Several Relief Well Design Considerations for Dams and Levees

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Abstract: Seepage beneath flood protection structures can be controlled by established measures including impervious riverside blankets, cutoff walls, landside drainage trenches, seepage blankets, and relief wells. While different approaches exist for designing and evaluating relief well systems, they can be somewhat cumbersome, and computed solutions are sensitive to a number of parameters. This paper presents an overview of several concepts critical for analyses and illustrative examples from work performed at two flood protection projects (Portsmouth, Ohio and Lawrenceburg, Indiana). Appropriate effective and total stress safety factor formulations for evaluating vertical seepage-related heave and uplift potential are presented. The importance of considering well elevation- and efficiency-related aspects is discussed, and an example of well rehabilitation with transmitting capacity assessment is presented. A practical method for considering finite well line effects during system design using plan view finite element modeling is shown. Additionally, some proposed corrections to design equations presented in the widely utilized U.S. Army Corps of Engineers (USACE) relief well design manual (EM 1110-2-1914) are presented.

Relief Well Design Methods

Relief well systems are designed with the objective of reducing foundation seepage pressures to a tolerable level. Early design requirements for seepage control measures were published in TM 3-424 (USACE, 1956), and Sills and Vroman (2007) discuss the development of USACE levee seepage criteria with time. Regardless of the requirements adopted for a given project, well system design regularly focuses on providing a certain safety factor against seepage-related erosion initiation; probabilistic well design calculations (Guy et al., 2010) are also beneficial to complete and consider when possible. The establishment of a design safety factor for a project always requires thorough consideration, and the decision along with the actual design can be well informed by a risk assessment involving loading frequency and consequences consideration. An overview of factor of safety calculation methods is presented next, followed by additional well design-related discussions.

Factor of Safety Calculation

While factors of safety against heave and uplift for vertical seepage conditions are not factors of safety against structure failure (progression to failure depends on many

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additional factors), they provide useful information regarding stability and serve as a basis for relief well system (and other seepage control measures) design. Separate effective and total stress formulations have been used to calculate factors of safety, and while effective stress analyses are typically referenced in relief wells-related literature, an overview of both methods is provided here. Prior useful discussions on these methods are contained in Cedergren (1989), Doerge (2009), Duncan et al. (2011), Evans (1955), Lambe and Whitman (1969), McCook (2006), and Pabst et al. (2013); the terms “heave” and “uplift” are used in this paper as they are defined in Pabst et al. (2013).

For cohesionless soil vertical seepage conditions, the effective stress factor of safety \( (FS_e) \) against heave (condition of zero effective stress) is evaluated by comparing the vertical hydraulic (“exit”) gradient \( (i_v) \) to the critical hydraulic vertical gradient \( (i_{cv}) \) as shown in Equation 1. Horizontal exit seepage conditions, which are not covered here, are addressed in Duncan et al. (2011) and O’Leary et al. (2013). For saturated confining soil (vertical) seepage conditions, the effective stress factor of safety \( (FS_e) \) against uplift is formulated in terms of weights and forces, but a mathematically equivalent solution to the gradient-based (heave) formulation is obtained (Equation 1). A difference in reality can exist, as in the cohesionless soil (heave) case there may be little frictional resistance to the erosion of materials, whereas in the confining soil (uplift) case there may be some cohesive resistance. Uplift factor of safety calculation methods do not consider soil strength (a conservative approach), nor do they directly apply to a confining soil having a defect (resulting in a concentrated seepage situation). The total stress factor of safety \( (FS_t) \) against uplift formulation yields a different solution than \( FS_e \) (except at the limit state where they agree). Duncan et al. (2011) caution against using a “misleading” total stress formulation when tail water is above the ground surface, as certain formulations (e.g., Method 2 in their paper) can yield dissimilar \( FS_t \) values when applied to separate situations where solutions should be the same. For example, with a “misleading” formulation, different \( FS_t \) values may result if applied to two cases where all conditions including the excess head value are the same (meaning computed \( FS_t \) values should be the same) but the tail water elevation is simply different between the two cases. A recent review of Doerge (2009) reveals total stress is stated in such “misleading” fashion in Fig. 1 of the paper (mentioned here for reader awareness). The below formulations for effective and total stress factors of safety (Equations 1 to 3) assume tail water at or above ground surface. The formulations and relationships assume saturated soil conditions (typically a reasonable assumption), but the formulations can be modified - for example if it is desired to consider partial confining soil saturation.

\[
FS_e = \frac{i_v}{i_{cv}} = \frac{\gamma_b z}{\gamma_w h_{\text{excess}}} = \frac{FS_e (i_v + 1) - 1}{i_v} \tag{1}
\]

\[
FS_t = \frac{i_v + 1}{i_v} = \frac{\gamma_{\text{sat}} z}{\gamma_w (h_{\text{excess}} + z)} = \frac{(FS_e i_v) + 1}{i_v + 1} \tag{2}
\]

\[
i_v = \frac{(1 - FS_e)}{(FS_t - FS_e)} \tag{3}
\]
where $\gamma_b$ is buoyant unit weight of confining soil (assumed saturated); $z$ is vertical thickness of confining soil; $\gamma_{sat}$ is total saturated unit weight of confining soil; and $h_{excess}$ is excess head above the ground surface (or excess head above tail water elevation if it is above the ground surface). For clarity, if a piezometer has its sensing tip in a pervious foundation soil at the base of a surficial confining soil, and it reads 5.0 feet above tail water (ponded 2.0 feet above the ground surface), then the value of $h_{excess}$ to be utilized above would be 5.0 feet. If tail water immediately receded, then $h_{excess}$ would be 7.0 feet.

Regarding relief well system (and other seepage control measures) design, effective stress methods typically appear in guidance documents and are widely utilized, whereas little guidance exists regarding acceptable total stress factors of safety. Regardless of the analysis approach (whether evaluating $FS_e$ or $FS_t$), the above formulations and Fig. 1 illustrate the relationship at a given gradient between $FS_e$ and $FS_t$. For relief well system design, effective stress analysis is typically utilized and appropriate, although a corresponding $FS_t$ directly calculated or determined using Fig. 1 can be considered if determined to be advantageous. For example, in a situation where $i_v$ is equivalent to 0.5 and the confining layer has an average $\gamma_{sat}$ of 124.8 pounds per cubic foot (corresponding to an $i_{cv}$ of 1.0), values of $FS_e$ of 2.0 and $FS_t$ of 1.33 (with $FS_e/FS_t = 1.5$) can be computed using Equations 1 and 2 or directly obtained from their solutions which are plotted in Fig. 1.

**System Geometry Determination**

The design of relief well systems generally involves determining an optimized combination of geometrical and installation parameters (including well penetration, well spacing, well diameter, well screen, filter characteristics, and well discharge elevation) to provide tolerable foundation seepage pressures. Various methods such as closed form equations, superposition calculations, graphical solutions, blanket theory computations, and 2D and 3D finite element modeling can be employed along with empirical data and engineering judgment to determine what constitutes an appropriate well system for a given situation. Design computations are often based on certain assumptions (such as the boundary conditions, levee and well line being parallel to one another and of infinite length), and when a method’s assumptions do not well represent the conditions being analyzed, an attempt to account for such differences should be made. As EM 1110-2-1914 (USACE, 1992) is widely utilized for relief well system design, some proposed corrections to equations contained in the manual are presented below, followed by an expanded discussion on certain infinite length assumptions and effects for relief well lines.

**Design Equation Corrections**

As mentioned, relief well design is often approached utilizing blanket theory and guided by methods in EM 1110-2-1914 (USACE, 1992); a few proposed corrections to the EM are presented here for reader awareness and application.

First, with regard to the theoretical well factors solutions developed by Barron (1982) and presented in Table 5-1 of the EM, the $\Delta \theta$ value for W/D (100%) is incorrectly listed as 1.0; $\Delta \theta$ is the change in $\theta_a$ and $\theta_m$ per one(1) log cycle of $a/r_w$. As mentioned in Guy et
al. (2010) the value should instead be listed as 0.3665, and the basis for this is documented here for reference. From Equation 5-18 of EM 1110-2-1914:

$$\theta_{av} = \frac{1}{2\pi} \ln \left( \frac{a}{2\pi r_w} \right)$$  \hspace{1cm} (4)

where $\theta_{av}$ is the average uplift parameter, $a$ is the design well spacing, and $r_w$ is the effective well radius; note that $\theta_a$ and $\theta_{av}$ are both often used in the literature to represent the average uplift parameter. From Equation 4, for a given log cycle of $a/r_w$ (100 to 1000 in this example), the correct $\Delta \theta$ is calculated as:

$$\Delta \theta = \frac{1}{2\pi} \ln \left( \frac{1}{2\pi(100)} \right) - \frac{1}{2\pi} \ln \left( \frac{1}{2\pi(1000)} \right) = 0.3665$$  \hspace{1cm} (5)

While this example computes $\Delta \theta$ using an average uplift parameter-based approach, a similar $\Delta \theta$ value can be computed using a midwell uplift parameter-approach, or determined graphically from Figure 5-8 of EM 1110-2-1914.

Second, this paper recommends revision of the equations for average net head in the plane of wells corrected for well losses ($h_{av}$) and net head midway between the wells corrected for well losses ($h_{m}$) provided in EM 1110-2-1914. As presented, Equations 7-2 and 7-4 of the EM have the average uplift parameter as an exponent; however, it should be multiplying instead, as shown in the equations below:

$$h_{av} = \frac{h\theta_{av}}{S/a + \theta_{av} \left( \frac{S + x_3}{x_3} \right)}$$  \hspace{1cm} (6)

$$h_{m} = \frac{h\theta_{m}}{S/a + \theta_{av} \left( \frac{S + x_3}{x_3} \right)}$$  \hspace{1cm} (7)

where $S$ is the distance from the landside toe to the effective source entry; $x_3$ is the distance from the landside toe to the effective seepage exit; $\theta_m$ is the midwell uplift parameter; and the other terms are the same as defined above. It is recommended that Equations 6 and 7 be utilized rather than the net head Equations 7-2 and 7-4 in EM 1110-2-1914.

Third, it is noted the equation shown within the top stratum of the lower half of Figure 5-4 in EM 1110-2-1914 is in error. In the equation, the numerator $(S+x_3)$ is multiplied by $H$ (net head on the well system); however, the equation should multiply $(S+x_3)$ by $H_w$ (total well losses including elevation and efficiency-related losses). The proposed correction to the EM equation is shown just above the top stratum (labeled “blanket”) in
Fig. 4 of this paper; note that this formulation also includes the term \( x_w \), which is the distance of the well line downstream of the embankment toe (Guy et al., 2010).

Fourth, the y-axis of Figure 7-3 in EM 1110-2-1914 is labeled as RATIO \( H_{mn}/H_{m∞} \), (indicating plotted curves show the ratio between net head midway between the wells - for finite systems relative to an infinite well line); however, the axis label should read RATIO \( h_{mn}/h_{m∞} \) (indicating plotted curves show the ratio between net head midway between the wells corrected for well losses - for finite systems relative to an infinite well line). The basis for this correction is demonstrated by Figure 7-2 of the EM, which illustrates that the net head midway between wells \( (H_m) \) is equivalent to the sum of \( h_m \) and \( H_w \).

Fifth and lastly, as noted in Guy et al. (2010) and repeated here for awareness, the nomograph based on approximate well factors solutions presented as Figure 5-8 of EM 1110-2-1914 does not include the required “pole” point (on the D/a line for \( θ_{av} \)). A nomograph version with the “pole” point included for use is however presented as Figure 60 of USACE (1956).

**Relief Well Design Considerations**

A number of considerations, which may not be explicitly incorporated into the design method(s) being employed, can often be relevant during the design of a given well system. For example, blanket theory-based design methods often include assumptions of a well line of infinite length, a homogeneous and isotropic foundation, and no hydraulic head losses for the wells. As mentioned above, when conditions for the design situation vary significantly from design methodology assumptions, then an attempt to account for such differences should be made. While the scope of this paper is not to discuss every possible significant difference between actual field conditions and design method assumptions, a few examples from work performed at two flood protection projects (Portsmouth, Ohio and Lawrenceburg, Indiana) focused on illustrating the importance of considering finite line effects and total well losses (elevation and hydraulic) are presented next.

**Projects Overview**

The Portsmouth Flood Protection System (Fig. 2) is located along the Ohio and Scioto Rivers in Portsmouth and New Boston, Ohio and was constructed by USACE in the 1940’s. It consists of 20,000 feet of earthen levee and 21,400 feet of concrete floodwall with 12 interior pumping stations. The project’s top of protection elevation is 548 feet; the maximum height is 58 feet (in the Pump Station Number 5 reach); and the 100-year flood level is 536 feet. The project was constructed on a relatively impervious top stratum underlain by a pervious sand and gravel foundation (averaging 30 feet thick). As part of levee certification-related work, the need for uplift pressure reduction (to obtain the target safety factor for the 100-year loading condition) was determined necessary in the Pump Station Number 5 reach. While toe or collector drains and ditches were originally installed along certain sections of the levee, no seepage control measures currently exist in this area. For the selected seepage control alternative, which consists of a relief well system to be installed in the Pump Station Number 5 ponding area, a brief
overview of the design approach (blanket theory and finite element modeling) is presented below.

The Lawrenceburg Flood Protection System (Fig. 3) is located along the Ohio River in Lawrenceburg, Indiana and was constructed by USACE in the early 1940’s. It consists of an 18,300-foot long earthen levee having a maximum height of 44 feet; the top elevation (constructed based on a 1937 flood elevation of 503 feet) is 504 feet; and the 100-year flood level is 490 feet. The levee was constructed on a relatively impervious top stratum underlain by a pervious sand and gravel foundation (averaging 80 feet thick), and 164 (3-inch diameter) pressure relief wells were installed along the landside levee toe. While USACE (1992) indicates the first use of relief wells for uplift pressure reduction occurred during 1942-43 at Fort Peck Dam, the Lawrenceburg relief well system is another early application. The spacing of existing wells ranges from 50 to 150 feet, and their design penetration into the aquifer was to be 20 feet. The relief well system was not designed according to current methods, and recent seepage analyses have determined additional uplift pressure reduction is necessary. To estimate the existing system’s potential for positively contributing to pressure reduction as part of a new seepage control system, hydraulic testing and rehabilitation (via induced resonance) were recently conducted on a sub-set of the existing relief wells. A brief overview of these relief well activities and evaluations is presented later.

**Finite Line Effects**

When effective seepage (entrance and exit) boundary conditions are parallel to a line of relief wells, and impervious boundaries exist at the ends of the line, the system may generally be evaluated mathematically by considering an infinite number of wells. For such conditions the flow to each well and the head distribution around each well are uniform along the line, and there is no substantial component of flow parallel to the line. Whereas dams often have a line of relief wells installed across a valley and terminating at valley walls, well lines along levees rarely terminate at impervious boundaries, and therefore finite line effects are often an important consideration. Under such conditions there is a substantial component of flow parallel to the well system and a non-uniform distribution of well discharges and heads result. Since uplift pressures between, and downstream of, wells in a finite line will exceed those in an infinite line, relief wells can be installed deeper, with reduced spacing, and/or with variable geometry at the ends of a finite line in order to achieve similar uplift reduction as an infinite line. Data are presented in USACE (1963) to evaluate various finite system geometry effects, and different modeling approaches can be employed for this purpose; however, such effects are also often empirically and judgmentally addressed in practice. In this section a brief example of utilizing blanket theory in conjunction with plan view finite element modeling, to evaluate foundation seepage conditions and design an optimized finite relief well system (as further documented in Darko-Kagya and Guy, 2014), is presented within the context of the Portsmouth Flood Protection System project.

Shown in Fig. 4 are the loading (100-year flood level) and geometrical parameters representing the Portsmouth levee Pump Station Number 5 reach (Fig. 2); variables in the figure are as defined in Guy et al. (2010) and below. As indicated in Fig. 4, the effective
seepage entrance is located at the upstream embankment toe, and a relatively impervious landward blanket results in an effective seepage exit several hundred feet downstream of the embankment. An average 30-foot deep pervious sand and gravel foundation exists at this location, and extrapolation of piezometer data with blanket theory calculation indicates greater than 15 feet of excess head would exist beneath the landward blanket for the 100-year loading. For the base condition (no relief wells installed) the computed $F_{Se}$ value at this location is less than 0.5 for the 100-year loading; thus, design of a relief well system to reduce uplift pressures and provide adequate safety factors in this project reach was completed. Using blanket theory methodology as detailed in USACE (1992) and the computer program from Guy et al. (2010), the spacing for an infinite (fully-penetrating) well line to obtain an $F_{Se}$ value of 1.5 (corresponding to an $F_{St}$ value of 1.2 for an $i_v$ of 1.0) for the 100-year loading condition, was determined to be 50 feet.

Given the relatively high landward elevation (and substantially thicker landward top blanket) of project reaches adjacent to the ponding area, relief wells are not required for adjacent levee reaches. However, installation of a few wells along the levee toe in the pump station vicinity alone, at the penetration and spacing computed by infinite well line methodology, would result in an actual $F_{Se}$ (and $F_{St}$) value much less than that which would be achieved by an infinite line. Therefore, plan view finite element modeling was completed to evaluate such effects, and to determine an optimized finite well system geometry for the ponding area reach, which will provide an adequate reduction in uplift pressure.

Presented in Fig. 5A is a plan view finite element model constructed using SEEP/W (GEO-SLOPE International, 2007) with parameters specified equivalent to those utilized in the above-described (infinite relief well line) blanket theory analysis (Fig. 4). While plan view analyses have certain limitations and do not have true three-dimensional capability, in this situation, direct agreement between blanket theory and plan view modeling (in terms of uplift pressures and relief well flows) is obtainable since this design involves fully-penetrating relief wells. A practical approach for utilizing plan view modeling for partial-penetration analysis will be the focus of another publication. For the model in Fig. 5A, constant head boundaries for the 100-year loading condition (pool and tail elevations of 536 and 502.5 feet, respectively) were specified along the southern and northern plan model faces (in the x-direction); boundary distances from the well line were taken as the lengths of equivalent impervious top strata computed using blanket theory. Model faces in the y-direction were represented as impervious boundaries located one-half of the well spacing from wells located at the ends of the line. The lower and upper model faces (in the z-direction) were also impervious boundaries (required condition of this modeling approach) representing confining units below and above the pervious granular foundation. A constant head boundary condition of 505.4 feet was applied to each of the relief well nodes; as discussed below, this value is equivalent to tail water elevation plus total well losses. As Fig. 5A illustrates, the resultant head distribution with an infinite well line is uniform along the line, and there is no appreciable component of flow parallel to the line. For the modeled 50-foot well spacing, heads along and downstream of the well line do not exceed elevation 507 feet, which is the maximum allowable head in the low-lying ponding area reach to achieve a $F_{Se}$ value of 1.5 ($F_{St}$ value of 1.2). Well discharges of 190 gallons per minute (gpm) for...
the modeled infinite line are also uniform, and along with head projections they closely agree with blanket theory results for these load conditions.

After establishing agreement between blanket theory and plan view modeling results, the plan view model was modified to determine a finite system geometry for achieving design objectives in the ponding area reach. Shown in Fig. 5B is the head distribution for only 1 relief well, and this hypothetical geometry simulates a field condition of 2 wells (fully-penetrating with 50-foot spacing) installed just beyond the levee toe in the pump station reach. Note that only one half of the 2 well system in the y-direction is required to be modeled, as head distributions would be identical in east and west directions from the plane of symmetry. The model parameters and boundary conditions are similar to the infinite well line model (Fig. 5A); however, to evaluate finite system effects, the eastern impervious model boundary (in the y-direction) was located an infinite distance (rather than one-half of the well spacing) from the well. The western boundary was kept as impervious, and again, this boundary serves as a plane of symmetry for the ponding area reach analysis. The results in Fig. 5B illustrate that for this situation, there is an appreciable component of flow parallel to the well system and a non-uniform head distribution. As a result, the installation of only 2 wells would allow uplift pressures in the ponding area that significantly exceed (by approximately 2.5 feet) those for an infinite line. Thus, such geometry would provide an FS\textsubscript{e} value of only 1.0 for the 100-year flood level rather than the target value of 1.5. For the case of 3 wells (Fig. 5C) installed in a line with 50-foot spacing (simulating a field condition of 6 wells around the plane of symmetry) a more favorable head distribution with respect to target criteria is obtained. While heads around and downstream of the third well significantly exceed those for an infinite line, this would not be problematic because the ground elevation here is actually much higher than in the ponding area. While the modeled head is slightly higher (0.5 feet) in the ponding area than the allowable head, the results indicate a system comprised of 7 or 8 wells in a line (on 50-foot centers) just beyond the levee toe could meet design objectives. A practical concern with such an approach, however, is that the high top of ground elevations adjacent to the ponding area would require very deep well housings and extensive excavation for lateral collector conduit. Therefore, a system layout further optimized with respect to practical construction concerns and cost was pursued (as discussed below).

Further comparison of resultant head distributions (total head values as measured along the finite element model north-south well line plane) for the above-discussed infinite wells, finite (1 well), and finite (3 wells) cases is shown in Fig. 6. For the infinite case, the net head midway between wells (\(H_{m,\text{infinite wells}}\)), when expressed as a total head equivalent (y-axis value), is seen to be below the allowable maximum head of 507 feet. Note the magnitude of \(H_{m}\) for different modeled scenarios (indicated by vertical arrows in Fig. 6) represents the net head midway between the wells, with \(H_{m}\) equivalent to \(h_{m}\) (Equation 7) plus \(H_{w}\). When added to tail water elevation (502.5 feet in this case), the \(H_{m}\) values can be expressed as total head elevation equivalents (y-axis values). For the finite (1 well) case (Fig. 5B), the net head midway between wells (\(H_{m,1 \text{ well}}\)) when expressed as a total head equivalent significantly exceeds the allowable maximum head of 507 feet in the ponding area region (Fig. 6). For the finite (3 wells) case (Fig. 5C), the net head
midway between wells ($H_{m,3 \text{ Wells}}$) also exceeds the allowable maximum head in the ponding area region, but as mentioned such a geometry is close to an acceptable solution.

As illustrated in Fig. 5D, a finite nonlinear well geometry can be employed in the ponding area reach to achieve similar uplift reduction as that indicated by infinite well line (blanket theory) design computations. By modifying the model used to produce the results in Figs. 5B and C (keeping all parameters similar but modifying well positions and number), a head distribution can be obtained for the low-lying ponding area that is acceptable with respect to the 100-year loading design criteria. By installing a total of 8 relief wells with an approximate 50-foot spacing at similar elevation (exact positions are yet to be determined in the field, but perhaps along the 506 surface elevation contour for example) around the ponding area (Fig. 2B), it is seen in Fig. 5D that the allowable maximum head of 507 feet will not be exceeded downstream of the well system in the ponding area. The geometry of the wells alignment (in a concave shape) is a result of both the finite system effects and topography of the ponding area. Again, note that only one-half of the reach is modeled here, as the results in eastern and western directions would be symmetrical. While project objectives and available funding required focus on meeting target stability for the 100-year event at this time, and the designed well system geometry provides a computed factor of safety that is above the limit state for the top of levee (elevation 548) loading condition, the safety margin would ideally be higher for the project design loading condition (e.g., considering the amount of uncertainty associated with design data and computations). Therefore, residual stability conditions (subsequent to installation of the designed well system) for the design loading case will be further considered during future levee risk assessments, and the well system may be supplemented in the future to achieve a higher safety margin for design loading conditions. The results in Figs. 5 and 6 demonstrate the potential importance of finite system effects during relief well system design and demonstrate a methodology using plan view finite element modeling (with support from blanket theory computations) to consider them. While finite line effects can be considered and addressed by different approaches (as mentioned above), the plan view modeling approach in conjunction with blanket theory can assist in considering such effects as well as in optimizing the geometry of a well system. A total head plot of $H_m$ values (as shown in Fig. 6 for various hypothetical well lines) is not presented for the optimized well system geometry (Fig. 5D) given its nonlinear nature; however, the plan view model head distribution results illustrate that the optimized geometry meets design objectives as stated above for the 100-year loading condition.

**Elevation and Hydraulic Head Losses**

Along with relief well geometry, many other factors have significant effect on the pressure reduction potential of a system. Elevation and hydraulic head loss components of the well screen, riser elevation, and discharge must be considered during design and also monitored in the future. Head losses during the flow of water through these features are important, as they result in a decrease of well discharge, and thus raise the potential along a line of wells by an amount equal to such losses. As examples, if a system has its well risers set too high, or if the screens are allowed to foul over time via lack of
maintenance, lower system discharge efficiency and higher foundation heads (along and beyond the well line) will result.

For the Portsmouth Pump Station 5 ponding area, the optimized well system geometry (Figs. 2B and 5D) was designed based on an assumed tail water elevation of 502.5 feet (ponded water located above the prevailing low ground elevation of 490 feet). For the same model, lowering the tail water elevation to 500 feet while keeping all other input parameters the same would reduce \( FS_e \) from 1.5 to 1.0 (and would reduce \( FS_t \) from 1.2 to 1.0), whereas raising the tail water above 502.5 feet would increase the \( FS_e \) value. As tail water is often an important random variable, a condition of the optimized well system design is that a minimum tail water elevation of 502.5 feet must be maintained in the ponding area during a 100-year loading condition. This should be maintained for higher loadings too (e.g., in order to achieve an \( FS_e \) value above 1.0 for the top of levee loading condition). While the required tail water can be maintained without adverse implications according to project personnel, its elevation is expected to frequently rise above elevation 504 feet (and possibly greater); therefore, potential back-flooding and optimal riser elevation were important considerations during system design. For the optimized system, well housings will be installed along the 506 foot surface elevation contour (for example, exact positions are yet to be determined in the field) of the ponding area, and well risers will be installed at elevation 505 feet. Check valves will also be installed on each riser. This approach achieves a balance between locating the wells too low (such that they are constantly inundated) and too high (such that high elevation head losses become excessive). For the designed riser elevation of 505 feet, elevation head loss is equal to 2.5 feet (riser elevation minus tail water elevation) for the 100-year loading. If the risers were located at 508 feet instead for example, and tail water was located at 502.5 feet, elevation head loss would be increased by 3 feet (relative to risers at 505 feet), the foundation potential along the well system would be raised by 3 feet, and a much lower \( FS_e \) would therefore result. Raising the tail water to 507 feet (with risers at 508 feet) would reduce uplift potential with respect to a lower tail water case; however, the \( FS_e \) would still be below design criteria.

Together with elevation head loss, hydraulic (efficiency-related) head losses were also considered during well system design for the Portsmouth ponding area. In practice, efficiency-related losses have historically been considered by estimating entrance, friction, and velocity losses for a system and then adding these to the maximum computed landside head. Note that design equations such as those in USACE (1992) generally assume well-related losses equivalent to zero, and therefore, they must be separately estimated and incorporated. Well-related losses can also be measured in the field via pump testing after well installation, and/or estimated and incorporated into design in a manner that (at least in theory, although sometimes difficult in practice) will allow post-installation measurements of well performance to be related to original design assumptions and \( FS_e \) values (Guy et al., 2010). For the Portsmouth ponding area design, a well efficiency value of 75 percent (theoretical versus actual drawdown for 100-year loading discharge) was estimated based on regional knowledge and judgment (75 percent efficiency corresponds to estimated hydraulic well losses of 0.4 feet). As with tail water and many other factors, well efficiency is also an important random variable, with the computed \( FS_e \) value ranging from 1.5 to 1.2 for the efficiency range of 75 to 50 percent.
Therefore, it is important that actual efficiency be measured (via pump testing) during wells installation to ensure it is equal to or above the design efficiency, and that actual efficiency also be monitored (and maintained as necessary) on a recurring schedule in the future to ensure it does not problematically decline. For the Portsmouth ponding area system, the total well losses were computed to be 2.9 feet for the 100-year loading condition (the sum of an elevation head loss of 2.5 feet and hydraulic head losses of 0.4 feet). The total well losses were incorporated into blanket theory design computations (Fig. 4) in a manner discussed in Guy et al. (2010). For the finite element modeling (Fig. 5) losses were included in the total head specification for each relief well. A constant head value of 505.4 feet was assigned to each well, and this value represented the tail water elevation plus the estimated total well losses. As elevation and efficiency head losses affect the net head on the well system (and vice versa with regard to efficiency), they effect the average net head in the plane of wells and net head midway between wells (parameters governing design). Therefore, it is important to consider total well losses during design and monitor/maintain efficiency-related well losses into the future. An example of well losses and well performance assessment is presented next using data acquired at Lawrenceburg Levee.

**Well Performance Assessment**

For the Lawrenceburg Flood Protection System (Fig. 3), underseepage analyses for different project reaches yielded FS<sub>e</sub> values ranging from 0.6 to 2.2 for the 100-year flood loading condition (pool elevation of 490 feet). Since FS<sub>e</sub> values along two project reaches were found to be below typical design standards, 29 new (10-inch diameter) relief wells, with designed penetrations ranging from 25 to 80 percent, have been designed for future installation in these reaches (Thelen Associates Inc., 2014). The design of the new wells was conducted in accordance with USACE (1992) and to achieve a design loading (pool elevation of 503 feet) maximum exit gradient (i<sub>v</sub>) of 0.5, which in this case (where i<sub>cv</sub> = 1.0) corresponds to an FS<sub>e</sub> of 2.0 and a FS<sub>t</sub> of 1.3 (Fig. 1). General information on the Lawrenceburg Flood Protection System is provided above.

To estimate the existing (early 1940’s) relief well system’s potential for positively contributing to seepage control as part of a newly designed system to achieve target factor of safety values, hydraulic testing before and after rehabilitation (via induced resonance technology) was performed for 7 of the existing (3-inch diameter, perforated wrought iron pipe) wells. As mentioned, Lawrenceburg Levee was constructed on a relatively impervious top stratum underlain by a pervious sand and gravel foundation (ranging from 59 to 94 feet and averaging 80 feet). The screen lengths of the wells selected for evaluation range from 4 to 19 feet, and their penetration ranges from 6 to 27 percent (Table 1); note that one of the inspected wells (RW-151) was actually found through evaluation to have no screened portion penetrating the aquifer. In terms of diameter and intake area, the existing wells have smaller diameter than those typically installed in practice today (6 to 18 inches), and their screen intake areas (ranging from 3.7 to 4.8 percent, and averaging 5 square inches of open area per foot of screen) are much less than those of modern continuous wire-wrapped screens. Frictional resistance encountered by flow being concentrated in getting to, and through, relatively small screen openings, as well as associated with flow upwards through relatively small diameter pipe,

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is capable of reducing relief well discharge (and thus increasing potential along the well line). Laboratory experiments by Clark and Turner (1983) indicate that entrance velocity into a well is governed by intake area up to a value of about 10 percent, and that the performance of screens deteriorates with a decrease in open area below this value due to increasing well losses. Since increased velocity generally leads to increased turbulent losses, design methods (USACE, 1992) often recommend limiting intake velocity to 0.1 feet per second (ft/sec). While Clark and Turner (1983) did not measure significant well losses at entrance velocities up to 0.5 ft/sec, they still recommend designing for a maximum velocity of 0.1 to 0.25 ft/sec as a conservative measure. With respect to these often-recommended design criteria, the projected entrance velocities for the existing Lawrenceburg wells significantly exceed them (Fig. 7); entrance velocities were calculated by dividing discharge rate by the well screen open area. Irrespective of hydraulic testing, the limited penetration, small diameter, low open area, and projected high entrance velocities suggest that well losses associated with the existing Lawrenceburg relief wells will have a negative effect on pressure relief potential for design loading conditions.

The hydraulic testing and rehabilitation program for the 7 existing Lawrenceburg relief wells consisted of video logging, pre-rehabilitation step-drawdown testing, rehabilitation using induced resonance technology, and post-rehabilitation step-drawdown testing. Based on video logging and observations during the prior abandonment/extraction of several existing project wells, the existing wells have likely experienced performance decline over time due to silting and bio-fouling mechanisms, although historic discharge data do not exist to further determine an extent of decline. In addition to well losses associated with original well system construction details (discussed above), additional meaningful well loss components are therefore possible during design discharge rates due to a reduction in well permeability associated with clogging. The before and after rehabilitation step-drawdown tests on existing wells were generally performed at the pumping rates of 5, 10, 15, and 30 gpm with a minimum duration of 60 minutes for each step. Although it is advantageous to conduct hydraulic testing at anticipated design discharge rates (ideally the discharge rate used should sufficiently stress the entire aquifer) to reduce uncertainties associated with estimating well performance parameters at discharge values higher than those used during testing, the small diameter of existing wells prevented the use of higher capacity pumps in this case. The rehabilitation work utilized induced resonance technology (“hydropuls”) in combination with pumping. The hydropuls method was performed with a high pressure nitrogen pulsing unit equipped with a pressurized hose and valve system. During the process, the impulse generator was placed in the well screen, and it released impulses of high pressure nitrogen in short, repetitive bursts. More than ten cycles (one cycle being a full pass of entire well screen up and down) of the impulse action were made for each well. The impulse actions created an air-lift effect, which in turn vibrated and loosened existing incrustation- and biofouling-related debris, such that it could be removed from the well by mechanical air-lifting and over-pumping.

Generally speaking, the total drawdown in a pumped well consists of two components: the aquifer losses and the well losses. Aquifer losses are time-dependent, and their variation is linear with time, whereas well losses are divided into linear and nonlinear
head losses. The linear total well loss potentially has several important components, including aquifer loss and other losses attributable to losses through the filter and well screen or losses associated with partial well penetration. During data analyses, if the specific capacity measured during testing is constant with increasing well discharge, then laminar flow is assumed; whereas if specific capacity decreases as discharge increases, then turbulent conditions are inferable. If an increase in specific capacity occurs during step-drawdown testing, then this usually suggests: that the well was previously underdeveloped; that development is occurring during testing; and that well loss parameters are not inferable from the data.

Shown in Fig. 8 (top) are step-drawdown data acquired before and after the rehabilitation of well RW-85 at Lawrenceburg Levee; data for the other site wells, which were evaluated, are contained in Thelen Associates Inc. (2014). Analyses of the data shown in Fig. 8 (bottom) were performed using the Hantush-Bierschenk method (Kruseman and de Ridder, 1994), which is based on Jacob’s (1947) equation. This method of analysis involves plotting specific drawdown (s/Q) versus Q, where s is the stabilized drawdown at the end of each step and Q is the step’s pumping rate. As seen in Fig. 8, the diagnostic plot data for RW-85 generally exhibit straight lines having positive slope (indicating specific capacity is decreasing with increasing Q). Therefore, the linear (B) and nonlinear (C) loss coefficients were able to be estimated using:

\[ s = BQ + CQ^2 \]  

(8)

While total linear losses could be estimated from acquired data, it was not possible to reliably estimate individual linear loss components (e.g., those potentially caused by well skin and partial penetration effects in addition to aquifer loss) without having results from constant rate pump testing (providing transmissivity and storage coefficient values). Thus, calculation of a theoretically comprehensive well efficiency (expressed as a ratio of aquifer head loss to total head losses) was not possible, and estimates of Driscoll’s \( L_p \) parameter (expressed as a ratio of laminar head loss to total head losses) were made as follows:

\[ L_p = \left( \frac{BQ}{BQ + CQ^2} \right) (100\%) \]  

(9)

While \( L_p \) values provide useful well performance information, they will usually overestimate well efficiency, and they are often erroneously represented as well efficiency values in practice (Boonstra, 1999). To potentially relate field hydraulic testing results to underseepage safety factor values in a theoretically correct manner for relief well design (Guy et al. 2010), the individual components of the linear total well losses must be determined (ideally at design discharge), such that well efficiency (rather than \( L_p \)) can be computed and used in projecting uplift pressures; this will be the topic of a separate paper.

A summary of measured and projected well performance parameter values based on hydraulic testing for well RW-85 is contained in Table 2; the linear and nonlinear
coefficients measured from hydraulic testing are shown on Fig. 8 (bottom). From a comparison of the C coefficients before and after rehabilitation, an improvement in well condition due to rehabilitation is apparent. However, according to the criteria of Walton (1962), the degree of well deterioration is still “severe” after rehabilitation, and with a post-rehabilitation C coefficient value of greater than 40 sec²/ft² (greater than 0.01 min²/ft²) it is “difficult and sometimes impossible” to restore original well capacity; Kasenow (1998) indicates that well capacity can be difficult to restore when the C coefficient value is greater than 0.0002 ft/gpm². The computed Lₚ values also illustrate well improvement due to rehabilitation; however, the values also clearly indicate increasingly poor well performance and related head losses as discharge increases, suggesting limited capacity of the well. The specific capacities were increased by rehabilitation, but even afterwards they are still an order of magnitude lower than those of many other relief wells in similar (glacial outwash) foundation materials that the authors have previously tested. Key general differences between the Lawrenceburg wells and these other wells are that the other wells have larger diameters, larger screen intake areas, and larger effective foundation penetrations. Without having accurate estimates of transmissivity and storage coefficient values from constant-rate testing, and without being able to step-test the Lawrenceburg wells at higher discharge rates often employed for this foundation-type (permeability on the order of 0.1 cm/s), inferences regarding the degree of partial penetration losses to be expected at higher discharge values could not be inferred. However, as the penetrations of all site wells are quite limited (Table 1), it would be expected that partial penetration-related losses could be measurable and significant if testing were performed with an order of magnitude increase in discharge.

For this reason and others discussed above, there can be large uncertainty associated with attempting to project Lₚ (or well efficiency) values very far upwards beyond the highest field step. In the case of the Lawrenceburg wells, while direct reliable estimation of head losses at design discharge rates was not possible (often the case for most situations in practice), useful judgment regarding capacity of the existing wells at design loading of the levee was able to be made by considering projected Lₚ values in conjunction with blanket theory-based well design methodology. By incorporating projected Lₚ values into the well design process (Guy et al., 2010), head losses for the Lawrenceburg wells at levee design loading conditions were able to be estimated. However, it is recognized that there is uncertainty in predicted head losses for the above-discussed reasons, and that because Lₚ overestimates well efficiency, head losses and uplift pressures would be somewhat underestimated using Lₚ. From such estimates resultant uplift pressures from the existing well system were able to be predicted with consideration of well losses (a substantially better approach than one that assumed no well losses, which would be incorrect), and an overall estimate of the potential contribution towards foundation pressure reduction from the existing well system was made. These analyses were useful in determining that the existing Lawrenceburg wells, while they will not provide target stability for design loading conditions, do have some limited transmitting capacity, and will therefore provide some limited future benefit.

The above discussion regarding the testing and evaluation of the Lawrenceburg relief wells demonstrates the importance of considering well losses and efficiency concepts for both existing and new relief well systems. In the case of the existing Lawrenceburg wells, hydraulic testing has indicated that high well losses can be expected during
discharge at design loading conditions, and as well losses directly increase foundation uplift pressures, the existing system is not sufficient alone to meet target safety factors. As existing wells are structurally sound and will provide some discharge capacity though when the levee loading increases, they will still remain and provide some project benefit after installation of the (above-mentioned) newly designed Lawrenceburg relief well system.

**Conclusions**

This paper has provided discussion and illustrative examples concerning several concepts critical for relief well system analyses which the authors hope practitioners will find useful. Appropriate effective and total stress safety factor formulations for evaluating vertical seepage-related heave and uplift potential have been presented, and as mentioned, design decisions can be well informed by a risk assessment involving event tree, loading frequency, and consequences consideration. Some proposed revisions to design equations contained in EM 1110-2-1914 (USACE, 1992) have been presented and are recommended for future application. A practical method for considering finite well line effects during system design using plan view finite element modeling has been demonstrated, and the importance of considering well elevation- and efficiency-related aspects has been discussed. While such factors may not be explicitly incorporated into existing design methodologies, they can have a significant effect on well discharge and resultant groundwater head distribution, and thus a project's underseepage stability. Therefore, when conditions for a design situation vary significantly from design methodology assumptions, an attempt to account and design for such differences should always be made.

**References**


Fig. 1. Relationship between effective stress ($F_{Se}$) and total stress ($F_{St}$) factors of safety against erosion initiation (for vertical seepage conditions) as a function of critical hydraulic vertical gradient ($i_{cv}$) and actual vertical hydraulic gradient ($i_v$).
Fig. 2. Representative aerial photograph (A) of the Portsmouth Levee Pump Station Number 5 reach (looking northeast), and layout of a finite relief well system designed to reduce uplift pressures during the 100-year loading condition (B).
Fig. 3. Layout of existing relief wells superimposed on an aerial photograph of the Lawrenceburg Flood Protection System. The spacing between Well No. 1 through Well No. 18 is 100 feet; the spacing between Well No. 18 through Well No. 60 is 50 feet; the spacing between Well No. 60 through Well No. 74 is 150 feet; the spacing between Well No. 74 through Well No. 126 is 100 feet; and the spacing between Well No. 126 through Well No. 164 is 150 feet. See text for discussion.
Fig. 4. Conceptual diagram illustrating relief well system design parameters for an infinite well line; variables are as defined in the text and Guy et al. (2010). Load conditions and parameter values indicated were used for the Portsmouth Pump Station Number 5 system design. This drawing is not to scale and plotted head curves are for illustration purposes.
Fig. 5. Resultant head distributions for fully-penetrating relief well system geometries (computed by plan view finite element modeling) in the Portsmouth Pump Station Number 5 reach (Fig. 2A). Loading conditions represent the 100-year flood and modeled geometries include: A) an infinite line of wells with 50-foot spacing; B) 1 well simulating a field condition of 2 wells around the lower model symmetry plane; C) 3 wells simulating a field condition of 6 wells; and D) an optimized relief well system layout consisting of 4 wells simulating a field condition of 8 wells (Fig. 2B). See text for discussion.
Fig. 6. Variation of total head along infinite and finite well lines modeled for the Portsmouth Levee Pump Station Number 5 reach. The infinite, finite (1 Well), and finite (3 Wells) curves correspond to node values measured along the well lines in Figs. 5A, B, and C respectively; the intermediate model used to produce the finite (2 Wells) curve is not presented in Fig. 5. Values of $H_m$ represent net head midway between the wells and are expressed as total head elevation equivalents (by adding $H_m$ values to the tail water elevation of 502.5 feet). Values for net head midway between the wells corrected for well losses ($h_m$) and total well losses ($H_w$) are also indicated; recall that $H_m = h_m + H_w$. The allowable total head in the ponding area just downstream of the well line for the 100-year loading condition is 507 feet. See text for further discussion.
Fig. 7. Projected well entrance velocities versus discharge rate for existing Lawrenceburg relief wells (Fig. 3). The intake area per foot of screen varies from 4.2 to 5.5 square inches corresponding to open areas of 3.7 to 4.8 percent (Table 1). Entrance velocities were calculated by dividing discharge rate by open area; note results for Well No-131 and Well No-152 are similar, and results for Well No-151 are not presented because the well screen was found to not penetrate the aquifer.
Fig. 8. Step-drawdown data acquired before and after rehabilitation for Lawrenceburg relief well RW-85 (top) with step-drawdown diagnostic plots and computed B and C loss coefficients (bottom). Values of s/Q represent specific drawdown.
Table 1. Characteristics of existing Lawrenceburg relief wells (Fig. 3) that were evaluated. Note that well RW-151 was determined through inspection to not penetrate into the aquifer; this well pumped dry at a testing discharge rate of 10 gpm.

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<th>Lawrenceburg Levee Inspected Existing Relief Well Characteristics</th>
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Table 2. Measured and projected performance parameters for Lawrenceburg relief well RW-85, based on step-drawdown testing before and after rehabilitation. \( L_p \) represents computed Driscoll’s parameter values (ratio of laminar head loss to total head loss), \( s_1 \) and \( s_2 \) represent drawdown values computed from the linear (B) and nonlinear (C) well loss coefficients (Fig. 8), and \( Q/s_1 \) (discharge per total drawdown) is specific capacity computed from B and C coefficients. See text for discussion.

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