Typical Levee Sections and Geotechnical Considerations

Chehalis Basin
Lewis County, Washington

for
Moffatt & Nichol

June 5, 2025

1101 Fawcett Avenue, Suite 200
Tacoma, Washington 98402
253.383.4940

GeoEngineers

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Chehalis Basin
Lewis County, Washington

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Prepared for:

Moffat & Nichol
600 University Street, Suite 610
Seattle, Washington 98101

Attention: Emy Carpenter, PE

Prepared by:

GeoEngineers, Inc.
1101 Fawcett, Suite 200
Tacoma, Washington 98402
253.383.4940

Garrett D. Timm
Geotechnical Engineer

Lyle J. Stone
Principal Geotechnical Engineer

GDT:LJS:lmm

GeoEngineers

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1.0 Introduction

The City of Chehalis (City) is considering a system of flood prevention levees to mitigate the flood risk within the greater Chehalis River Basin. The City would like to use or expand existing flood prevention structures throughout the cities of Chehalis, Centralia, and Adna. They are also considering the construction of new structures such as earthen embankments and flood walls. This strategy is being proposed to reduce flood damages during the extreme flood event without the need for a new dam in the upstream portion of the basin.

Over the last several years, many studies have been completed across the basin to evaluate the environmental impacts associated with various flood mitigation strategies, and those studies have been summarized in the 2023 preliminary report *Chehalis Basin LAND (Local Actions Non-Dam Alternative)*. According to this report, eight levees positioned across the Chehalis River Basin within both rural and urban environments are proposed. The combined length of the proposed levee system will be approximately 126,600 feet, or 24 miles. The predicted water levels against the levees across the basin during notable flood events varies between very little or none (i.e. only providing freeboard), to greater than 15 feet above existing grade. A vicinity map (Figure 1) showing the location of proposed levees and other improvements is attached.

The following memorandum is intended to provide the Moffatt & Nichol team with a brief outline of example typical levee sections annotated with details specific to the Chehalis LAND Alternatives Levee analysis, as well as a summary of where each structure is appropriate and could be implemented.

We previously delineated levee sections by type to identify what portions of which levee are appropriate for each typical section. Road raising, earthen embankments, embankment turnouts and ramps, riverbank protection, and walls were considered. Typical section examples were derived from various engineering manuals and past projects completed by GeoEngineers using United States Army Corps of Engineers (USACE) guidelines. This information was incorporated with other data by the Moffatt & Nichol team to develop the preliminary alignment and assessment of anticipated levee types. These alignments are shown in Figures 2 through 4. Levee Structure and Geology.

2.0 Design Standards

We understand that the levee system will be designed so that it can be accredited by the Federal Emergency Management Agency (FEMA) for modifying flood insurance rate maps. We also understand that the system will be enrolled in the USACE Public Law 84-99 program for levee maintenance and repair. These programs allow for the system to be used to lower flood insurance rates and to be eligible for federal funds for levee repair.

The requirements for the National Flood Insurance Program (NFIP) are set forth in the Code of Federal Regulations (CFR) at Title 44, Chapter 1, Section 65.10 (44 CFR Section 65.10).

While FEMA and the NFIP do not specifically require following USACE design standards, these standards are widely accepted and should serve as the baseline for all engineering design. Any proposed deviations from these standards should be well thought out, documented, and justified based on sound engineering principles.

Items from 44 CFR Section 65.10 that should be considered at this preliminary assessment stage include the requirements for freeboard (see excerpt below):

Riverine levees must provide a minimum freeboard of three feet above the water-surface level of the base flood. An additional one foot above the minimum is required within 100 feet in either side of structures (such as bridges) riverward of the levee or wherever the flow is constricted. An additional one-half foot above the minimum at the upstream end of the levee, tapering to not less than the minimum at the downstream end of the levee, is also required.

The NFIP also requires that all sections used for flood control can be accessed and maintained by the owner and operator of the levee system. This means that the levee system must be located on public right of way under the jurisdiction of the levee owner, or that access and maintenance easements are established with other property owners. Where a levee crosses or encompasses public property such as a state highway that is not within the jurisdiction of the levee owner, sometimes franchise agreements, similar to that those used for utilities, can be used to provide the required access permissions.

The USACE PL 84-99 program does require maintenance to be performed and access provided in accordance with some USACE standards. The appropriate standards are included and referenced in this memorandum.

3.0 Geologic Setting

A hydrogeologic report of the Chehalis River Basin was completed in 2011 by Andrew Gendaszek of the United States Geological Survey (Gendaszek, 2011). This report identified five hydrogeologic units and a confining bedrock unit across the basin. Generally, these units consist of alluvial and glacial outwash deposits due to the proximity of the basin to the southern extent of the Puget Lobe of the Cordilleran Ice Sheet. The youngest lobe of the icesheet extended into the north portion of the basin, so shallow glacial till is not likely to be encountered in our site. A description of each geologic unit can be found on the geologic maps cited below in the levee sections geologic unit summary table.

The entire drainage basin is approximately 2,700 square miles and includes dozens of smaller creeks including those located within Chehalis and Centralia that may be impacted by this project (Skookumchuck, Salzer, and China creeks). The site area receives a mean annual precipitation equal 43 to 50 inches, which is more than anywhere else in the greater hydrologic basin, indicating that evaluation of proper drainage systems will need to be considered to evaluate water ponding within the interior of the levee system during a combined storm and high water event.

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3.1 GEOLOGIC HAZARDS

The site area is relatively flat therefore landslide hazards are expect to be minor. However, seismic risk may be significant. As shown on Exhibit 1, the Centralia Quadrangle lies within a region which is strewn with faults, including the Scammon Creek Fault, Salzer Creek Fault, Chehalis Fault, Kopiah Fault, and Doty Fault. A closer look at the intersection of actual faults and proposed and existing structures is warranted. Although Adna does not have mapped faulting, it is likely that the structures in that area will also be impacted by nearby fault ruptures.

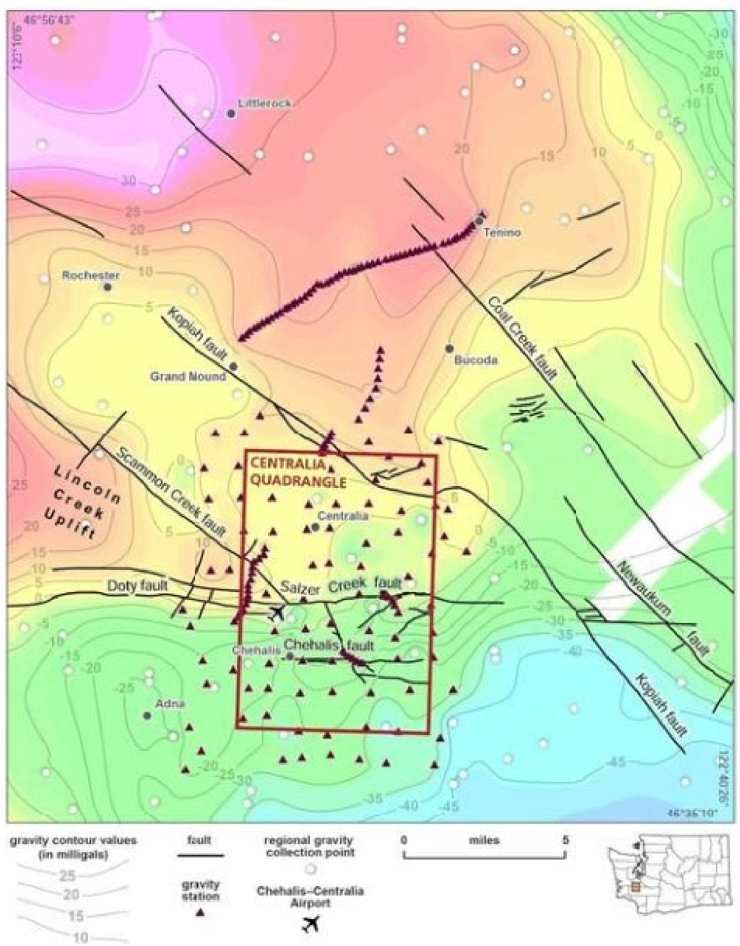


Exhibit 1: Gravity Anomaly Map of Centralia Quadrangle (Sadwoski et al. 2018)

Not all levees include seismic loading in design. Those that do are typically designed for 100-year return periods or to the same level of the design hydraulic protection. When considering seismic performance of levee systems, USACE guidance allows considering maintenance and repair, specifically considering "the capability to repair all earthquake-damaged reaches prior to the next flood event" rather than seismic structural resilience. Any levee that is located near a riverbank will need to consider the potential for seismic lateral spread and large seismic deformations.

3.2 ANTICIPATED GEOLOGIC MATERIALS

Our understanding of geologic materials at the site is based on review of geologic maps as well as review of select reports from our inhouse files and from publicly available sources such as the Washington State Department of Transportation (WSDOT) geotechnical boring database. The boring locations we reviewed and considered applicable to the project are shown in Figure 5, Reviewed Public Subsurface Data. The boring logs are included as Appendix A, Public Subsurface Data.

Geologic maps, for reference, are included as part of Figures 2 through 4. The proposed levee alignments are located predominately in areas mapped as alluvium (Qa). Alluvium is defined as any geologic material deposited by flowing water. As such, the composition of alluvium can vary greatly. Geologic maps of the area describe the alluvium as containing "pebbles, cobbles, sand, silt, clay, peat, and boulders in varied amounts".

Based on our review of select explorations, locally the alluvium is observed to consist of an upper layer of loose or soft fine grained alluvium (silt and clay) and a lower layer of relatively denser coarse grained alluvium (sands and gravels). The thickness of the upper layer is variable but observed to generally be 15 to 25 feet thick.

Portions of alignments extend into areas mapped as various types of glacial drift. Glacial drift is defined as geologic materials deposited by a glacier. Based on our review of select explorations this material can consist of dense to very dense sand and gravels with silt.

3.3 RIVER CONVEYANCE AND POTENTIAL LEVEE FILL SOURCES

We understand that improved river conveyance is being proposed to widen the Chehalis River through a portion of the project area, as shown in Figure 2. The conveyance improvements will require excavation and removal of approximately 800,000 cubic yards of soil. We understand that the volume of soil estimated to be required for levee construction is approximately 600,000 cubic yards. The excavation for the river conveyance could be considered for beneficial use as a source of fill for the levee.

The upper layers of alluvium are expected to be fine grained and therefore will be moisture sensitive. In order to function as practical levee fill, the material will need to be placed at or near the optimum moisture content for compaction. This will likely require some drying of the material as, at least some portion, is expected to be derived from below the natural groundwater level.

Excavation of the river conveyance area will need to be staged and coordinated with considerations for reusing the soil as fill. This will likely require staging the excavation in a way that keeps the portion closest to the river in place to act as a temporary dike and prevent occasional high-water events from inundating the excavation area. It will also likely require using a large portion of the site to process and dry soil to the optimum moisture content before it is loaded and transported to fill areas.

It is an advantage that only about three quarters of the excavated volume is required for levee construction. This means that the wettest or least suitable quarter of the material encountered can be discarded and does not need to be processed for use as fill. This does mean that a disposal site for approximately 200,000 cubic yards of material unsuitable for levee will need to be identified. In our opinion this material could potentially be used in planting berms or other non-structural areas.

In addition to the bulk levee fill, the project will also require more processed sand and gravel fill material for levee access road surfacing, drainage layers, and backfill around select structures. It is possible that the underlaying coarser alluvium could be used for these purposes or for more robust structural fill. Excavating deeper than the planned bottom of the conveyance would, of course, be additional effort both for excavation and replacing the material, but it could be an overall benefit if the deeper alluvium produces a higher quality fill material.

4.0 Typical Levee Sections

4.1 ROAD RAISING

Typical road raising as described in USACE Engineering Manual (EM) 1110-2-1913 (USACE, 2000) consists of the removal of the upper 18-inches of existing road grade fill benched at a maximum bench height of 2 feet, with a corresponding bench width of 4 feet, such that compaction equipment can be used. The new levee slope will be constructed by replacing with new structural fill to the required crest elevation. Side slopes may vary from 3H:1V (horizontal to vertical) for grass slopes and 2H:1V for armored slopes. Armored slopes typically consist of larger rock and/or “soft armoring” such as large tree stumps and logs depending on the environmental conditions and objectives. GeoEngineers’ environmental restoration specialists may contribute to soft armoring design in the future.

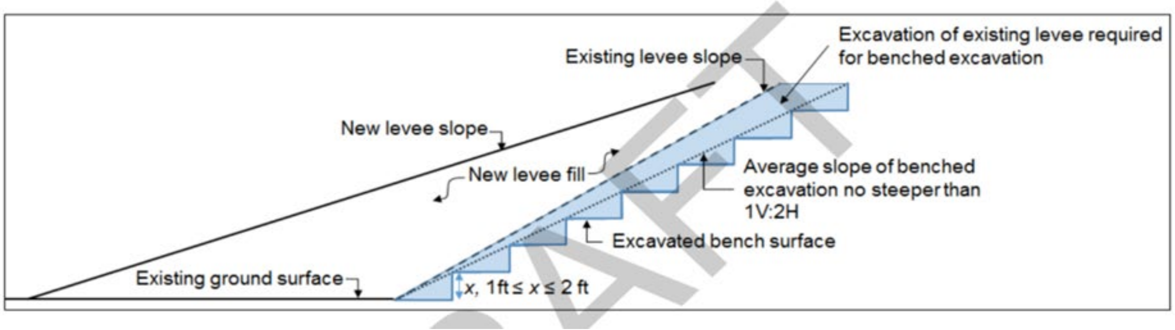


Exhibit 2: Typical Road Raising (USACE, 2000)

4.2 EARTHEN EMBANKMENT LEVEE

The following information is provided by reference to USACE EM 1110-2-1913 (USACE, 2000), and the draft version of the same document from December 2023 (USACE, 2023).

4.2.1 Geometry

As seen below (Exhibit 3) from the 2023 Draft EM 1110-2-1913, a typical earthen levee section may include a 10- to 12-foot-wide crest with two side slope options: 3H:1V for grass slopes, and 2H:1V for armored slopes. A typical section for an inspection trench within the levee's central portion is also provided. An armored levee slope for riverside slopes is also provided.

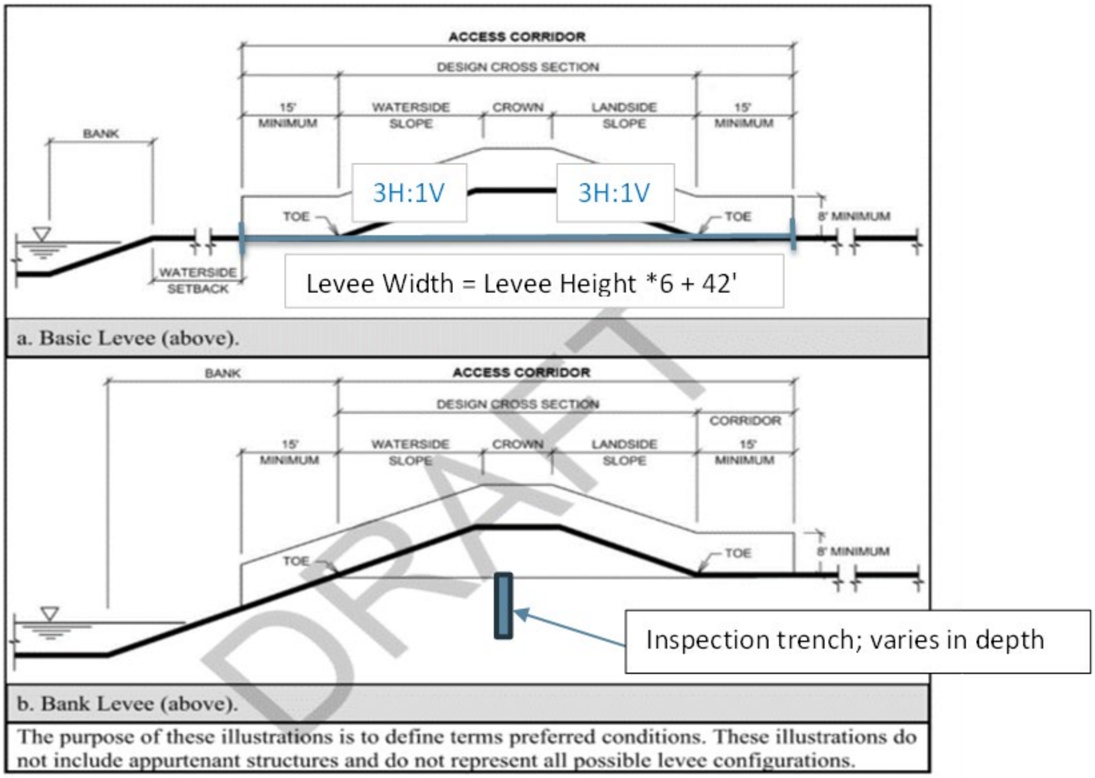


Exhibit 3: Typical Earthen Levee Cross Sections (USACE, 2023) with annotations by GeoEngineers

4.2.2 Exploration (Inspection) Trench

A detail showing the location of a typical inspection trench is provided in the Geometry section of this memorandum (above). In summary, the following is provided by the USACE regarding exploration/inspection trenches:

From the 2000 Manual:

> *f. Exploration trench*. An exploration trench (often termed "inspection trench") should be excavated under all levees unless special conditions as discussed later warrant its omission. The purpose of this trench is to expose or intercept any undesirable underground features such as old drain tile, water or sewer lines, animal burrows, buried logs, pockets of unsuitable material, or other debris. The trench should be located at or near the centerline of hauled fill levees or at or near the riverside toe of sand levees so as to connect with waterside impervious facings. Dimensions of the trench will vary with soil conditions and embankment configurations. Backfill should be placed only after a careful inspection of the excavated trench to ensure that seepage channels or undesirable material are not present; if they are, they should be dug out with a base of sufficient width to allow backfill compaction with regular compaction equipment. To backfill narrower trenches properly, special compaction procedures and/or equipment will be required. Trenches should have a minimum depth of 1.83 m (6 ft) except for embankment heights less than 1.83 m (6 ft), in which case the minimum depth should equal the embankment height. Exploration trenches can be omitted where landside toe drains beneath the levee proper constructed to comparable depths are employed (toe drains are discussed in more detail later in this chapter).

From the 2023 Draft Manual:

> 10.2.6.2 Dimensions and depth of the trench will vary with soil conditions and embankment configurations. Side slopes of the exploration trench should be stable and sufficient to allow personnel entry for inspection of conditions. Backfill should be placed only after a careful inspection of the excavated trench to ensure that seepage channels or undesirable material are not present; if they are, they should be dug out with a base of sufficient width to allow backfill compaction with regular compaction equipment. To backfill narrower trenches properly, special compaction procedures and/or equipment will be required. Trenches should have a minimum depth of 6 feet except when levee embankment height will be less than 6 feet, in which case the minimum depth should be equal to the embankment height or 3 feet, whichever is greater. Generally, it is preferred that the exploration trench is deeper than the typical depth of utility installation in the project area. If local practice dictates that utilities are typically installed deeper than 6 feet, the exploration trench should be dug to a depth 1 foot deeper than the typical utility installation depth. The bottom width of the trench should be wide enough to accommodate compaction equipment (tractor with towed roller or a self-propelled roller), which is generally a minimum of 8 to 12 feet. Depending on local conditions, exploration trenches may be eliminated if a seepage barrier or cutoff wall is constructed through the levee and into the foundation to a minimum depth of 6 feet into the foundation or deeper depths. Exploration trenches may be eliminated for modification to existing levees when conditions within the existing levee have been evaluated and there is not an expectation of adverse foundation conditions within the existing fill.
>
> 10.2.6.3 Contract specifications should address requirements for advance notification before trenches are excavated, to assure adequate opportunity for inspection and documentation by qualified personnel (typically provided by the owner of the levee construction project). After the trenches have been inspected and prior to backfilling, the trench subgrade should be scarified, moisture conditioned, and compacted. The required minimum relative compaction of the subgrade is typically less than the required trench backfill due to compacting upon native subsoil conditions. Trenches should be backfilled with compacted fill consistent in quality with the material that will be used in the overlying embankment. This work should be performed in a timely manner to limit exposure to possible rain events that could cause ponding and saturation of the foundation materials. If adverse foundation conditions that may impact the performance of the levee project are observed in the exploration trench, these conditions should be immediately brought to the attention of the designers or project engineer of the levee construction project before backfilling the exploration trench. The designer or project engineer for the levee construction project should determine the necessary actions to remedy the adverse foundation condition such that the requirements of the levee project are met.

In our experience inspection trenches can be narrower than noted above provided proper compaction techniques and equipment are used. Inspection trenches are critical and may need to be extended to greater depths or extents in urban areas where the risk of constructing a levee over unknown and unidentified utilities is greater. An unknown and improperly abandoned pipe below a levee would create a critical flaw in the embankment foundation that could lead to rapid levee failure.

4.2.3 Clearing and Grubbing

From the 2000 Manual:

7-2. Foundation Preparation and Treatment

a. General. Minimum foundation preparation for levees consists of clearing and grubbing, and most levees will also require some degree of stripping. Clearing, grubbing, stripping, the disposal of products therefrom, and final preparation are discussed in the following paragraphs.

b. Clearing. Clearing consists of complete removal of all objectional and/or obstructional matter above the ground surface. This includes all trees, fallen timber, brush, vegetation, loose stone, abandoned structures, fencing, and similar debris. The entire foundation area under the levee and berms should be cleared well ahead of any following construction operations.

c. Grubbing. Grubbing consists of the removal, within the levee foundation area, of all stumps, roots, buried logs, old piling, old paving, drains, and other objectional matter. Grubbing is usually not necessary beneath stability berms. Roots or other intrusions over 38.1 mm (1-1/2 in.) in diameter within the levee foundation area should be removed to a depth of 0.91 m (3 ft) below natural ground surface. Shallow tile drains sometimes found in agricultural areas should be removed from the levee foundation area. The sides of all holes and depressions caused by grubbing operations should be flattened before backfilling. Backfill, consisting or material similar to adjoining soils, should be placed in layers up to the final foundation grade and compacted to a density equal to the adjoining undisturbed material. This will avoid "soft spots" under the levee and maintain the continuity of the natural blanket.

d. Stripping. After foundation clearing and grubbing operations are complete, stripping is commenced. The purpose of stripping is to remove low growing vegetation and organic topsoil. The depth of stripping

4.2.4 Levee Fill Materials

Levee fill materials are expected to be both strong and relatively impermeable. Additional discussion of local fill sources is provided above. Examples of fill specifications used in a levee rehabilitation project in the City of Kent is provided below. This can be used as a basis for levee fill used in the region and derived from local geologic units.

As shown in the City of Kent project sections below, levee material is usually truncated by a maximum particle size of 2- to 4-inches, and by a minimum particle size equal to 20 to 30 percent by weight passing the No. 200 sieve. If cohesive material which does not contain organics is used, a minimum plasticity index of 5 and maximum liquid limit of 40 is required.

Excerpts of specifications from the City of Kent, WA Horseshoe Bend Levee Improvements East and West Riverbend Secondary Levees Phase II project are provided below:

SECTION 9-03.14 IS SUPPLEMENTED BY ADDING THE FOLLOWING NEW SECTION:

9-03.14(5) Gravel Borrow for Levee Embankment

Material for the levee embankment shall consist of granular material, either naturally occurring or processed, and shall be homogenous in nature consisting of at least 20% by weight passing the No. 200 sieve, and within 3 percent of its optimum moisture content, defined as the moisture content corresponding to the maximum modified Proctor dry density as determined by ASTM D 1557 test procedure. All particles 4-inches and larger shall be removed from the material. Levee embankment material shall also be free of organic and other deleterious material.

Sieve analysis shall be used to verify that this requirement is met. Recycled materials such as broken concrete or asphalt, shall not be allowed unless specifically authorized in advance by the Engineer.

Excerpts of specifications from the City of Kent, WA Briscoe-Desimone Levee project:

SECTION 9-03.14 IS SUPPLEMENTED BY ADDING THE FOLLOWING NEW SECTION:

9-03.14(5) Levee Embankment Fill

Levee Embankment Fill (or Low Permeability Fill) shall be clay, silt or silty/clayey sand having a maximum particle size of 2 inch, at least 25 percent by weight passing the No. 200 sieve, a minimum plasticity index of 5, a maximum liquid limit of 40 and be free of organic and deleterious materials.

Allowable Unified Soil Classification System (USCS) (ASTM D2487) classifications: CL, ML, SC and SM.

Blend adequately during placement such that the compacted material forms a uniform, homogeneous, stiff, void free, and relatively impervious compacted fill.

Sieve analysis (ASTM D422) and Atterberg limits (ASTM D4318) shall be used to verify that this requirement is met.

Recycled materials such as broken concrete or asphalt shall not be allowed.

Excavated riverbank soils that meet this specification shall be used as Levee Embankment Fill unless deemed unacceptable by the Engineer. Contractor shall segregate and stockpile excavated riverbank soils to maximize their use as Levee Embankment Fill. Use imported Levee Embankment Fill only when approved in writing by the Engineer prior to importing.

4.2.5 Preloading and Overbuilding

Embankment levees constructed on alluvium will likely experience some settlement. The degree of settlement will depend mostly on the thickness and composition of the upper layer of fine grained alluvium. Compared to other areas in western Washington, the fine grained alluvium in the Chehalis area is relatively limited and therefore could have less overall settlement than some other comparable projects.

Preloading the levee can be an effective way of minimizing settlement after construction. It is used in levee construction to help mitigate the potential for differential settlement between relative heavy levee embankment material and relatively light concrete material, such as those used for closure structures, and portions of the levee embankment that contains a pile-supported wall.

Where settlement-sensitive structures are planned adjacent to larger earth embankments, the earth embankments should be constructed first to allow for settlement of the embankment and compression of the underlaying soils prior to the installation of the settlement sensitive structures. Generally the majority of damaging settlements occur within weeks to months of the initial fill placement.

Embankment levees can be overbuilt (i.e. built to a higher elevation than the required crest elevation) to allow for future settlement. As general guidance, the levees should be overbuilt by about 5 percent to allow for future settlement. The actual amount required or recommended will depend on the thickness of compressible layers in the foundation soil and the proposed construction staging.

Another approach for over building embankments is to phase the construction so that the levee is rough graded early in the construction sequence and then scheduling final grading months to a year after the initial fill is placed. This will allow for adding material to account for the settlement and subgrade compression.

4.2.6 Levee Turnouts and Ramps

As presented in EM 1110-2-1913 Section 8-9, turnouts and ramps are required to allow for vehicle egress along the length of a levee. The following provides additional information and a typical detail (Exhibit 4).

8-9. Access Roads

a. Access road to levee. Access roads should be provided to levees at reasonably close intervals in cooperation with state and local authorities. These roads should be all-weather roads that will allow access for the purpose of inspection, maintenance, and flood-fighting operations.

b. Access road on levee. Access roads, sometimes referred to as patrol roads, should be provided also on top of the levees for the general purpose of inspection, maintenance, and flood-fighting operations. This type of road should be surfaced with a suitable gravel or crushed stone base course that will permit vehicle access during wet weather without causing detrimental effects to the levee or presenting safety hazards to the levee inspection and maintenance personnel. The width of the road surfacing will depend upon the crown width of the levee, where roadway additions to the crown are not being used, and upon the function of the roadway in accommodating either one- or two-way traffic. On levees where county or state highways will occupy the crown, the type of surfacing and surfacing width should be in accordance with applicable county or state standards. The decision as to whether the access road is to be opened to public use is to be made by the local levee agency which owns and maintains the levee.

(1) Turnouts. Turnouts should be used to provide a means for the passing of two motor vehicles on a one-lane access road on the levee. Turnouts should be provided at intervals of approximately 762 m (2500 ft), provided there are no ramps within the reach. The exact locations of the turnouts will be dependent upon various factors such as sight distance, property lines, levee alignment, and desires of local interests. An example turnout for a levee with a 3.65 m (12-ft) levee crown is shown in Figure 8-4.

(2) Turnarounds. Turnarounds should be provided to allow vehicles to reverse their direction on all levees where the levee deadends, and no ramp exists in the vicinity of the deadend. An example turnaround for a levee with a 3.65-m (12-ft) crown is shown in Figure 8-5.

EM 1110-2-1913
30 Apr 2000

at other convenient locations to serve landowners who have property bordering the levee. Ramps are also provided on some occasions on the riverside of the levee to connect the access road on top of the levee with existing levee traverses where necessary. The actual locations of the ramps should have the approval of the local levee agency which owns and maintains the levee. When used on the riverside of the levee, they should be oriented to minimize turbulence during high water.

b. Ramps are classified as public or private in accordance with their function. Public ramps are designed to satisfy the requirements of the levee owner: state, county, township, or road district. Private ramps are usually designed with less stringent requirements and maximum economy in mind. Side-approach ramps should be used instead of right angle road ramps because of significant savings in embankment. The width of the ramp will depend upon the intended function. Some widening of the crown of the levee at its juncture with the ramp may be required to provide adequate turning radius. The grade of the ramp should be no steeper than 10 percent. Side slopes on the ramp should not be less than 1V on 3H to allow grass-cutting equipment to operate. The ramp should be surfaced with a suitable gravel or crushed stone. Consideration should be given to extending the gravel or crushed stone surfacing to the levee embankment to minimize erosion in the gutter. In general, private ramps should not be constructed unless they are essential and there is assurance that the ramps will be used. Unused ramps lead to maintenance neglect.

c. Both public and private ramps should be constructed only by adding material to the levee crown and slopes. The levee section should never be reduced to accommodate a ramp.

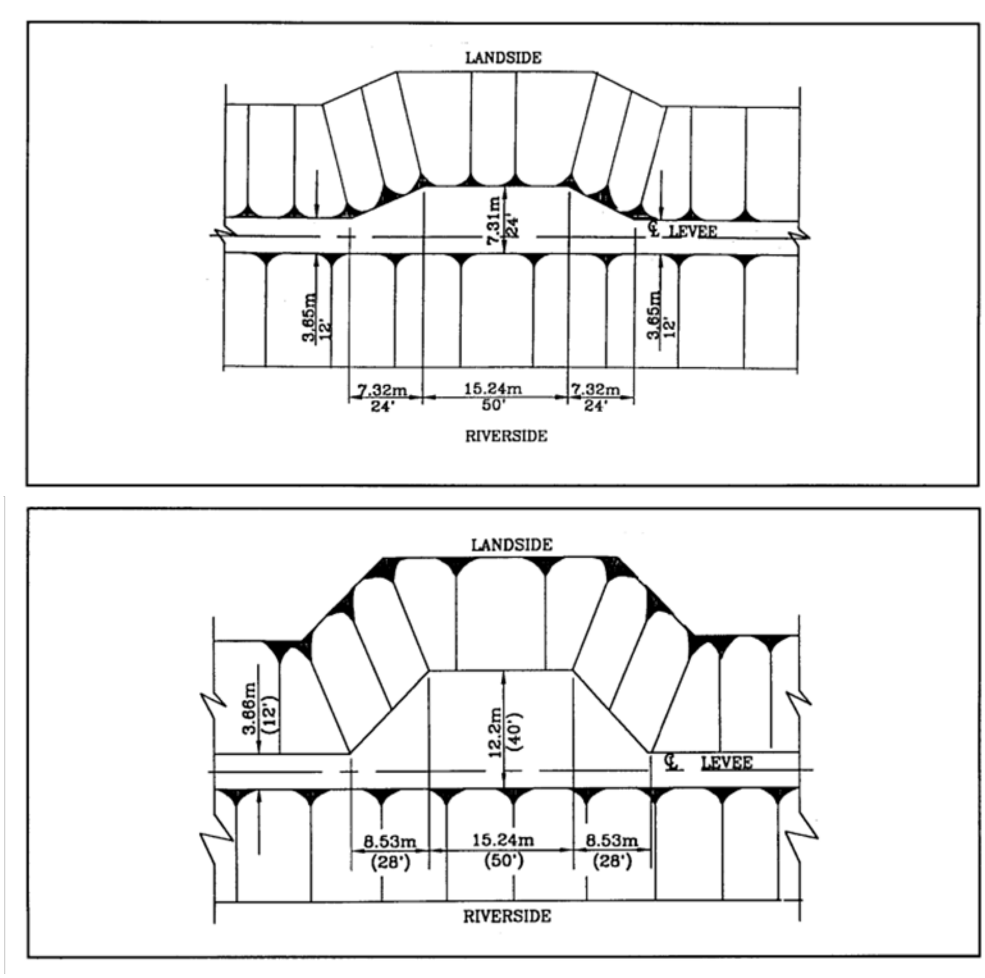


Exhibit 4: Example of Levee Turnout (USACE, 2000)

5.0 Walls

5.1 I-WALLS (CANTILEVER)

Information regarding, I-Walls has been provided through reference of the USACE Engineering Technical Letter (ETL) 1110-2-575 (USACE, 2011). I-Walls are most suitable on relatively flat ground with limited space. We expect they will mostly be used in urban areas. Below is a table and figure (Exhibits 5 and 6) providing a guide for typical I-Wall heights by foundation soil type.

The primary structure of an I-Wall is a sheet pile. A plain steel sheet pile can extend above the ground surface to the design height or the above ground portion can consist of a concrete cap. An advantage of the concrete cap is that it can limit corrosion by preventing exposed structural steel. It is also usually preferred aesthetically. Exhibit 7 provides example details.

There is some risk in designing I-Walls in the project area. The relatively soft upper layers provide little lateral support for the I-Walls. This requires deeper embedment for stability. The relatively shallow denser gravel layers could make installation of deeper sheet piles difficult. A thorough subsurface exploration program is prudent wherever I-Walls are proposed.

	Foundation Type			
	Sand	Soft Clay	Stiff Clay	
Load Category	$\phi' \geq 32.5$, $D_r \geq 0.50$	$s_u \leq 300$ psf	$s_u \geq 1{,}500$ psf	Wall on Earthen Levee Embankment
Usual	7 ft. (2.1 m)	5 ft. (1.5 m)	8 ft. (2.4 m)	4 ft. (1.2 m)
Unusual	9 ft. (2.7 m)	7 ft. (2.1 m)	12 ft. (3.7 m)	4 ft. (1.2 m)
Extreme	11 ft. (3.4 m)	8 ft. (2.4 m)	15 ft. (4.6 m)	4 ft. (1.2 m)

Notes:

(a) D_r = relative density

(b) The heights in the table are the distance from the elevation of the top of wall to the ground surface on the landside of the wall, as shown in Figure 9.12.

(c) Linear interpolation is permitted for clay strengths between those shown, but extrapolation is not permitted.

(d) Limited data has been developed for deformation of walls on levees. Walls on levees with heights greater than 4 ft. (1.2 m) as shown in Figure 9.12 require analysis by full numeric analysis methods.

(e) For layered soils, the engineer will base the maximum height on the predominant soil type resulting in the lowest water height for deformation control. The soil type used is determined by the soil on the landside of the wall extending from the ground surface to the elevation of maximum net passive pressure from the CWALSHT or CI-WALL analysis. Questionable sections will require analysis by full numeric analysis methods.

(f) Walls founded in soils with properties outside the range of soils in the table must be analyzed with full numeric analysis methods.

Exhibit 5: Maximum Wall Heights for Deformation Control of Cantilever Pile Walls Used for Floodwalls (USACE, 2011)

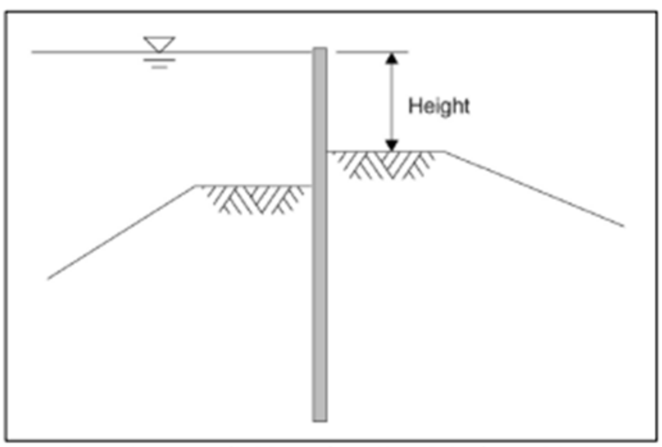
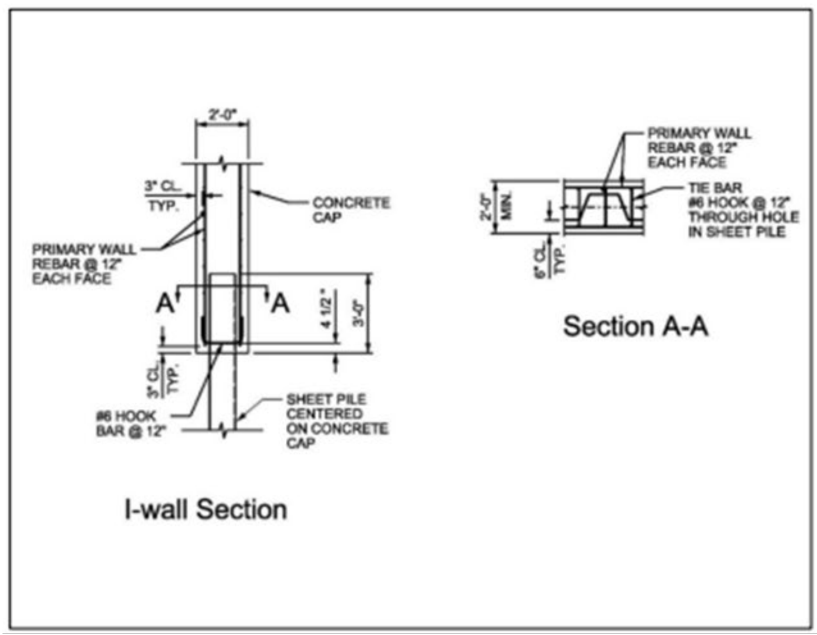


Exhibit 6: Definition of Wall Height Used in Exhibit 5 (USACE, 2011)



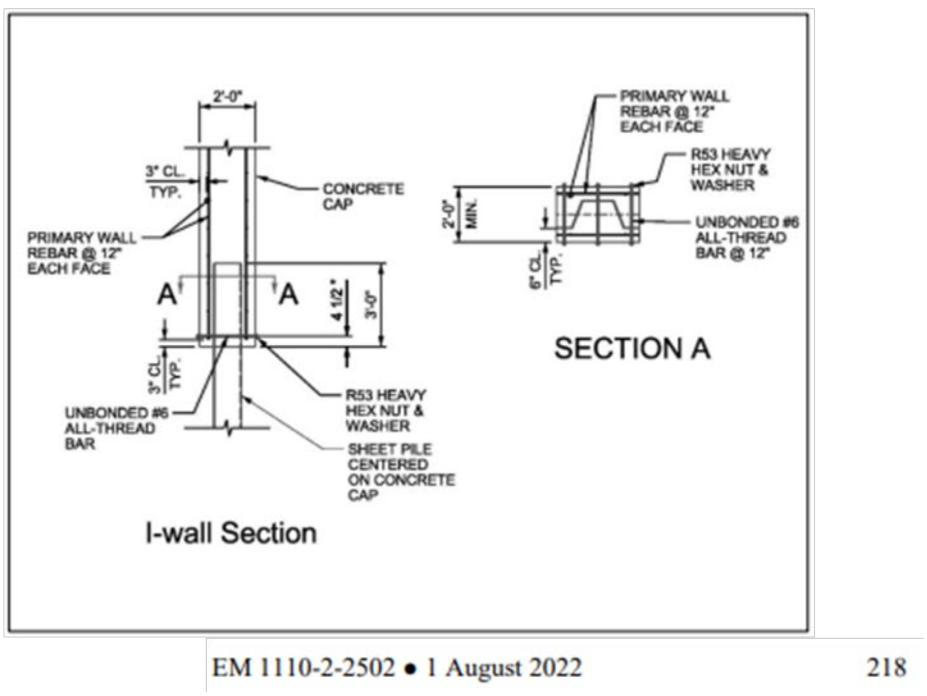


Exhibit 7: Typical Details for I-Wall Structures (USACE, 2022)

5.2 T-WALLS

Information regarding T-walls has been provided through reference of the USACE EM 1110-2-2502 (USACE, 2022).

T-Walls are generally more robust than I-Walls as the battered piles provide additional lateral support. Exhibit 8 provides an example from the USACE manual. Within the project area the battered piles are expected to be cost effective as the presence of the coarser alluvium provides relatively higher capacities at relatively shallow depths. Shorter T-Walls, on the order of and 5 feet or less can be designed and constructed without pile support; however, this should be considered on a case-by-case bases. We suggest assuming all T-Walls include some support piles for budgeting purposes.

Although the above-ground profile is identical to an I-Wall; however, the T-Walls require a larger footprint to accommodate the below ground structure.

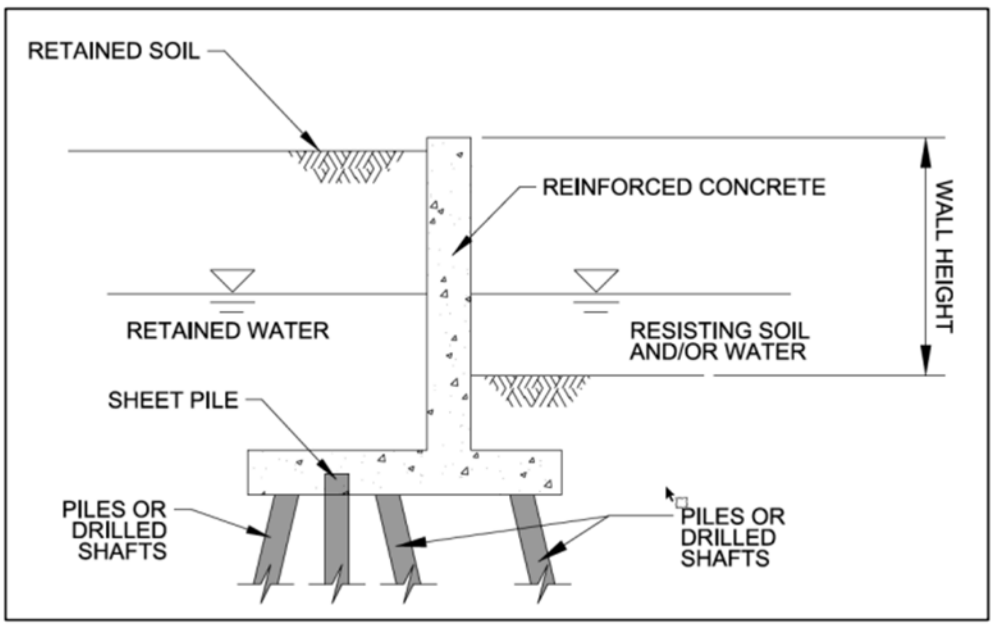


Exhibit 8: T-Wall Supported by Deep Foundation (USACE, 2022)

6.0 Utility Penetrations

Typical detail for utility penetrations into walls is provided below (Exhibits 9 and 10); the USACE EM 1110-2-2902 (USACE, 2020) (Exhibit 9) and; Schedule A Details from the City of Kent, WA Horseshoe Bend Levee Improvements East and West Riverbend Secondary Levees Phase II project (Exhibit 10). Utility penetrations can be considered a detail for further stages of design. However, understanding the effort will be important to developing construction costs. Specifically, estimating the difference in costs for levees or floodwalls constructed through relatively rural areas with fewer utility crossings and levees or floodwalls constructed through urban areas that could have many more crossings per length of levee.

6.1 INVERTED T-WALL PENETRATIONS

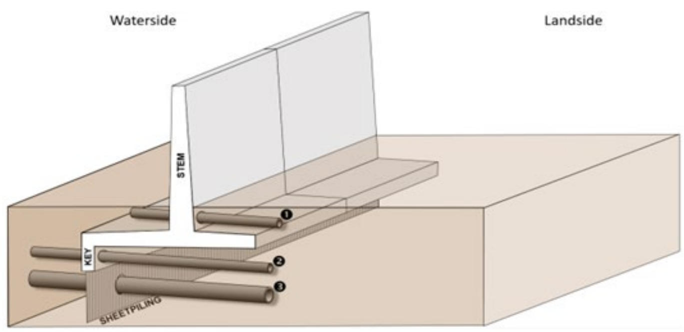


Exhibit 9: Potential Pipe Penetration Locations (USACE, 2020)

6.2 I-WALL PENETRATIONS

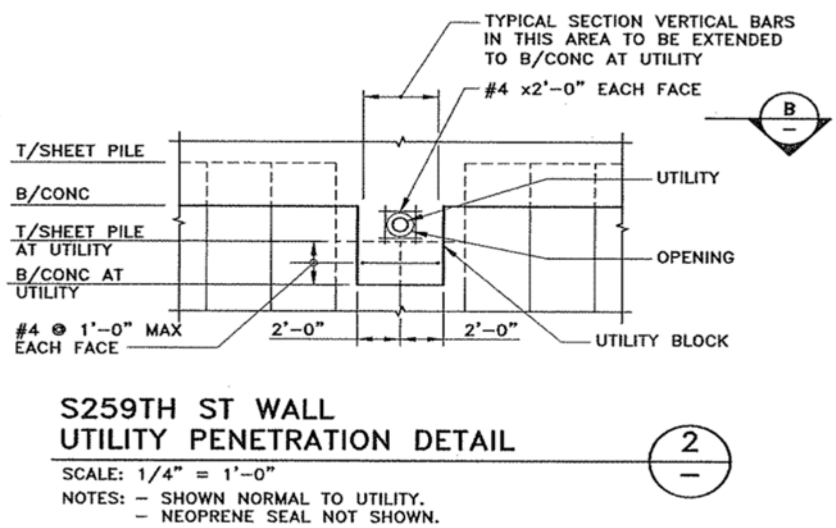


Exhibit 10: City of Kent, S 59th Street Wall

7.0 Wall to Levee

7.1 TRANSITIONS

Transitions between walls and embankment levees require additional detailing due to the potential for flood concentration and erosion around walls during an embankment failure, which could lead to failure of wall. This is especially a concern for I-Walls as the concentrated erosion can result in a catastrophic failure. Additional riprap is required at transitions as presented in the figure from USACE ETL 1110-2-575 (Exhibit 11) Engineering and Design Evaluation of I-Walls below.

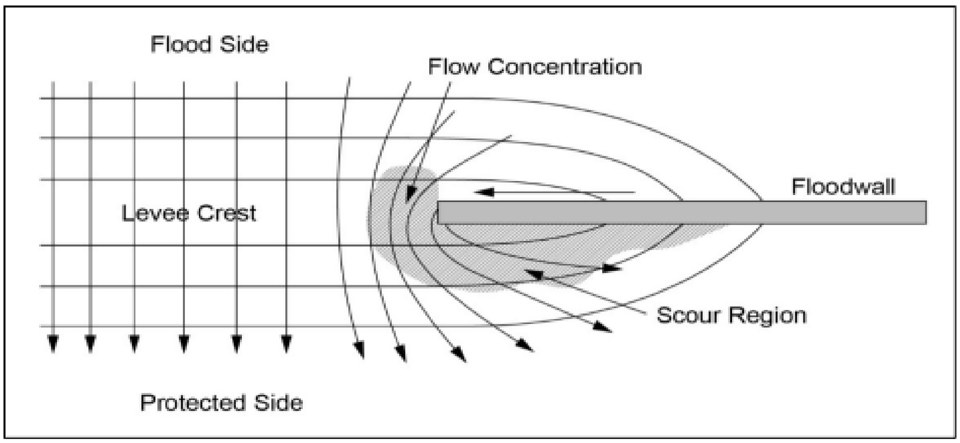


Exhibit 11: Diagram of Overtopping Erosion at Levee-Floodwall Transition (USACE, 2011)

8.0 Limitations

We have prepared this report for the exclusive use of the Moffatt & Nichol and their authorized agents for Chehalis LAND project. Our services for this project include identifying general geotechnical issues that could be present in the area as well as providing generally accepted methods to address those issues. No specific design recommendations have been provided. Any future design must consider site specific geologic and geotechnical conditions.

Qualified engineering and construction practices can help mitigate flooding risks, but they cannot eliminate those risks. Levee systems require periodic inspection to confirm that all critical components continue functioning as intended. Confirmation that design flood flows and/or elevations have not significantly changed also requires the periodic review of design criteria and other potential contributing factors including, but not limited to, changes in surrounding development, weather patterns, system operational policies, or sedimentation.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

9.0 References

Chehalis Basin Strategies Local Actions Non-Dam Alternative (LAND) Draft Report

United States Army Corps of Engineers (USACE), Engineer Circulars (EC) 1110-2-6067 USACE Process for the National Flood Insurance Program (NFIP) Levee System Evaluation, August 31, 2010

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United States Army Corps of Engineers (USACE), Engineering Manual (EM) 1110-2-2502 Floodwalls and Other Hydraulic Retaining Walls, August 1, 2022.

United States Army Corps of Engineers (USACE), Engineering Manual (EM) 1110-2-2902 Conduits, Pipes, and Culverts Associated with Dams and Levee Systems, December 31, 2020.

United States Army Corps of Engineers (USACE), Engineering Technical Letter (ETL) 1110-2-575 Evaluation of I-Walls, September 1, 2011.

United States Geological Survey (USGS). Hydrogeologic Framework and Groundwater/Surface-Water Interactions of the Chehalis River Basin, Southwestern Washington, Scientific Investigations Report 2011-5160, website: sir20115160.pdf (usgs.gov). (Gendaszek 2011).

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Washington Geologic Survey (WGS). Geologic Map of the Adna 7.5-minute quadrangle, Lewis County, Washington 1:24,000 (Sadowski et. al, 2019) https://ngmdb.usgs.gov/Prodesc/proddesc_109371.htm

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