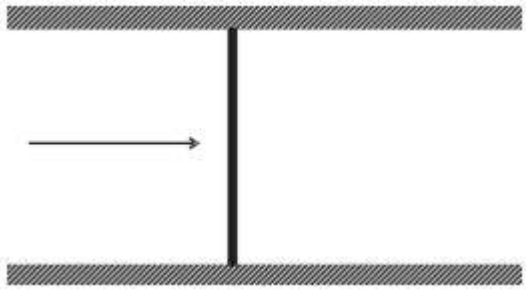

FIGURE 5. Orthogonal arrangement (Kitchen et al. 2016)

A diagonal arrangement (illustrated in figure 6.) spans the channel at an angle to the flow to increase the crest length, thereby increasing the discharge capacity of the weir. Then enhancement to discharge capacity is less pronounced for higher flow, where the weir begins to behave similarly to the equivalent orthogonal weir. Diagonal weirs can create more complex approach and downstream hydraulic conditions and making the stilling basin design challenging.

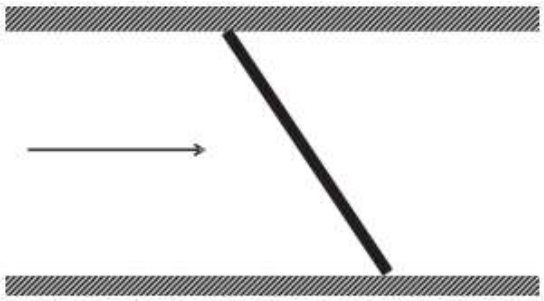

FIGURE 6. Diagonal arrangement (Kitchen et al. 2016)

A labyrinth weir (demonstrated in figure 7.) is a linear weir that is corrugated in plan-view to increase the crest length and discharge capacity of the weir. The enhancement to discharge capacity is less pronounced for higher flows, where the weir begins to behave similarly to the equivalent orthogonal weir.

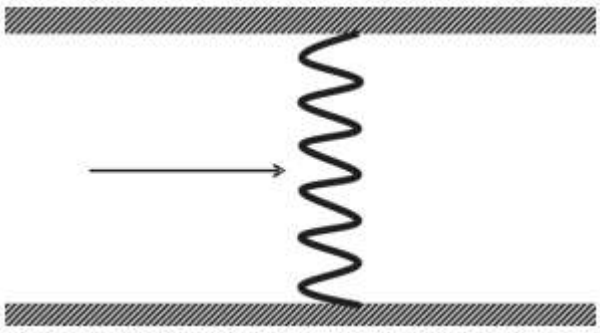

FIGURE 7. Labyrinth arrangement (Kitchen et al. 2016)

The curved alignment (illustrated in figure 8.) of the weir acts to increase the crest length, raising the discharge capacity of the weir. At very high flows, the weir will behave similarly to an orthogonal weir of the same type. These types of weirs often have high visual appeal.

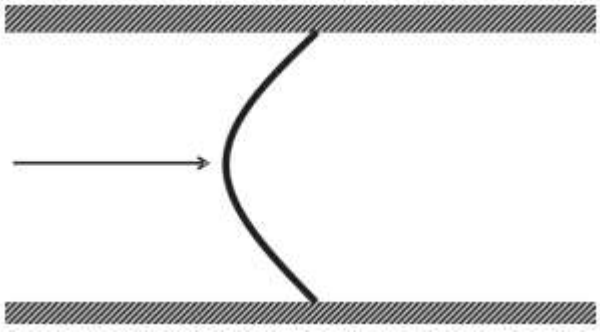

FIGURE 8. Curved arrangement (Kitchen et al. 2016)

Side weirs (depicted in figure 9.) are used widely to divert flows from rivers, canals, and reservoirs. However, their hydraulic behaviour is complex and difficult to predict accurately by simple methods.

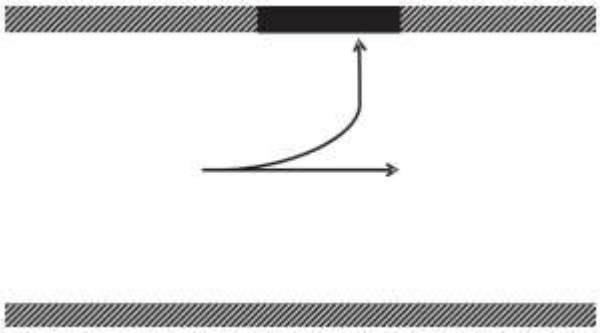

FIGURE 9. Side weir arrangement (Kitchen et al. 2016)

Gated weirs (shown in figure 10.) often have a fixed weir located alongside to limit the daily operation of the gates. Weir complexes comprise one or more different weirs across a watercourse, acting in parallel to control upstream water levels and convey flow downstream.

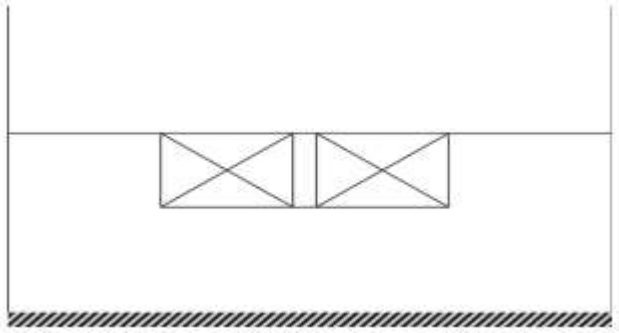

FIGURE 10. Gated weir arrangement (Kitchen et al. 2016)

3.3 Fixed crest weir sections

Broad-crested weir structures (illustrated in figure 11.) can have control sections of a variety of shapes including triangular, parabolic, trapezoidal and circular. The use of a broad-chested weir for flow measurements is dependent on the weir crest hight above the channel bed being sufficient to generate critical depth at a control section on the crest. Critical depth is the depth of flow where energy is at a minimum for a particular discharge. Most weirs in rivers can be approximated to broad-crested weirs although their structure and form vary significantly.

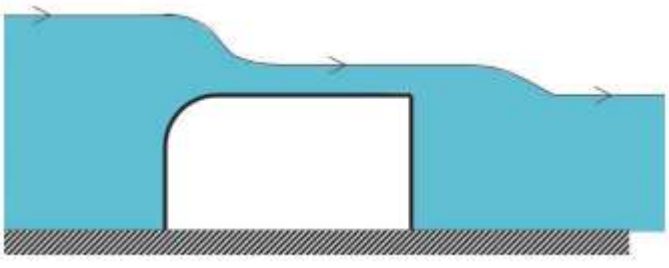

FIGURE 11. Broad-crested weir section (Kitchen et al. 2016)

A sharp-crested weir or thin plate weir (depicted in figure 12.) is an overflow structure where the length of the crest in the direction of the flow is extremely short, typically in the order of two millimetres or less. Sharp crested weirs are often used in laboratories, or as temporary gauging structures in small streams and sediment-free water.

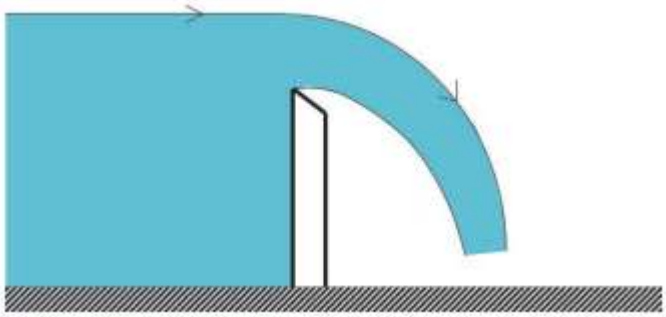

FIGURE 12. Sharp-crested weir section (Kitchen et al. 2016)

Crump weirs (illustrated in figure 13.) are of standard geometry used for gauging flow. These structures have inclined upstream and downstream faces with an intersection at a horizontal crest line across the channel. The standard gradients for the structure are 1:2 on the upstream face and 1:5 on the downface. This less obstructive design allows for greater sediment transportation, easier access for aquatic creatures and minor siltation build up.

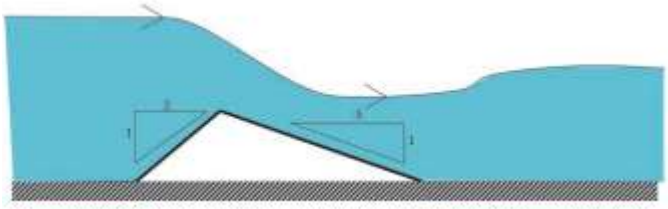

FIGURE 13. Crump weir section (Kitchen et al. 2016)

The shape of the weir for ogee weirs is curved to match the profile of the water surface under the nappe of water flowing over the structure (shown in figure 14.) This is generally recognised as being the ideal weir profile shape for the conveyance of flood water and is commonly used in reservoir spillway design.

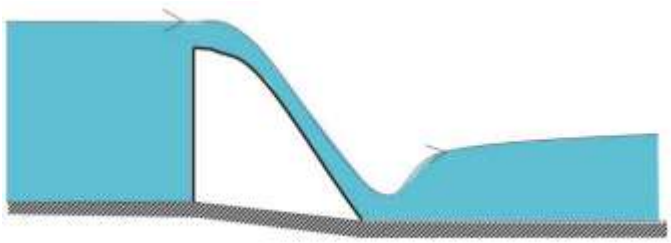

FIGURE 14. Ogee weir section (Kitchen et al. 2016)

Straight drop weirs (depicted in figure 15.) consist of a sharp drop after a weir crest, usually directly into a stilling basin. The steep downstream face of the weir results in plunging flow and tends to create dangerous recirculation in the stilling basin, thus causing severe scour. These types of weirs can be a significant safety hazard. and are no longer seen as good practice. Existing weirs of this type can usually be altered to improve hydraulic safety.

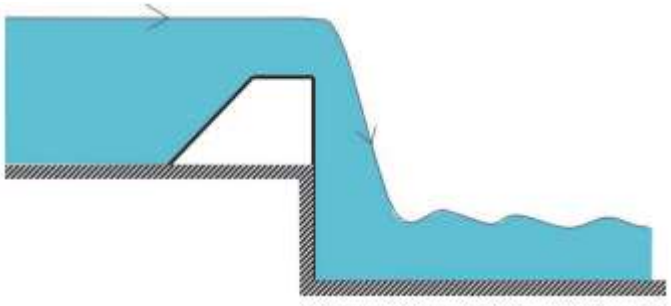

FIGURE 15. Straight drop weir section (Kitchen et al. 2016)

Rock weirs and rock ramps (shown in figure 16.) are formed from dumped rock sized to withstand the flow conditions at the weir site. They may be used to create adequate head for diversion/abstraction i.e., helps to dissipate energy and lowering the speed current downstream. Rock weirs may maintain fish passage during low flows and to regulate channel gradient for erosion control. A rock ramp is usually provided with a low-flow channel designed to give suitable conditions for fish passage during low-flow conditions.

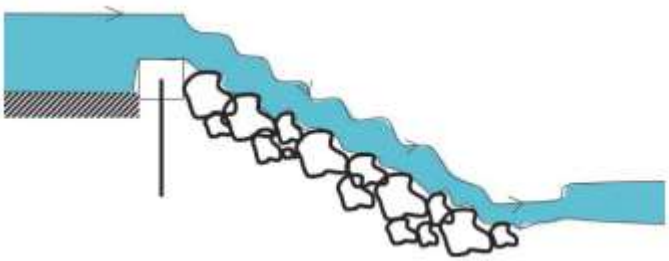

FIGURE 16. Rock weir section (Kitchen et al. 2016)

A rarely employed alternative is the air-regulated siphon (illustrated in figure 17.) These structures are self-regulating with no moving parts and are able to achieve large discharges for small increases in upstream water level. Although there are some examples in Europe, these structures have rarely been constructed in the past, partly due to historic concerns around trash and ice blockages and lack of design precedents.

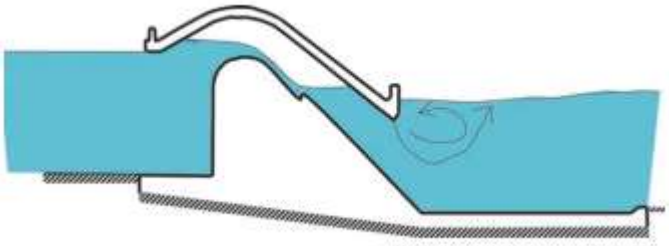

FIGURE 17. Air-regulated siphon weir section (Kitchen et al. 2016)

Tyrolean and Coanda screen weirs (shown in figure 18.) are used for in-channel intakes on steep streams. Water is abstracted from the main flow through a screen over a gutter. A flat or inclined rack of bars is placed on the downstream of the weir, to allow water to drop into an off-taking channel, which flows by gravity into the rest of the system. Trash and coarse sediment are excluded by the bars. (Kitchen et al. 2016)

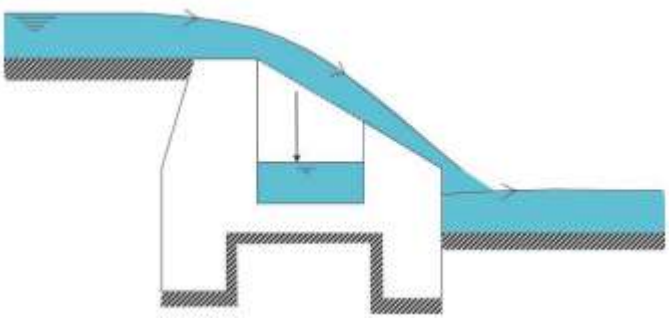

FIGURE 18. Tyrolean weir section (Kitchen et al. 2016)

4 WEIR SITE SELECTION

Detailed location considerations for the selection of the site are flow conditions, landscape factors, soil properties, coastal land uses and ecological constraints. However, for the purpose of this thesis, this section will only note geomorphological, hydraulic and geotechnical considerations and constraints.

Rivers will try to move over time as natural processes allow it to change its path and grade to reach equilibrium with its environment. A weir can present a significant imbalance to this equilibrium. Over long periods of time, a weir will be subjected to the natural forces of the river, which will strive daily to erode, undermine, circumnavigate, and otherwise remove the structure in its path. Weirs should be carefully located, designed constructed, monitored, maintained, and adapted as necessary in order to remain functional and safe over their design life. (U.S. Department of the Interior Bureau of Reclamation Technical Service Center 2016.)

The purpose of a geomorphic investigation is generally to (1) determine the areal distribution and physical characteristics of the various surficial deposits, (2) reconstruct the general fluvial history of the area, (3) conduct stratigraphic studies for the purpose of correlating various environmental settings and understanding the geomorphic processes responsible for their formation, and (4) provide a scientific basis in support of proposed channel modifications and design. (U.S. Department of the Interior Bureau of Reclamation Technical Service Center 2016.)

The design of the hydraulic capacity for a weir is site-specific, considering the acceptable afflux with respect to the topography of the upstream riverbanks and that the weir capacity is measured to prevent out-of-bank flow for a given return period flood and to avoid impending drainage to areas upstream. (Kitchen et al 2016.) The use of coastal lands may also affect the location of the weir. For example, low-lying coastal fields and their irrigation may be adversely affected by the weir during seasonal floods. (U.S. Department of the Interior Bureau of Reclamation Technical Service Center 2016.)

Ground investigations are needed for the general planning phase, as then the final location for the weir can be decided. In addition to the general plan, necessary water right permits require alternative selection of locations that should also be adequately investigated to find the most advantageous point.
Soil investigations are to be conducted with field surveys to give a general picture for necessary evaluation. (Heikkilä 2013.)

Weir locations must be investigated with a weight sounding to determine the thickness of soft layers and to compare alternative locations. In addition, soil samples are to be taken to define soil types. For geotechnical design, additional information can be complimented by acquired weight sounding and a combination of cone penetration test and ram drilling. Vane shear test can determine the shear strength of soft soils (i.e., clays and silts).

From soil samples, soil type, index properties and water saturation can be determined. Soil samples can be used to assess accurately soil water permeability, strength and deformation parameters and erosion susceptibility.

A primary requirement of weirs regardless of type or location, is to limit fluctuations in water level to maintain sufficient draught (above and below water). Overtopping of the canal embankment should be avoided as this can lead to external erosion and breach. (U.S. Department of the Interior Bureau of Reclamation Technical Service Center 2016.)

5 STRUCTURAL LOADS

5.1 Weight of structure and soils

The weight of the weir structure is calculated using the volumetric weight of reinforced concrete 25 kN/m^3 and natural stone 27 kN/m^3. Volumetric weight properties for materials used in engineered fill can be observed in chart 1. Other values applicable when determined.

TABLE 2. Volumetric weight properties of fill ad soils (adapted from Vesihallitus 1983)

Soil type			Volumetric weight (kN/m³)	
			Above groundwater level	Below groundwater level
Clay			15 . . . 17	5 . . . 9
Mire	(saturated)		11 . . . 16	
Peat	(saturated)		10 . . . 12	
Silt	(dry)		15 . . . 18	
Silt	(moist)		16 . . . 20	8 . . . 12
Sand		(L)	15 . . . 17	9 . . . 11
d_{10} ≤ 0,06 mm		(MD)		9 . . . 11
		(D)	16 . . . 18	9 . . . 11
Sand		(L)	16 . . . 18	9 . . . 11
d_{10} > 0,06 mm		(MD)		10 . . . 12
		(D)	17 . . . 19	10 . . . 12
Ruble		(L)	17 . . . 19	10 . . . 12
		(MD)		10 . . . 12
		(D)	18 . . . 20	10 . . . 12
Moraine		(VL)	16 . . . 19	10 . . . 12
		(L)	17 . . . 20	10 . . . 12
		(MD)	18 . . . 21	11 . . . 13
		(D)	19 . . . 23	11 . . . 14
Compacted fill		blasted stone	15 . . . 18	9 . . . 11
		aggregate	19 . . . 22	11 . . . 13
		gravel	18 . . . 21	11 . . . 13
Macadam d ≤ 60 mm			15 . . . 18	

(VL) very loose (MD) mildly dense
(L) loose (D) dense

5.2 Lateral earth pressure

Lateral earth pressure is the horizontal pressure that soil exerts against a structure, that must be taken into consideration in the geotechnical design of weirs. The magnitude of lateral earth pressure is dependent on the interaction between a retaining structure and its adjacent fill. When the structure does not experience movement, the in situ lateral pressure of soil can be estimated from the coefficient of earth pressure at rest, denominated as K_o.

Active earth pressure is soil pressure that is trying to move the structure. When the retained fill is at a point of incipient failure by shearing due to unloading in the lateral direction, coefficient of active pressure it is denominated as K_a.

Passive earth pressure is the pressure that is trying to keep the structure in place. When the retained fill is at the point of incipient failure by shearing due to loading in the lateral direction, coefficient of passive pressure it is denominated as K_P.

General equations for lateral earth pressures in cohesionless soils at depth z.

Active pressure:

$$P_a = \gamma \cdot z \cdot K_a \quad (2)$$

Passive pressure:

$$P_p = \gamma \cdot z \cdot K_p \quad (3)$$

Pressure at rest:

$$P_o = \gamma \cdot z \cdot K_o \quad (4)$$

Where γ is the volumetric weight of soil (below the water level γ' is used, see table 1.)

Active earth pressure coefficient:

$$K_a = \frac{1 - \sin \emptyset'}{1 + \sin \emptyset'} \quad (5)$$

Passive earth pressure coefficient:

$$K_p = \frac{1 + \sin \emptyset'}{1 - \sin \emptyset'} \quad (6)$$

Coefficient of earth pressure at rest for granular and normally consolidated fine-grained soils:

$$K_o = 1 - sin\emptyset'$$

Friction angle of the soil:

$$\emptyset'$$

In the case of seepage flow, the seepage influence on the volumetric weight decreases when flow is downstream and increases when flow is upstream. (Vesihallitus 1983).

Anchored weirs i.e., concrete structure imbedded into the bedrock can be regarded as rigid, where the exerted earth pressure is defined as earth pressure at rest. (Vesihallitus 1983).

The total pressure at rest down to a depth z = h, that is vertical and against a smooth weir anchored to the bedrock can be calculated using the following formula. The rest pressure point of efficacy is in accordance to figure 19. at 1/3 h.

$$P_o = \frac{\gamma\, h^2}{2} K_o \quad (7)$$

Weirs with a cantilever retaining foundation can be considered as a transitional rigid structure to which the ground pressure is characterised as active pressure.

The total active pressure down to a depth z = h that is vertical and against a smooth weir with a shallow foundation can be calculated using the following formula.

$$P_a = \frac{\gamma\, h^2}{2} K_a \quad (8)$$

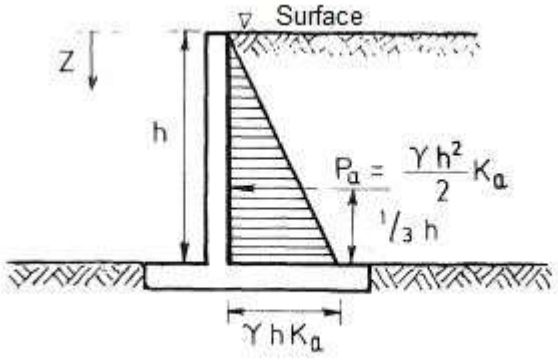

FIGURE 19. Total active pressure P_a point of impact (Vesihallitus 1983)

5.3 Water pressures

Hydrodynamic loads are the result from the water flowing against and around a rigid structure element. (American Society for Civil Engineering 2007). Hydrodynamic loads include external water pressure, pore pressure and uplift. Hydrodynamic loads impeding on a weir are shown in figure 20.

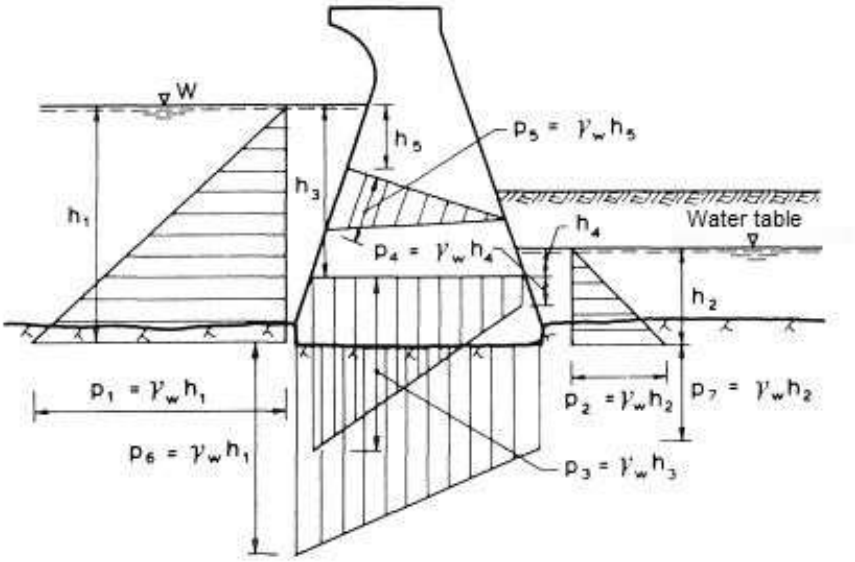

FIGURE 20. Definitions of water pressures (adapted from Vesihallitus 1983)

P_1, P_2 External water pressures

P_3, P_4, P_5 Pore water pressure

P_6, P_7 Uplift

h_1, h_2 Distance to the surface of water

h_3, h_4, h_5 Depth from the surface of water to considered intersections

γ_w Density of water

5.3.1 External water pressure

External water pressure is defined by height in water levels upstream and downstream to the weir or the depth of the water table. It effects perpendicularly against all surfaces in contact with water. When the water level is above the crest of the weir, the water pressure is calculated according to figure 21.

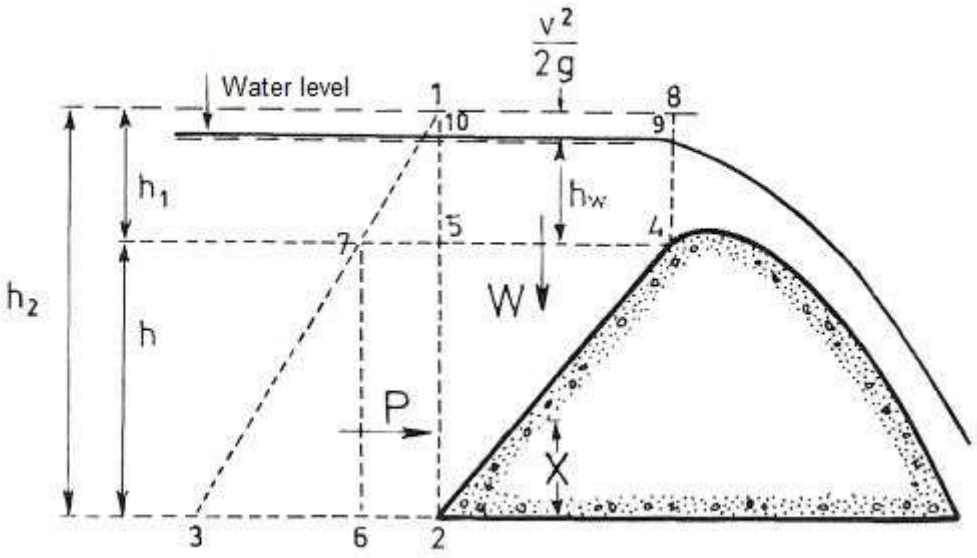

FIGURE 21. External water pressure P and weight of water W flowing over the crest (adapted from Vesihallitus 1983)

$$P = \frac{1}{2}\,\gamma_w \left({h_2}^2 - {h_1}^2\right) \quad (9)$$

$$X = \frac{1}{3}\,h\,\frac{h_2 - 2h_2}{h_2 + h_1} \quad (10)$$

W = centre of gravity for the weight of water in areas 1-2-4-8 (W = 0, when weir is vertical.)

5.3.2 Pore water pressure

Pore water pressure refers to the pressure of water held within soil or rock in gaps and between particles. The pressure developed in pores depend upon the depth of ground water and seepage flow condition. Pore water pressure is used for finding the effective stress beneath the soil. Difference between pore water stress and total stress is said to be effective stress. The pore pressure in soil is divided into two categories, negative pore pressure and positive pore pressure.

The zone of soil that is below the water table will be fully saturated. The pressure in the pore is greater than atmospheric pressure. Therefore, is defined as positive pore pressure. Above the water table, soil remains unsaturated and pressure in pores is lower than the atmospheric pressure. Hence, defined as negative pore pressure. Pore water pressure can affect compaction of soil, settlement of soil and shear strength.

5.3.3 Uplift

Uplift acts between the weir and the foundation level perpendicular to the structure. When the weir is founded on bed rock, the uplift is equal to the pore water pressure. If the weir is founded on soil, the uplift is determined more accurately by means of a flow network which requires knowledge of soil permeability coefficients of founding soils and hydraulic gradient, see section 6.3. This thesis will not divulge into the complexities of flow networking. The weir can fail hydraulicly due to uplift, this occurs when the pressure exerted by water seeping under the structure is sufficient to lift the structure vertically or reduce vertical stress under the structure to a degree that it becomes unstable. Flotation can be a risk under high groundwater levels.

5.3.4 Flowing water

Flowing water load perpendicular to the weir is denoted as P_w and is defined in the formula 11.

$$P_w = \mu_k \cdot A \cdot p \quad (11)$$

μ_k hydrodynamic coefficient (depending on Reynolds number 0,7 – 1,2)

A Face area of the weir

p Pressure caused by flowing water

Pressure caused by flowing water is calculated with:

$$p = \frac{\gamma_w \, v^2}{2g} \quad (12)$$

γ_w density of water (kN/m^3)
v flow of water (m/s)
g acceleration of gravity (m/s^2)

5.3.5 Parallel flowing water

Parallel flow load against the weir is determined by:

$$P_1 = \mu_1 \cdot A \cdot p \quad (13)$$

μ_1 surface roughness coefficient depending on structure

smooth surface (e.g., ice)	μ_1 = 0,005
rough surface (e.g., concrete)	μ_1 = 0,015
uneven surface	μ_1 = 0,01 . . . 0,10

A surface area of structure (m^2)
p see formula (12).

The point of application of the load caused by flowing water under the surface is assumed to be the centre of gravity at the floor of the weir structure.

Measuring the height of the water level should be conducted when afflux is occurring. For weir design the most hazardous situation depends on the fluctuations of head and down stream flow, which is not always the greatest at time of high-water level. In some situations, the water level difference can be greatest at normal water-level. Gauging the velocity of water should be conducted during peak velocity, which not always is the velocity of high water. Drag and flow have an immense impact when designing rock weirs.

5.4 Ice impact

The thickness of a solid ice sheet formed against a weir depends entirely on the water flow over the weir, and the potential load-bearing thickness that must be evaluated in each case separately. In the event where the flow is non-existing/minuscule, the thickness of ice can be substantial to the loadbearing thickness. Whereas if the flow is moderate then there are no ice loadbearing forces.
The breaking up of ice and possible thrusts depend on local circumstances, which will not be discussed in the thesis.

Water level fluctuation can cause ice to adhere to weir structures due to vertical motion and can cause significant damage.
Horizontal loads due to the accumulation of ice floe can be determined by the following equation. (Vesihallitus 1983). The load resultant is at a depth h/3 from the surface of the ice (h= thickness of ice).

$$P = P_1 + P_2 + P_3 + P_4 + P_5 \quad (14)$$

P_1 parralel windload to the ice

$$P_1 = \mu_1 \cdot q_k \cdot A\ (kN) \quad (15)$$

μ_1 ice coarseness coefficient (=0,001 – 0,003)

q_k wind pressure (= 0,5 kN/m^2)

A = Surface area of the ice floe (m^2)

P_2 parralel load of water flow under the ice floe (see formula 13.)

P_3 ice floe horizontal weight component

$$P_3 = G \cdot i\ (kN) \quad (16)$$

G = weight of the ice floe

i = gradient of the water level (1:n)

P_4 flow load imposing on the ice floe edge (see formula 11.)

P_5 = wave forces imposing on the ice floe edge (≈ 0)

For a singular vertical structure shearing a motile ice floe, the maximum static load P_1 can be determined by the following formula:

$$P_1 = k_1 \cdot k_2 \cdot k_3 \cdot b \cdot h \cdot \sigma_{ip} \quad (17)$$

k_1= shape coefficient (= 1,0)

k_2 = contact coefficient between ice and structure (= 1,0)

k_3 = form quotient coefficient

$$k_3 = 1 + 1{,}5\ \frac{h}{b} \quad ()$$

b width of structure at ice adherence. (m)

h ice thickness (m)

σ_{ip} ice compression strength eigenvalue (1,0 – 3,0 MN/m^2)

Water level fluctuation causing adhered ice to inducing vertical loads against the weir can be determined with the following formula:

$$V = k_s \cdot v_v \cdot t \cdot \sqrt[4]{h^3}\ \left(\frac{kN}{m^3}\right) \quad (18)$$

k_s = coefficient (see figure 22.)

v_v = water level decrease or increase speed (m/h)

t = time (in hours) from the moment water level fluctuates

h = thickness of ice (m).

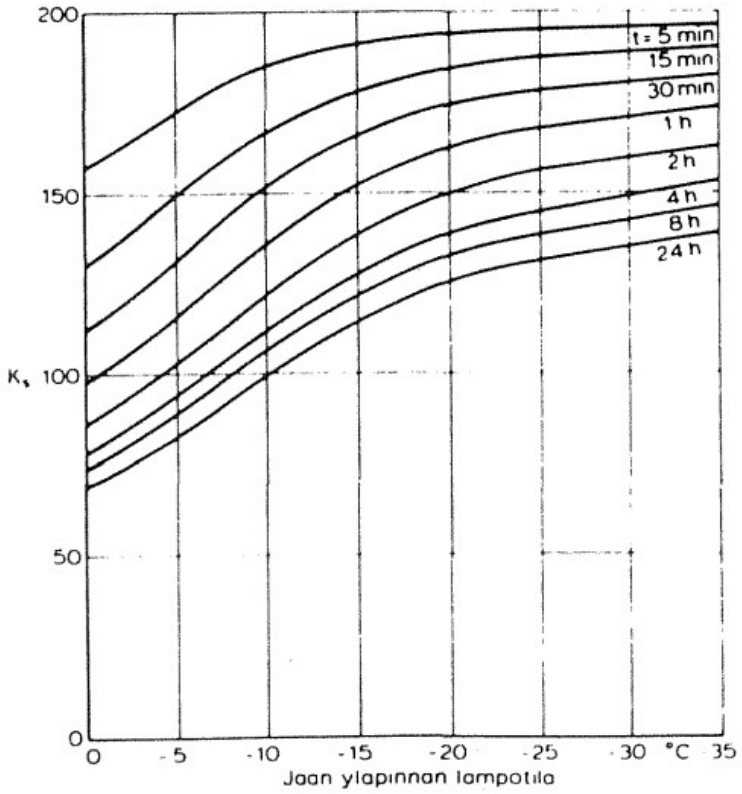

FIGURE 22. Coefficient k_s (t is the moment from when the water level fluctuates) (Vesihallitus 1983)

The increase in rate of flow during freezing temperatures, can cause a phenomenon known as frazil ice. Frazil ice is amorphous ice formed by the accumulation of ice crystals in water that is too turbulent to freeze. Studies and evidence of how to restrict the build-up of frazil ice around a weir are minimal. However, through the principles of hydrology frazil ice can be reduced by limiting the flow rate of water upstream. Frazil ice can block the flow of water thus, may impair the proper function of the weir.

6 GEOTECHNICAL ASPECTS

6.1 Subsidence risks

Significant settlement can have detrimental effect on the operation of a weir. Therefore, it is essential to choose the location of the weir where soil subsidence does not occur or is slight. When designing a weir, the crest cannot afford any decline in height. The crest of the weir can however be raised afterwards. The magnitude of subsidence is also affected by the height. Subsidence in moraine and silt are small. Excess pore water pressure dissipation and secondary consolidation (creep) in clays and organic subsoils can cause large deformations of the weir structure. (Briaud 2013). Therefore, it is important to investigate deformation properties of subsoil from laboratory testing on soil samples.

6.2 Strength properties of soils

The shear strength of soil is commonly described as the sum of a cohesive and a frictional component. It can be expressed in the following formula:

$$\tau = c' + \sigma' \emptyset' \quad (19)$$

τ = Shear strength
c' = effective cohesion
σ' = effective normal stress
$\emptyset'$ = effective angle of friction

Knowledge of strength parameters c' and $\emptyset$ are necessary when detailed stability, load-bearing capacity and earth pressure calculations are done.

Soil effective shear strength parameters c' and $\emptyset$ are determined in a laboratory setting by triaxial or direct shear tests. In cohesive soils, the undrained shear strength (s_u) can be determined in the field with a vane shear test (VST) or in the laboratory by undrained triaxial or direct simple shear testing. Strength properties

can be further assessed by penetration tests. Note that for cohesive soils (i.e., clays, silts) equation (19) becomes $\tau = s_u$. (Briaud 2013).

The significance of the soils shear strength for the stability of the weir is paramount and must be examined especially when clay and silt are present.

6.3 Permeability

The ability of soil to allow flow of water though it is called permeability of soil.
Darcy's law states that the rate of fluid flow through porous medium is proportional to the potential gradient with that fluid. Darcy's permeability is a property of both porous medium and the fluid moving through the porous medium.
Flow of water through soil follows the principles of Darcy's law that is denominated as:

$$v = ki \quad (20)$$

v = volumetric flow rate (m/s)
k = permeability coefficient or hydraulic conductivity (m/s)
i = hydraulic gradient (head loss over the displacement of flow)

Soil permeability depends on the density (porosity) and the grain size.
Variation in soil permeability coefficients for different soil types is presented in table 2.

TABLE 2. Soil permeability coefficients (Vesihallitus 1983).

Soil type	Permeability coefficient k (m/s)	NB
Gravel	10^{-2} - 10^{-3}	Highly permeable
Sand	10^{-3} - 10^{-5}	Highly permeable
Silt	10^{-5} - 10^{-9}	
Sandy and silty moraine	10^{-6} - 10^{-9}	
Clay	10^{-9} - 10^{-11}	Practicly impermeable

The permeability coefficient can be determined through laboratory tests or in situ field testing. For further reading see Nagy, Takács, Huszák, Mahler, Varga 2013.

6.4 Hydraulic failure and erosion

Hydraulic failure of a weir foundation can either occur by two mechanisms. A piping failure occurs when the hydraulic gradient in the foundation material is sufficient to cause washout of material, undermining and ultimately collapse of the structure i.e., the movement of soil particles due to water percolating through soil, leading to internal erosion. And failure by uplift as previously discussed in chapter 5.3.3. Riverbed consistencies in Finland are predominantly layers of silt and clay, that do not often wash away like sand particles do.

Weir foundation design are generally split into two broad categories – permeable foundations and fine grain soils e.g., alluvial soils.

Where a weir is founded on a permeable foundation, seepage and piping failure must be considered. The exit gradient is the hydraulic gradient of the seepage flow under the base of the weir floor. (Rossi, Bačić, Kovačević, Librić, 2021) The term exit is used because the critical position of the hydraulic gradient is invariably at the downstream toe of a structure i.e., the point at which the seepage exits the foundation. If the gradient is too large ‘boiling’ of the foundation surface is initiated as soil is washed away by the percolating water. The flow concentrates into the resulting depression removing more soil and creating progressive backwards scour, which may eventually undermine the barrage structure. This mode of failure is called piping. Scour is the erosion of sediment from around the base of the weir, thus being a function of interaction between hydrodynamics (tides and waves) and geotechnical properties of the riverbed.

In fine grain soils the critical exit gradient, i.e., the point at which boiling is initiated, is around unity. However, due to local variations in the hydraulic gradient caused by inconsistencies in the soil matrix, the inability to inspect the many po-

tential seepage paths and the catastrophic nature of any potential failure mechanism, relatively high limit states in design are recommended. Limit states refer to where a structure's condition no longer fulfils its design criteria, such as the structural integrity and durability. Limit states are conditions of potential failure. (American Society for Civil Engineering 2007). The exit gradient can be calculated from Khosla's equation, see formula (21). Typical permissible exit gradients are listed in table 3.

$$i_E = \frac{h_w}{\pi \cdot d \cdot \sqrt{\frac{1+\sqrt{1+\left(\frac{B}{d}\right)^2}}{2}}} \qquad (21)$$

B = width of the weir

h_w = height of retained water

d = depth of cut-off.

TABLE 3. Typical acceptable exit gradients (Kitchen et al 2016).

Material	Permissible exit gradient
Fine sand	1:6 to 1:7
Coarse sand	1:5 to 1:6
Gravel	1:4 to 1:5

The differential head across the weir is fixed for a given design, so to reduce hydraulic gradients to the required level, the seepage path must be lengthened. This can be achieved by lengthening the impermeable floor of the weir (both upstream and downstream of the weir crest) or by providing impermeable barriers or seepage cut-offs below the weir.

According to recent studies conducted by Lantz, Crookston and Palermo (2021), the length or depth of a cut-off wall should be larger than the maximum scour depth, as the maximum scour depth occurs in correspondence or in proximity of the edge of the apron to protect from undermining of an erodible foundation. (Lantz et al. 2021).

The following formula is proposed.

$$L_c = FS \cdot Z_{max} \quad (22)$$

L_c = depth of cut-off wall
FS = factor of safety
Z_{max} = maximum scour depth

Maximum scour depth can be ascertained through physical investigation or through detailed calculation e.g., using Lacey's Theory where the design flood discharge is divided by partial size multiplied by various constants. (Lacey 1946). This thesis will not divulge into the calculations of scour depth.

For an in-depth analysis and calculation example of stability a weir against hydraulic failure in particular the verification of stability against piping regarding Eurocode 7 HYD limit states refer to appendix 1.

6.5 Frost heave

Stratums are subjectable to frost heave when the soil temperature is below freezing point, adjacent to a freezing front or the soil is permeable. Thus, the soil weight or lateral earth pressure does not suffice to compensate the pressure arising from frost heave.

Frost is the atmospheric moisture crystallized in the stratum that causes upwards swelling of soil.

Silty and loamy soils, which contain fine particles, are examples of frost-susceptible soils. Non-frost-susceptible soils may be too dense to promote water flow (low hydraulic conductivity) or too open in porosity to promote capillary flow.
In Nordic countries all weir locations are on frost susceptible soils excluding bedrock.

Frost heave in strata can be estimated with granular size grading curves shown in figure 23. Soils in area 1 are susceptible to frost heave. Soils in areas 2, 3 and 4 are exempt of frost heave if they are below the former curve.

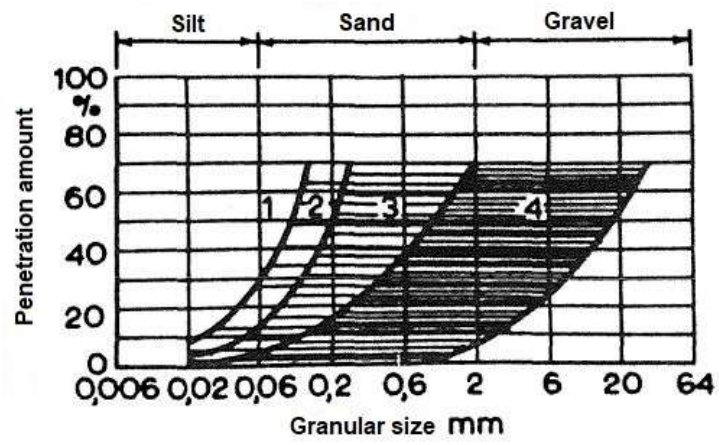

FIGURE 23. Frost heave estimates according to strata granular size (adapted from Vesihallitus 1983)

Weirs constructed from soils can be affected by frost heaving during cold spells if the water level remains low and the components of the weir are above the water level. Concrete weirs may be subjective to damage if the foundation for example if exposed due to low water level.

According to the figure 23, condensed soils used in weir fill i.e., sand, silt and moraine are susceptible to frost heave. Gravel moraine is regarded not suspectable to frost heave due to its granular size. Frost heave assessment on susceptible soils can be conducted in laboratory through capillary and frost heave tests.

Frost heave depth is the penetration depth from the surface of the ground.
Figure 24. shows the average relation - frost heave depth and frost hours per annum in various frost susceptible soils for clear ground i.e., not insulated by snow. Curves I, II and III indicate the penetrative depth of frost in soils that are not covered by snow.

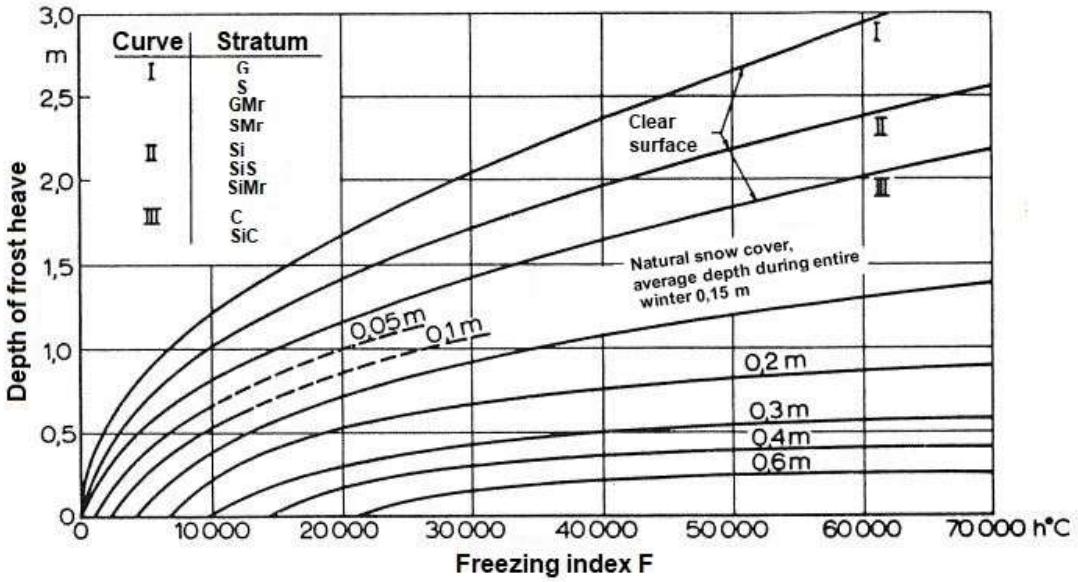

FIGURE 24. Freezing index relation to depth of frost heave in various soil types (adapted from Vesihallitus 1983)

Freezing index, that is frost hours per annum, are based on the average accuracy for structural design of a ten-year period. Geographically the freezing index for areas in Finland is illustrated in figure 25.

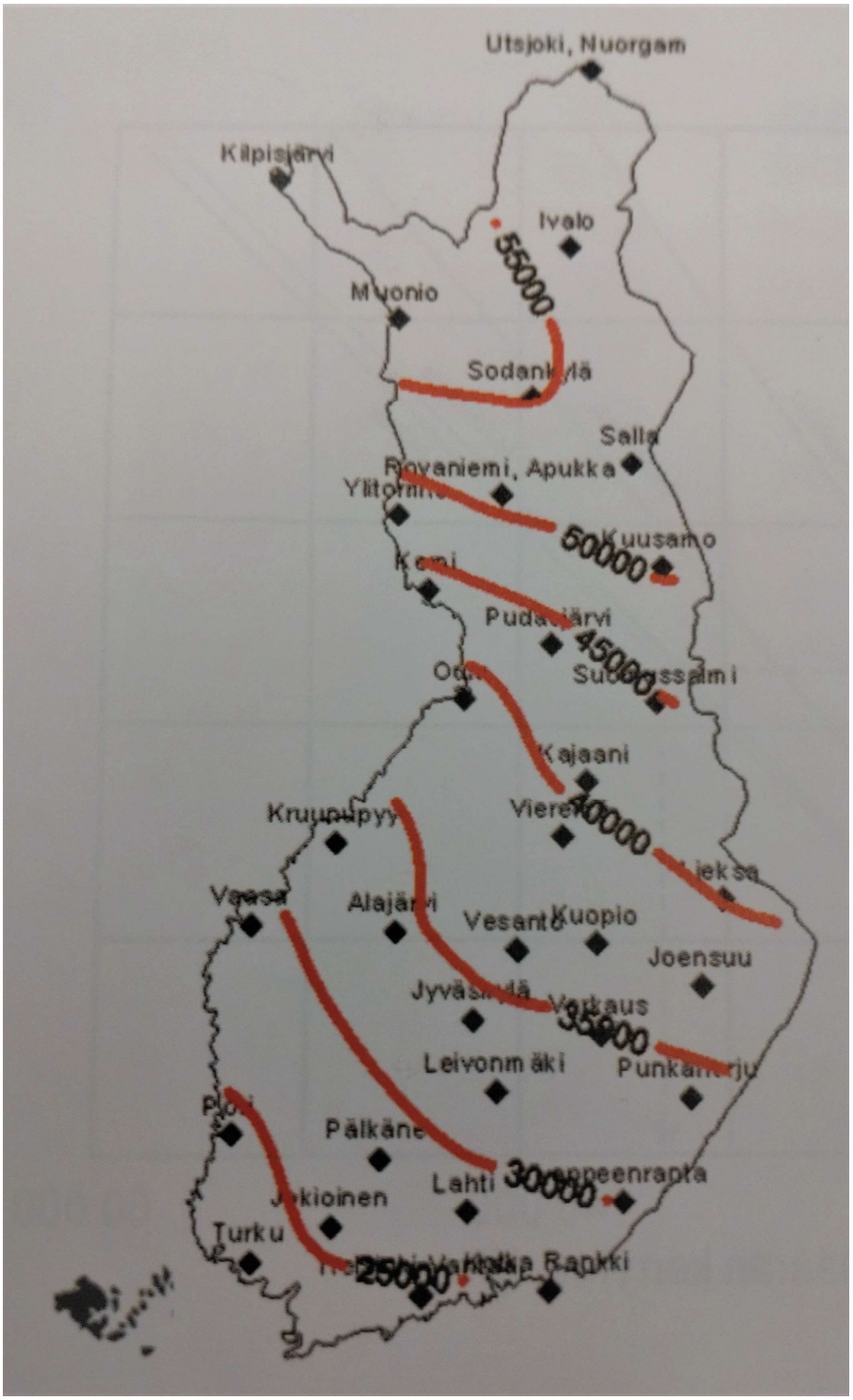

FIGURE 25. Freezing index based on a 10-year period for Finland (RIL 261-2013 2013)

7 CONCLUSION

The objective to outline the structural, geotechnical, and hydraulic design considerations of weirs was conducted in conjunction with various sources, resulting in a compendium of theoretical aspects, that are necessary for the design and for the integrity of the structure. Throughout the investigation into the complexities of weir design, this was a vast topic, and it is worth noting that not all design considerations are addressed due to the constraints of a bachelor's thesis.

However, through examination it cannot be overemphasised the importance of ground investigations and the requirement for alternative locations in the planning phase for weirs. Due to the geotechnical and climate constraints here in Finland, one can summarise that weirs constructed from soils are affected by frost heaving during cold spells and that horizontal loads due to build-up of ice sheets will affect the performance of the weir structure.

Further investigation concluded that excess pore water pressure dissipation and creep in clays and organic sediments can cause large deformations of the weir structure and that is paramount to investigate deformation properties from soil samples, especially when riverbed subsoil is made of soft organic material.

The use of Eurocode 7 provides formulas and guidelines for verifying the hydraulic failure limit state HYD. One important design consideration to reduce hydraulic gradients to the required level is to increase the length of seepage path. This can be achieved by lengthening the impermeable floor of the weir or by providing impermeable barriers or seepage cut-offs below the weir. Due to local variations in the hydraulic gradient caused by inconsistencies in the soil matrix relatively high limit states in design are recommended.

The content of this thesis is to assist its readers in structural and geotechnical design consideration in a concise manner and hoping that the reader finds this information beneficial. Recommendations for further improvement into this topic would require separate assessment and analysis into individual disciplines and complemented with field as well as numerical studies.

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APPENDICES

Appendix 1. Example: Stability of a weir against hydraulic failure

This is an example of how to assess the exit hydraulic gradient at the toe of the weir by an analytical approach without drawing a flow net for each change in geometry. The exit gradient for this example is calculated by substituting the values of B, h_w and d into Khosla's equation (21).

For limit state HYD, partial factors are given in EN-1997-1 Annex A5. (Bond & Harris 2008).

Eurocode 7 provides two expressions, 2.9(a) and 2.9(b), for verifying HYD. The former compares the design destabilizing pore water pressure to the design stabilizing vertical stress; the latter compares the design destabilizing seepage force to the design stabilizing submerged weight. Both equations are applied at the base of the column of soil under consideration. (Bond & Harris 2008).

Design situation

Consider a weir (see figure 26.) of width B = 14m which is retaining free water of height h_w = 6m. A cut-off wall of depth d = 3.4m helps to reduce the hydraulic gradient at the downstream end of the weir i.e., the exit gradient.

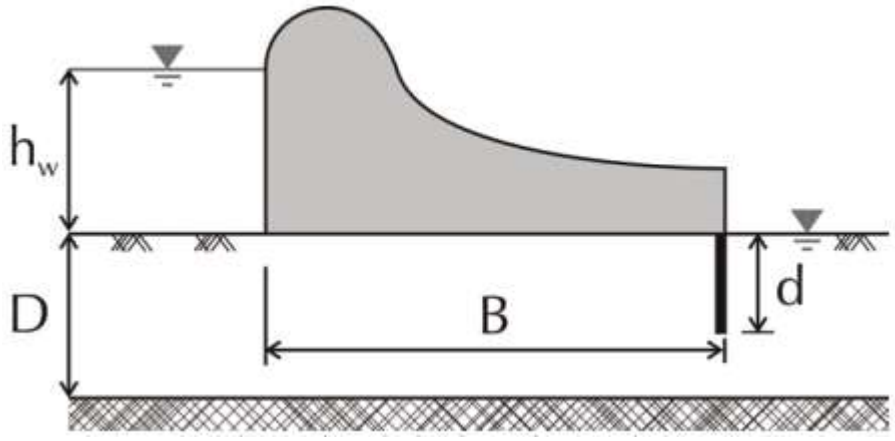

FIGURE 26. Weir subject to large hydraulic gradients at downstream toe (Bond & Harris 2008)

The weir is founded on a permeable stratum of characteristic weight density:

$$\gamma_k = 18{,}5\,\frac{kN}{m^3}$$

An impermeable stratum is located at depth D = ∞.

The characteristic weight of density of water is

$$\gamma_w = 9{,}81\,\frac{kN}{m^3}$$

Calculation model

For the situation where D = ∞, the exit gradient can be calculated from Khosla's equation:

$$i_E(B,\ h_w, d) = \frac{h_w}{\pi \cdot d \cdot \sqrt{\frac{1+\sqrt{1+\left(\frac{B}{d}\right)^2}}{2}}} \qquad (21)$$

where B is the width of the weir, h_w the height of retained water, and d the depth of cut-off.

Actions

The characteristic exit gradient for the design situation is:

$$i_k = i_E(B, h_w, d) = 0{,}347$$

At depth d on the downstream side of the cut-off, we have the characteristic destabilizing pore water pressure:

$$u_k = \gamma_w \cdot (1 + i_k) \cdot d = 44{,}93\ kPa$$

and the characteristic stabilising vertical stress:

$$\sigma_k = \gamma_k \cdot d = 62{,}9\ kPa$$

The characteristic seepage force on the downstream side of the cut-off is (Assuming a plan area A = 1m^2)

$$S_k = \gamma_w \cdot i_k \cdot d \cdot A = 11{,}58\ kN$$

The characteristics submerged weight of the downstream side of the cut-off is:

$$G'_k = (\gamma_k - \gamma_w) \cdot d \cdot A = 29{,}55\ kN$$

Effects of actions

Limit state HYD, partial factor on destabilising actions is:

$$\gamma_{G,dst} = 1{,}35$$

Limit state HYD, partial factor on stabilising permanent actions is:

$$\gamma_{G,stb} = 0{,}9$$

Using Eurocode 7. exp. 2.9 (a):

The design destabilizing pore water pressure is:

$$u_{d,dst} = \gamma_{G,dst} \cdot u_k = 60{,}7\ kPa$$

The design stabilising vertical total stress is:

$$\sigma_{d,stb} = \gamma_{G,stb} \cdot \sigma_k = 56{,}6\ kPa$$

Using Eurocode 7. exp. 2.9 (b):

The design seepage force is

$$S_{d,dst} = \gamma_{G,dst} \cdot S_k = 15{,}6\ kN$$

The design submerged weight is

$$G'_{d,stb} = \gamma_{G,stb} \cdot G'_k = 26{,}6\ kN$$

Verification of stability against hydraulic heave using Eurocode 7 exp. 2.9 (a)

The degree of utilisation:

$$\Delta_{HYD} = \frac{u_{d,dst}}{\sigma_{d,stb}} = 107\%$$

The design is unacceptable if the degree of utilisation is > 100 %

Verification of stability against hydraulic heave using Eurocode 7 exp. 2.9 (b)

The degree of utilisation:

$$\Delta_{HYD} = \frac{S_{d,dst}}{G'_{d,stb}} = 59\%$$

The design is unacceptable if the degree of utilisation is > 100 %

Traditional factor of safety against piping

The soils critical hydraulic gradient:

$$i_{crit} = \frac{\gamma_k - \gamma_w}{\gamma_w} = 0{,}9$$

Factor of safety on hydraulic gradient is:

$$F = \frac{i_{crit}}{i_k} = 2{,}55$$

The degree of utilisation using expression 2.9 (a) is over 100%, whereas using 2.9 (b) it is less than 50%. Eurocode 7 does not explicitly state where the partial factors should be applied, which leads to the discrepancy between these expressions. (Bond & Harris 2008).

The traditional global factor of safety for this situation is 3.18. Recommended values for the global factor are between 1,4 and over 4,0. Generally, where the consequences of piping failure may have serious effects, then higher global factors are adopted. It appears that when equation 2.9 (a) is used, equivalent traditional factors of safety are between 3,0 and 4,5 but, when equation 2.9 (b) is used, it is closer to 1,5. Therefore one can conclude the equation 2.9 (b) does not provide a sufficient level of reliability. (Bond & Harris 2008).