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Seepage Considerations in the Design of the Leeds Flood Defences

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Abstract

This paper considers the different ULS and SLS mechanisms where prediction of transient seepage is required as part of the geotechnical design of linear flood defences. A methodology is described for considering the adverse effects of seepage induced buoyancy on the sliding limit state and, loading eccentricity which in turn influences the bearing limit state of gravity flood walls. One part of the site is examined in detail where a new earth structure is required to prevent a railway from flooding. Initial single bore hole tests indicated extremely high soil permeability and the need for a high-cost deep seepage cut-off. Subsequent large scale field monitoring with digital divers in boreholes and river gauge enabled seepage back analysis to confirm the actual mass soil permeability was up to two orders of magnitude lower. Consequently, the solution presented is an economical passive drainage system leading to a new pumping station and outfall that fitted within the original land boundary and maintenance plan for the project.

Keywords: Hydraulic Conductivity, Permeability, Seepage, Uplift, Piping, Sliding

1. General Seepage Considerations

Dams impound water for long periods of time when seepage reaches theoretical steady state conditions. These structures experience seepage flux as the impounded water finds paths of least resistance through the soil driven by gravity. Excessive water flux can be a serious problem and can lead to instability due to long-term internal erosion of the soils from under or within an earth dam. Clients spend large sums of money on the Ground Investigation, Design and Construction of such structures to provide barriers to seepage, and provide robust operational inspections and maintenance interventions where excessive seepage is identified.

Flood defences impound water for a short period of time and seepage flux may be acceptable without compromising the purpose of the asset. To achieve affordable defences clients may accept a small quantity of short-term ponding on the 'dry' side of the defence as a consequence of the strategy, provided the design demonstrates that no breach of the defence can occur, and the long-term inspection and maintenance risk can be managed. This is alluded to in various publications, for example Ciria C749 states *'Acceptable seepage rates, potentially linked to the capacity of landward drainage to deal with the water flow, should be agreed on a project basis'.* The Environment Agency Fluvial Design User Guide (Rickard 2010) states that '*seepage through and under embankments is not uncommon'*. It advises that provision must be made for seepage to accumulate at suitable locations from which it can be pumped or discharged by gravity after the flood recedes. Flood defences are also to be checked for other limit states that are influenced by seepage when no cut-off is provided, including ground bearing resistance, sliding, piping and uplift.

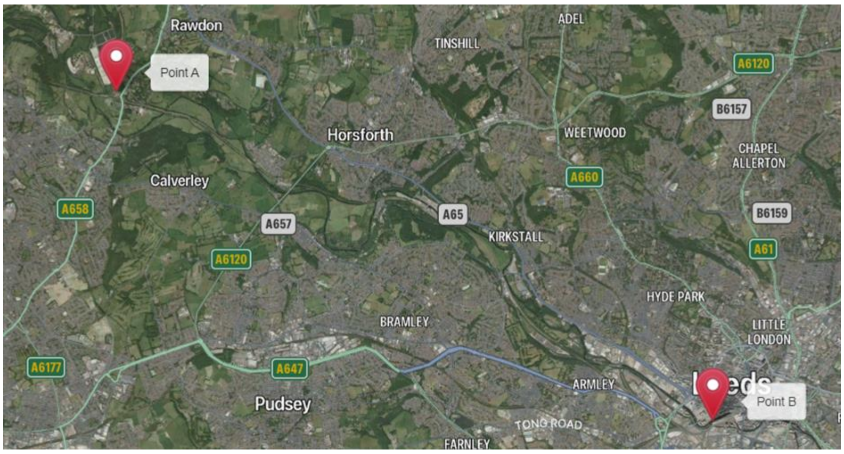

Figure 1: Leeds Flood Defences (Phase 2) – Project Location Plan.

2. Project Introduction

This project is located along the banks of the River Aire between Leeds Station (Point B Ch.0000) and Apperley Bridge (Point A Ch.16900), as illustrated in Figure 1. The project provides hard and soft flood defences comprising walls and earth structures. The locations of interest in this paper are a 1.25km soft defence in Kirkstall Meadows (Zone 13a Ch.6350-7600) and various hard defences in the city (Zones 11 to 12 Ch.2500-4000).

3. Urban Gravity Flood Walls

3.1 General

Within the city there were several low height flood walls founded on Made Ground. The most economical solutions were generally gravity walls, and the permeability of the Made Ground was of interest when checking limit states of sliding, bearing, slopes during rapid draw down, piping and SLS flux (seepage under the walls).

3.2 Geology

The geology of this part of the site comprised of Made Ground overlying fine and coarse-grained alluvium, which in turn overlies bedrock of the Pennine Lower Coal Measures, as illustrated in Figure 2.

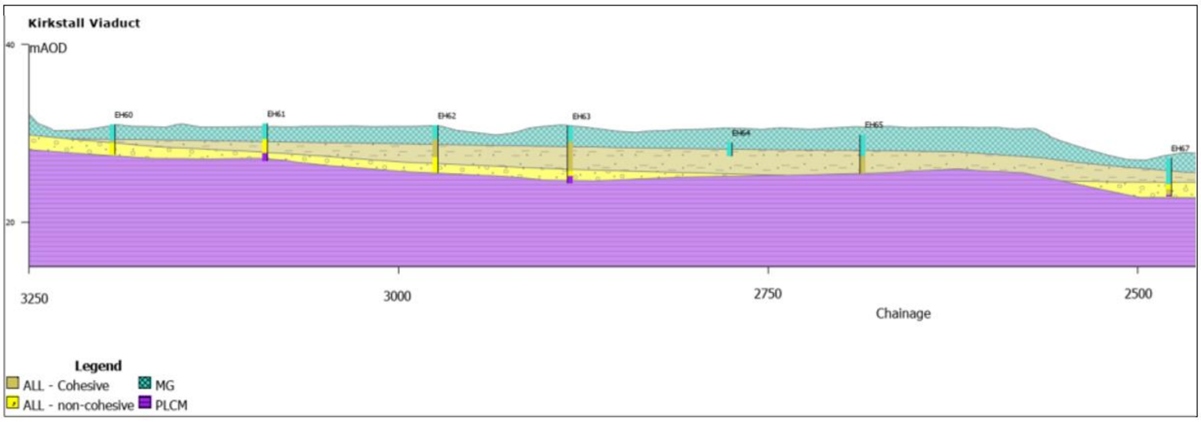

Figure 2: Geological ground section along the River Aire in Zone 11.

Trial pits were excavated to visually inspect the nature of the Made Ground, carry out soil infiltration tests to verify the absence of preferential pathways and estimate the soil permeability. The Made Ground encountered was generally 'stony cohesive' in accordance with the Specification for Highway Works grading range where the permeabilities estimated from in-situ infiltration testing were in the range of $k = 5 \times 10^{-5}$ to 5×10^{-6} m/s.

3.3 Piping (HYD)

Partial factors for ULS hydraulic failure are presented in the UK National Annex to Eurocode 7 and Ciria C749 Box 4.1. With water being considered as a permanent action in the 'design flood' event the overall Factor of Safety against piping must be at least 1.5, i.e. exit gradient less than 0.67 for typical soil specific gravity and voids ratio. As recommended in the SEEP/W manual, an average exit gradient was considered over a distance of 1 to 2m in the vicinity of the exit location, to avoid problems with the exit gradient at ground surface being affected by model geometry, 2D flow and low effective stress. Ciria C731 9.7.2.1 states *'As permeability is a particularly difficult geotechnical parameter to assess, it is usual for the designer to consider a range of possible values for design, rather than just a unique value'*. Therefore, the risk of uncertainties in understanding mass soil permeability, anisotropy, and precision of transient seepage analysis was managed by checking piping for the best estimate permeability and one order of magnitude above and below this, for example 1×10^{-4}, 1×10^{-5} and 1×10^{-6} m/s with an assumption of Made Ground being isotropic.

3.4 Flux (SLS)

Preferential flow pathways in Made Ground were identified at one location and short (4m length) sheet piles were used to provide a cut-off through Made Ground and into natural soils. For other locations, analysis indicated that seepage would not appear on the dry side of a low height gravity wall, when founded on stony cohesive Made Ground subject to transient flood events of overall duration less than 3 days. Confidence in this approach was also gained from the Sheffield Lower Don Valley Flood Defences where similar solutions have performed well in major flood events on 20.09.2018 and 08.11.2019.

3.5 Sliding & Bearing (GEO)

Eurocode 7 is clear about the application of partial factors to a particular failure mechanism, but unclear about compound failure, for example partial buoyancy from seepage reducing the frictional sliding resistance of a gravity wall or increasing the eccentricity and peak bearing pressure on the land side of the foundation. The EA FDG Ch.9 notes this risk by stating *'providing a land side seepage cut off will increase temporary uplift flood pressure on the base, impacting sliding and bearing resistance'*. The draft EN 1997-3 states that '*groundwater levels and pressures (including potential changes in them) could affect the bearing resistance and sliding resistance of spread foundations and shall be considered*'. This paper illustrates the method.

The most onerous condition for sliding of a flat-bottom gravity wall on stony cohesive soil is shortly after the peak flood where active water pressure is still high and uplift (that reduces the normal force and in turn base friction) is maximum. The ULS partial factor on sliding (tan ϕ' / 1.25) was deemed to be too small when applied in conjunction with the results of unfactored seepage analyses hence judgement was applied to consider credible ranges of soil permeability and upper and lower bound values of buoyancy.

For bearing checks small forces from low height flood walls could present theoretical geotechnical failures where the horizontal flood action generates large eccentricity. The ICE Manual of Geotechnical Engineering (2012) Chapter 53 recommends that the eccentricity is kept within the middle third of the base of a gravity structure and the Meyerhof (1953) approach using an equivalent base width adopted and states that *'If the eccentricity falls outside the middle third, then it is generally wiser to ensure more uniform loading by reconfiguring the foundation geometry rather than attempting more sophisticated analysis*'. Bowles (1968) acknowledges that '*there are occasions when it is impossible to keep the soil resultant inside the middle one-third of the base'* and a simple equation can calculate the peak of a triangular pressure distribution. This provides a method to check that the ground can resist the worst-case 'theoretical' peak pressure. There were several locations where transient uplift indicated potential sliding or theoretical bearing failure and shear keys were provided. These are most effective when located on the river side of the footing where uplift is reduced, as illustrated in USACE retaining and flood walls (1994) but there were often site constraints, such as riverbank slopes or granular revetment, that required larger land-side shear keys. Figure 3 illustrates where a shear key generates passive resistance to both sliding and to rotation in a general shear bearing failure mechanism.

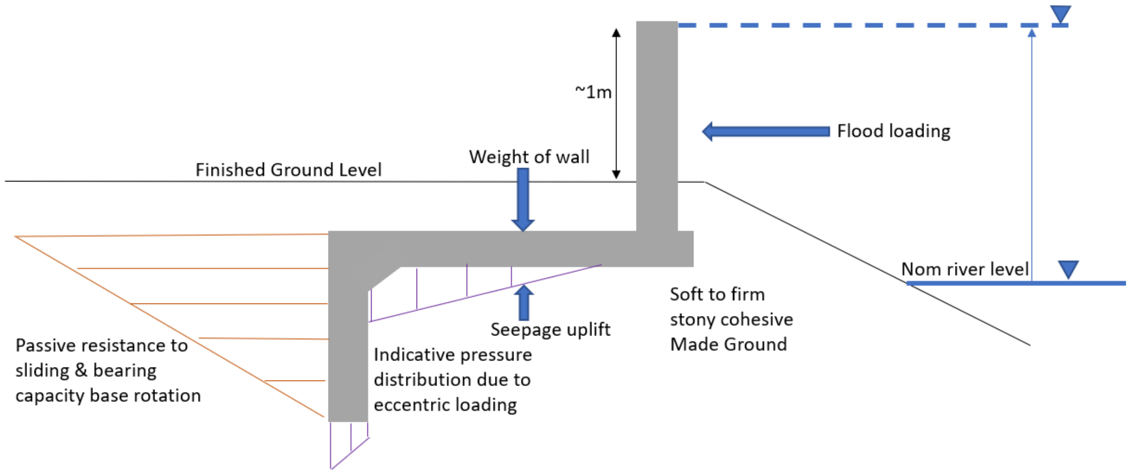

Figure 3: Indicative section of flood wall subject to seepage uplift and lateral flood pressure.

4. Kirkstall Meadows Flood Embankment

4.1 Overview

The Kirkstall Meadows flood risk is illustrated in Figure 4 with an image taken during the Storm Eva floods on 27.12.2015. It can be seen that extensive flooding has impacted many assets including the Leeds to Ilkley Railway line.

Figure 4: Aerial photograph of Kirkstall Meadows during the 2015 Storm Eva.

4.2 Geology

The geology at this location consists of topsoil overlying a thin layer of cohesive alluvium above a thick layer of sand and gravel, which in turn overlies bedrock of the Pennine Lower Coal Measures, as illustrated in Figure 5.

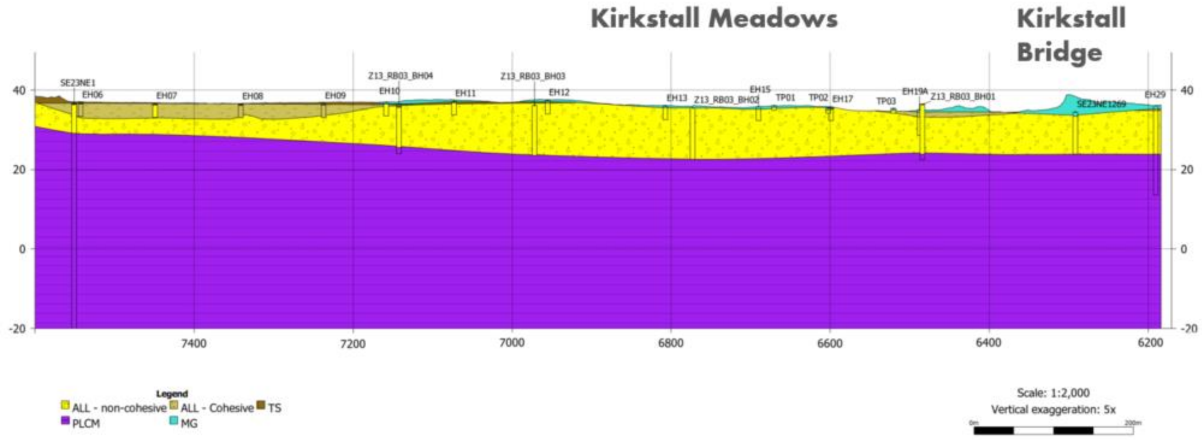

Figure 5: Geological ground section along the River Aire in zone 13a.

4.3 SLS Flux & Design Evolution

The design evolved from a simple earth structure at tender stage, to an earth structure with deep sheet piled cut-off, and back to an earth structure with passive drainage, storage and a pumping station with flap-valve outfalls, as stages of Ground Investigation were carried out and the understanding of seepage risk improved. Figure 6 illustrates a typical section.

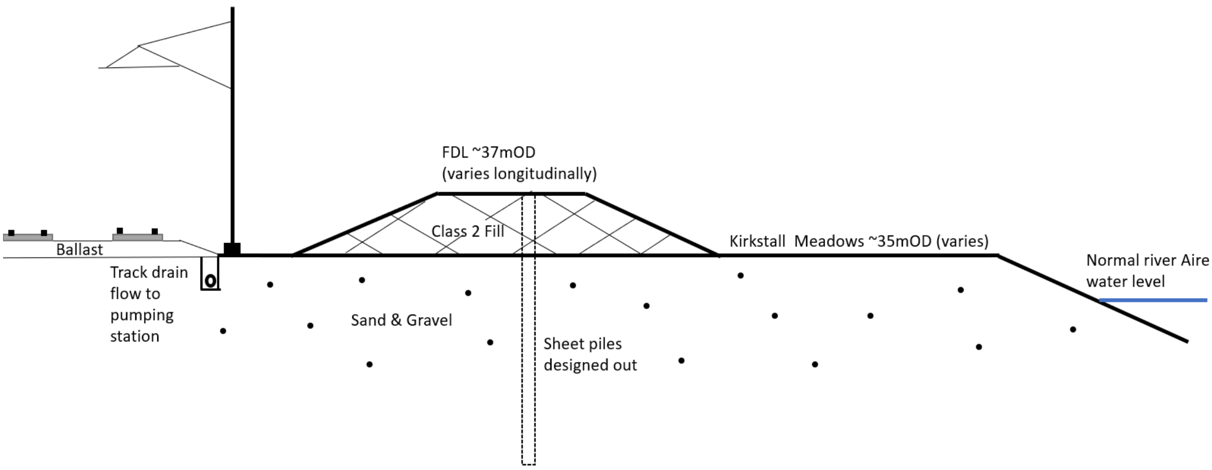

Figure 6: Typical section of new flood defences at Kirkstall Meadows.

4.3.1 Conceptual Design

Feasibility stage Ground Investigation included single hole constant head permeability tests, resulting in a reported Upper Bound soil permeability of the granular alluvium of $2x10^{-1}$ m/s. Single borehole tests are quick to carry out but tend to measure horizontal rather than bulk permeability and can give 'high' values. Seepage analyses considering this permeability indicated a cut-off to rock would be required to prevent unacceptable flow to the land side of the defence during design flood events. At this time steel prices were at record high levels and therefore considerable effort was made to find an acceptable alternative solution.

4.3.2 Soil Anisotropy and Mass Permeability

A literature search indicated that thin silt layers generally present in alluvial drift deposits and natural deposition modes of drift deposits (horizontal layering) result in anisotropy in the soil permeability, with vertical permeability significantly lower than horizontal permeability. For example, Todd et al (2005) suggests that ratios of k_h/k_v usually fall in the range of 2 to 10 for alluvium but can be up to 100 or more when clay layers are present.

The feasibility of a passive drain and pumping station was considered based on an assumed permeability of k_v $1x10^{-3}$ m/s and k_h of $2.5x10^{-3}$ m/s after a review of recorded particle size distributions and case history experience. Additional Ground Investigation was carried out to verify the assumed permeability values, which comprised of monitoring in a river gauge and 4No. bore hole standpipes equipped with electronic level loggers. Level loggers measure and record total pressure (water column equivalent plus barometric pressure), therefore compensation for elevation difference and compensation for barometric pressure variations had to be incorporated to convert the measured data to groundwater level variation in time.

Typically, for seepage assessment of flood defences, the relevant soil properties (permeability, volumetric water content, etc.) and boundary conditions (variation of river water level, rainfall data, etc.) must be defined in order to estimate the response of the numerical model, such as variation of groundwater level, groundwater flow or porewater pressure. The opposite was done for the back analysis at Kirkstall Meadows, where boundary conditions and model response (in the form of groundwater level variation from level loggers) were known, but the material properties were unknown. Inverse numerical modelling was therefore carried out using GeoStudio SEEP/W software by trial-and-error changes to soil permeability values until the resulting predictions (yellow lines) matched the field observations of groundwater level variation during a storm event (green lines).

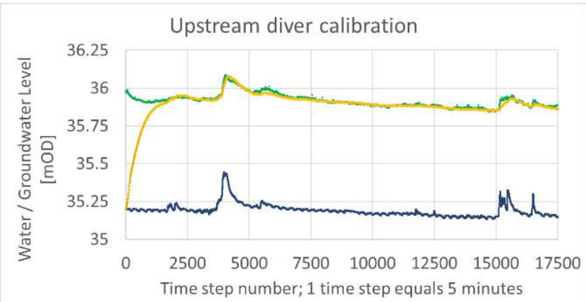

Figure 7: River gauge (blue) and BH diver (green) data enabled SEEP/W back analysis to deduce k_h $1x10^{-3}$ m/s and k_v $2x10^{-4}$ m/s (anisotropy 5).

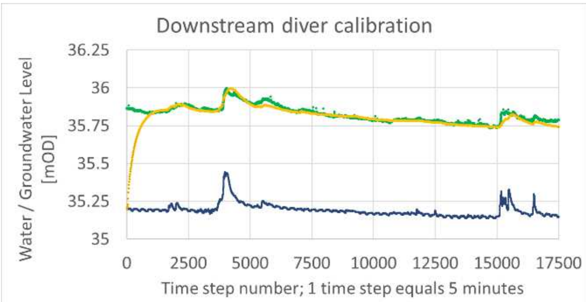

Figure 8: River gauge (blue) and BH diver (green) data enabled SEEP/W back analysis to deduce k_h $2x10^{-3}$ m/s and k_v $2x10^{-4}$ m/s (anisotropy 10).

4.4 Design Completion

The verified ground permeability and associated 2D seepage flow estimates contributed to the 4D combined ground and surface water model, which in turn enabled pipe and pump sizes to be established.

The maximum residual flood volume is shown to be $4000m^3$, which is the measured storage in the track bed from the upstream end of the Kirkstall Meadows to the new pumping station at Wyther Drive, approximately 200m downstream of Kirkstall Bridge. The analysis estimated that the required pump rate to mitigate estimated seepage flows during a 1 in 50-year event, as per the minimum performance agreed with Network Rail, would be approximately $0.25m^3/s$.

The tender design surface water pumping station was able to accommodate groundwater flows for the design flood events without change to the land footprint required or maintenance regime. The wet well comprises of a

4.5m diameter 6m deep caisson with twin pumps and a twin 250mm diameter outfall pipes discharging water through existing 'Under Track Crossings' and into the river via flap valves that open below water level when discharge is pumped under high pressure.

The solution provides a significant improvement to the current level of protection and is in compliant with the operational level of risk as defined by the NR design guidance for surface water drainage. In extreme events, the track may need to close for a very short period of time until the pumps are able to bring the flood water down to a safe level. During these rare events there are numerous other risks posed by flooding such as track electrical failures, scour or landslides that may impact the train operations beyond the project area therefore the design pump and outfall pipe size is unlikely to be the cause of future disruption to rail services.

5. Summary

5.1 Seepage design process

This case history illustrates the iterative process of preparing an initial conceptual ground model, carrying out initial seepage analysis, further Ground Investigation and ground water monitoring, and back analysis of the model before reliable design flows can be established for pipes and pumps to be sized, as shown in Figure 9.

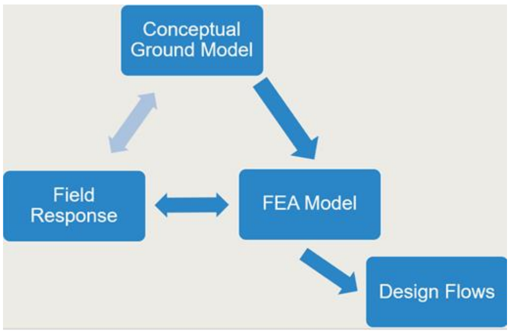

Figure 9: Flow chart of design process

5.2 Establishment of acceptable seepage rates for specific interventions

The assessment of impact of flooding on the railway at Kirkstall Meadows illustrates how establishing an acceptable seepage rate can be complex and the optimum balance of CAPX and OPEX may require agreement with stakeholders outside the core project delivery team.

At another location on this project a shorter length of defence protected a substation from flooding where excess seepage under the earth structure would suddenly cut the power to a large part of the city. Due to the consequences of any 'error' in the measurement of soil mass permeability, 2D seepage or 4D water modelling a sheet piled cut off was provided at this location rather than exploring the potential to manage the risk of seepage by additional ground investigation, analysis and stakeholder engagement.

5.3 Scale of field testing

This paper illustrates the potential error with a single bore hole test data which estimated a permeability some 100 times higher than the subsequent large scale field testing. The required pump rate of $0.25m^3$/sec would have been $17m^3$/s if a design permeability value of $2x10^{-1}$ m/s had been adopted and a single pumping station in the land available would not have been able to provide a feasible solution without a very high-cost seepage cut-off. Estimated costs were £7M for a sheet piled defence without pumping station and £0.5M CAPX plus £1.0M OPEX over a hundred years for the chosen solution.

6. Conclusions

6.1 Verification of published guidance on suitability of different soil permeability tests

The data obtained from the Kirkstall Meadows groundwater monitoring verifies existing published guidance on typical anisotropy in coarse alluvial soils. It also highlights how single bore hole tests are not reliable in sands and gravels. BS EN ISO 22282-2 (2012) advises that variable head test methods are suitable for soil permeability between 10-6 and 10-9 m/s and constant head test methods are suitable for soil permeability between 10-4 and 10-7 m/s. The user error with accurate timing and flow measurement of in-situ permeability testing in sands and gravels can be reduced with electronic measurements such as slug testing with pressure transducer and data logger but the fundamental limitations of only testing a small zone of soil around a single bore hole with bias measurement of horizontal flow can only be addressed by carrying out large scale field monitoring.

6.2 Seepage influences on sliding and bearing limit states

The current approach to design is to use unfactored characteristic values of transient uplift for ULS checks on sliding and bearing resistance of gravity flood walls. Definitions of characteristic values enables the designer to consider the accuracy of the values and consequences of these being incorrect in order to achieve the right level of overall safety in design. This paper applies the guidance in Ciria C731 to adopt a range of permeabilities rather than a single characteristic value where the quality of Ground Investigation is limited. It also highlights the importance checking the influence of seepage on other limit states, which will be clearly stated in EN 1997-3.

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