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In this issue of the ***Western Dam Engineering Technical Note***, we present articles on embankment slope stability with a focus on low hazard structures, a new tool for estimating precipitation, and the first in a series of articles on the importance of technical project specifications. This quarterly newsletter is meant as an educational resource for civil engineers who practice primarily in rural areas of the western United States. This publication focuses on technical articles specific to the design, inspection, safety, and construction of small dams. It provides general information. The reader is encouraged to use the references cited and engage other technical experts as appropriate.

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- Colorado Division of Water Resources
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GOOD TO KNOW

2013 Colorado Flooding Event Link

Upcoming ASDSO Webinar Dam Safety Training:

- *Internal Drainage Systems for Embankment Dams, By James R. Talbot, P.E., December 10, 2013*
- *Intro to Armoring Embankment Dams & Earthcut Spillways With ACBs, by Paul Schweiger, P.E., and Chris Thornton, Ph.D., P.E., January 14, 2014*

Upcoming ASDSO Classroom Technical Seminars

- *Inspection and Assessment of Dams, Little Rock, AR, March 4-6, 2014.*
- *Seepage Through Earthen Dams, Denver, CO, April 1-2, 2014*

ASDSO Training Website Link

In this Issue

Embankment Dam Slope Stability 101

Introduction

Design of new embankment dams, and the more common scenario of reviewing the conditions of existing dams, should, as general practice, include evaluating the stability of the embankment structure. Stability, in the simplest definition, refers to the ability of a slope to resist the driving forces tending to move earth materials downslope. The stability of an embankment can be adversely affected by excessive stresses on the crest or slopes, sudden addition or loss of water in the reservoir, changes in internal water pressures, or loss of materials due to erosion (both internal, such as piping, and external, such as surface erosion). Stability conditions of a dam can be assessed using both visual and analytical methods.

Recently, the central Front Range and surrounding areas in Colorado experienced historic rainfall that led to extensive flooding in the region. The rainfall and flood imposed loading conditions that many dams, both large and small, had never experienced. These events may have created changes of conditions, internally, in embankment dams. The Colorado State Engineer's Office recently completed emergency inspection reports for affected dams, some of which will require quantitative slope stability analyses to further assess their conditions and levels of safety.

The purpose of this article is to describe visual inspections of stability performance and identify triggers that may indicate the need for a more quantitative or analytical approach. This article is not intended to be prescriptive and provides only a general overview of assessing embankment stability. Future articles will provide more details in terms of strength characterization and specific analysis methodology for different loading cases.

Visual Inspection and Monitoring for Stability

For many western states, State Engineers have waived the requirements of performing stability analyses for low hazard dams if it can be demonstrated that the dams have conservative slopes and were constructed of competent materials. Generally, upstream earth embankment slopes should be no steeper than 3H:1V (horizontal to vertical), and downstream earth embankment slopes no steeper than 2H:1V. Regular visual inspections are always required, even if stability analyses have been waived, and such inspections can provide efficient means of monitoring embankment performance with respect to stability.

Regular visual inspection is the best tool an Owner can use to assess the safety of an embankment dam. Benchmarking photographs (those taken of the same feature from the same perspective, inspection to inspection) are invaluable to the monitoring process. Photos can be compared across multiple inspections to identify subtle changes in conditions, which may be an indication of a developing adverse condition that affects the stability and safety of the dam.

Visual indicators of developing instability may include:

- Longitudinal cracks on the dam crest or slope (see **Photo 1**).
- Wet areas on the downstream slope or toe (see **Photo 2**) indicating an adverse internal phreatic level within the embankment. The relationship between reservoir level and seepage quantity and quality should also be established and used to compare successive observations.
- An apparent slope failure or slump (see **Photo 3**).
- Erosion or sloughing of the downstream slope which results in oversteepening of the overall slope.
- Displaced riprap, crest station markers, or fence lines indicating movement.
- Bulges at or downstream of the toe.
- Depressions or sinkholes in the dam crest or slopes.
- Changes in the appearance of the normal waterline against the upstream slope at multiple water levels.

Photo 1. Severe longitudinal cracks in downstream slope

Photo 2. Seepage exiting dam face

Photo 3. Slope failure on downstream slope

Triggers for More Quantitative Analyses

Besides a change in conditions resulting from rainfall/flooding or other events, triggers requiring stability analysis be performed may include:

- Designing a new dam.
- Raising an existing dam.
- Construction of a berm.
- Potential reclassification of a dam to high hazard.
- Deterioration of existing conditions, i.e. oversteepening of embankment slopes for any reason.
- Reassurance that a latent, undetected issue has not developed – indicators of such an issue may include embankments with steep slopes (greater than 2H:1V), soft foundation conditions, high phreatic surface within the dam and/or foundation, seepage at the face or toe, depression/sinkhole formation or observed scarp or bulge.
- Indications from field observations that instability may be developing – i.e. observed scarps, toe bulges, longitudinal cracking along crest or slope.

Slope Stability Analysis Requirements

The analyzed stability of a slope is expressed as a Factor of Safety (FS). FS values greater than 1 indicate the estimated driving forces are less than the resistance forces. However, due to inherent uncertainties in the behavior and characterization of earth materials, regulations and good practice require FSs greater than 1 for most loading conditions. Each regulatory agency has its own FS requirements; however, the following table provides some commonly adopted values:

Loading Condition	Min. Factor of Safety
Steady State Drained	1.5
End of Construction	1.3
Rapid Drawdown	1.2
Post-Seismic	1.2
Pseudo-Static (where applicable)	1.0

To prepare a slope stability analysis, a model or sectional view of the slope is developed for the most vulnerable section, typically the maximum section of the dam, or where signs of distress are observed. The phreatic surface is included in the model and can be identified through piezometer readings, when available, by accurately located observations of wetness or free water on the embankment, or by estimating a typical phreatic surface shape. References such as Cedergren (1989) can be used to estimate the phreatic surface for various embankment zoning scenarios. Each material or soil type within the embankment and the foundation should be assigned appropriate properties for use in the analysis.

Slope stability is primarily a tool for comparing the relative stability of various possible designs at a site and benchmarking them against historically successful practice. It should not be relied upon as an absolute indicator of the safety of a particular design.

Drained or Undrained

It is important to understand whether the embankment or foundation soils have high permeability (e.g., can drain during a change in loading condition; drained behavior) or if they are a low permeability material (e.g. cohesive materials in which excess pore pressures due to loading takes longer to dissipate; undrained behavior). Duncan et al (1996) provides a logical base to estimate the degree of drainage to evaluate whether a material will behave in a drained or undrained manner during rapid drawdown. This basis can be extended to other possible loading conditions to evaluate whether undrained strengths would be induced. This is done by using the dimensionless time factor, T which is expressed as:

$$T = C_v t/D^2$$

in which C_v = coefficient of consolidation (ft^2/day or m^2/day); t= construction or loading time (days); and D = length of drainage path (feet or meters). Typical values of C_v for various soils are given in Duncan, Wright, and Wong (1992), and are summarized in the following table:

Type of Soil	Values of C_v
Coarse sand	>10,000 ft^2/day
Fine sand	100 to 10,000 ft^2/day
Silty sand	10 to 1,000 ft^2/day
Silt	0.5 to 100 ft^2/day
Compacted clay	0.05 to 5 ft^2/day
Soft clay	<0.2 ft^2/day

If the value T exceeds 3.0, it is reasonable to treat the material as drained. If the value T is less than than 0.01, it is reasonable to treat the material as undrained. If the value T is between these two limits, both possibilities should be considered. If the data required to calculate T are not available, it is usually assumed for problems that involve normal rates of loading, that soils with permeabilities (hydraulic conductivities) greater than 10^{-4} cm/sec will be drained, and soils with permeabilities less than 10^{-7} cm/sec will be undrained. If hydraulic conductivity falls between these two limits, it would be conservative to assume that the material is undrained.

Typical Soil Parameters

If available, investigation records including geologic assessments, drill logs, laboratory test data, in situ test data, or even construction specifications should be reviewed to identify material characterization properties (such as gradation, density, Atterberg limits) and ideally, if available, shear strength parameters (undrained and drained) for the embankment and foundation materials.

If strength parameters are not available from test data, index properties and blow counts can be used with published correlations to estimate strength parameter ranges for each type of soil. If index properties or blow count data are not available, only a screening level of analysis can be performed. For screening level analyses, published reference strength parameter values can be used. Reference and correlation values for engineering properties of gravels, sands, silts, and clays of varying plasticity can be found in the following manuals and papers (hyperlinks provided where available):

- NAVFAC Department of the Navy, NAVFAC DM-7.01, Soil Mechanics, US Department of Defense, Alexandria 2005.

- Lambe and Whitman, *Soils Mechanics, SI Version,* 1979.
- Hunt, *Geotechnical Engineering Investigation Manual,* McGraw-Hill, New York, 1984.
- Bell, *Engineering Properties of Soils and Rocks,* Butterworth-Heinemann, Oxford, UK, 1992.
- Duncan and Wright, *Soil Strength and Slope Stability,* John Wiley & Sons, 2005.
- U.S. Dept. of the Interior, Bureau of Reclamation, Design of Small Dams, Third Edition, 1987. Table 5-1 in this reference provides typical values for compacted embankment soils.
- USSD, Materials for Embankment Dams, January 2011.

Typical Loading Conditions

After the slope geometry, phreatic surface, and material properties estimates have been established, the potential loading conditions of the embankment should be evaluated. Typical loading conditions include:

- Steady-state Drained – This condition represents the stability of the dam under normal operating conditions with steady-state seepage conditions and is one of the fundamental analyses performed in any quantitative analysis. Drained parameters should be used. Laboratory tests to evaluate the drained shear strength could include consolidated undrained triaxial tests with pore pressure measurement (CU'), drained triaxial tests (CD), or direct shear tests. Pore pressures can be estimated using flow nets, empirical relationships, or other types of seepage analyses. Both internal pore pressures (downstream slope) and external water pressures (upstream slope) should be included in the analysis. In case of noncohesive, drained embankment shell materials, infinite slope formulations ("angle of repose analysis") could be used to analyze shallow failure surfaces.
- End of Construction – This case should be analyzed when either embankment or foundation soils (or both) are predicted to develop significant pore pressures during embankment construction (undrained conditions) and undrained strengths are estimated to be less than drained strengths. Factors determining the likelihood of this occurring include the height of the planned embankment, the speed of construction, the saturated consistency of foundation soils, and others. If the materials are free-draining, the drained shear strengths should be considered. If the soils are cohesive, then undrained shear strengths should be considered. The total stress undrained shear strength should be evaluated, and laboratory tests to evaluate this could include undrained unconsolidated triaxial shear tests (UU). In the case of soft clay foundation, this loading case should be analyzed first, since it will likely control the embankment design.
- Rapid Drawdown – Analyze the stability of the upstream embankment slope for the condition created by a rapid drawdown of the water level in the reservoir from the normal full reservoir level. Although there are several methods of analyses, each having a different method of modeling the phreatic pressures during a rapid drawdown condition, the three-stage method presented by Duncan et al for developing appropriate phreatic and pore pressure parameters is the authors' recommended approach. Different agencies also have different requirements for the assumed drawdown elevations of the pool. For rapid drawdown analysis, undrained shear strengths should be used for both noncohesive (if material is judge to behave undrained as discussed above) and for cohesive embankment soils. Laboratory test to estimate undrained strengths could include the isotropically undrained triaxial tests with pore pressure measurement (CU').
- Seismic – Dams requiring seismic analysis should be designed to withstand at least the predicted earthquake loads with a full reservoir under steady-state seepage conditions. This is often referred to as a "pseudo-static" or post-earthquake analysis. Typically, this loading condition applies to high hazard structures. Refer to the applicable state

regulations for additional guidance. This condition should be evaluated when estimated local seismicity is anticipated to generate ground motions greater than about 0.10g, or as otherwise required by applicable regulations. For example, current NRCS practice is that no seismic analysis would be required for: 1) design ground accelerations less than 0.07g, and 2) well-constructed embankment dams on competent clay foundations or bedrock, where the design earthquake is less than 0.35g. If seismic analysis is deemed warranted, then the selection of the appropriate method and strengths can be complex and very case specific. This issue is outside the scope of this article and will be discussed in future publications.

Analysis Results

Resulting FS values higher than the minimum required values indicate the embankment is expected to be stable under the applied loading conditions. If FS values are lower than the required values, a more detailed investigation may be warranted to further characterize the embankment and foundation materials to better represent the site conditions. FS values lower than one generally indicate potential instability.

If obtaining site-specific data is justified, consider excavating test pits, advancing drill holes, performing in situ testing (e.g. blow counts, torvane, pocket penetrometer, etc.), and installing piezometers. Useful laboratory tests include gradation, density, Atterberg limits, consolidation, and triaxial shear strength testing.

Conclusions

This article presented embankment slope stability with a focus on smaller structures that may have limited data. The reader is further encouraged to read the references. Future articles will provide more in depth discussion on topics such as:

- Strength characterization with respect to laboratory testing and evaluation of drained and undrained shear strengths.
- Specific analysis methodology for different loading cases (i.e. rapid drawdown and seismic analysis).
- Sensitivity of selected shear strengths for the various loading cases.
- Applicability of various available methods of slope stability analysis; limit equilibrium, i.e. Bishop, Janbu, Spencer; Finite Element Method (FEM), etc.

References

Cedergren, H.R., 1989, Seepage, Drainage and Flow Nets, Third Edition, John Wiley and Sons, Inc., 465 pgs.

Duncan, J.M., S.G. Wright, and K.S. Wong, 1992, "Slope Stability During Rapid Drawdown," Proceedings of the H. Bolton Seed Memorial Symposium, Volume 2, No. 4, p. 253-272, B-Tech Publishers, Vancouver, B.C.

Duncan, J.M. 1996. "State of the Art: Limit Equilibrium and Finite-Element Analysis of Slopes". Journal of Geotechnical Engineering. Vol. 122, No. 7. July.

Duncan, J.M. and S.G. Wright, 2005, Soil Strength and Slope Stability, John Wiley and Sons, Inc., 297 pgs.

TR-210-60: Earth Dams and Reservoirs (Revised July 2005) (7/2005), Natural Resource Conservation Service.

When it Rains *Does* it Pour? Design Precipitation Depths for Dam Safety

Introduction

If a dam and its spillway are not sized appropriately to pass the required inflow, a precipitation event can lead to dam overtopping and failure. Selecting the design precipitation is the first step in the hydrologic analysis used to size the dam and spillway. The design precipitation is typically based on either a selected precipitation frequency (i.e. 100-year event) or Probable Maximum Precipitation (PMP) event.

This article looks at the references available for estimating the design precipitation for small dams in Colorado, Montana, Utah, and Wyoming. The recent extreme precipitation event in Colorado is also examined in relationship to frequency estimates and discussed in the context of dam safety.

Colorado's 2013 Precipitation Event

The September 9-16, 2013, precipitation event was caused by a slow-moving cold front stalled over Colorado, clashing with warm humid monsoonal air from the south. The precipitation resulted in catastrophic flooding along Colorado's Front Range from Colorado Springs, north to Fort Collins. Numerous low hazard dams that were designed to withstand a 100-year precipitation event overtopped, with nine earthen dams breaching. According to the Colorado Division of Water Resources, the high hazard dams within the affected area performed well, with many conveying spillway flows for the first time since they were built.

The Hydrometeorological Design Studies Center (HDSC) developed maps for the September event showing the annual exceedance probabilities of the worst case precipitation in relation to published frequency data presented in National Oceanic and Atmospheric Administration (NOAA) Atlas 14. **Figure 1** shows the map for the full seven day storm duration. Maps for 24-hour and 48-hour durations are also available.

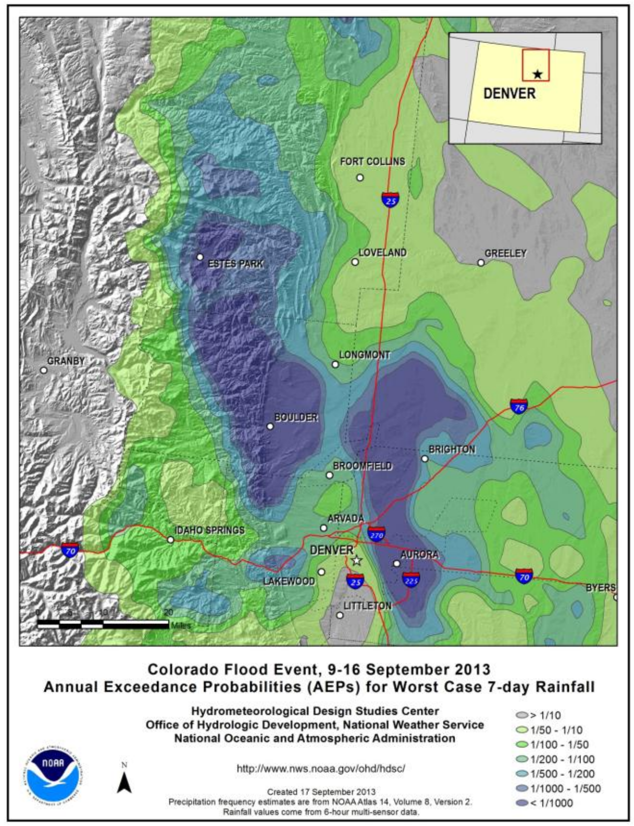

Figure 1: Worst Case 7-day Rainfall Annual Exceedance Probabilities

As shown in Figure 1, exceedance probabilities were estimated to be greater than the 0.1% (1/1000) for areas including Estes Park, Boulder, and Aurora.

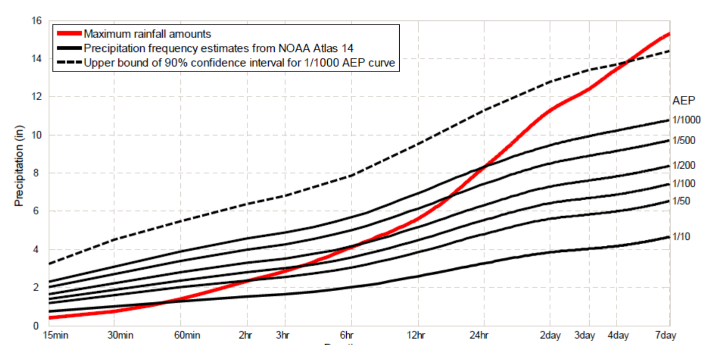

Figure 2: Maximum observed rainfall amounts in relationship to NOAA 14 estimates

Figure 2 shows the observed rainfall amounts for the Justice Center rain gauge located in Boulder, in relationship to the NOAA Atlas 14 precipitation frequency estimates. For the seven day duration, the observed precipitation was greater than the upper bound of 90% confidence interval for the 1,000-year precipitation event.

The September event is a reminder of the importance of designing a dam for the appropriate precipitation event and hazard classification. Flooding did result from the low hazard dam failures; however, there was little flooding from the state-classified high hazard dams, where failure would likely result in widespread damage and loss of human life, because these dams were designed appropriately for the PMP event.

State Criteria for Design Precipitation

The state criterion for determination of the dam design precipitation is based upon dam size and hazard classification. The hazard classification typically accounts for dam height, storage capacity, likelihood of failure (e.g. a dam located within a series of dams), and potential for loss of life and property, should a failure occur. The following discussion summarizes the hazard classification system and methods used to identify the dam design precipitation for Colorado, Utah, Montana, and Wyoming.

For Colorado, design precipitation is selected based upon dam size and hazard classification as presented in **Table 1**. Additional guidelines are available for altitude adjustments in the Colorado Rules and Regulations for Dam Safety and Dam Construction.

Table 1: Colorado Inflow Design Flood Requirements

INFLOW DESIGN FLOOD REQUIREMENTS FOR COLORADO USING HYDROMETEOROLOGICAL REPORTS (HMR)				
DAM SIZE	HAZARD CLASSIFICATION			
	High	Significant	Low	NPH
Large	0.90 PMP	0.68 PMP	100 YR	50 YR
Small	0.90 PMP	0.45 PMP	100 YR	25 YR
Minor	0.45 PMP	100 YR	50 YR	25 YR

Note: NPH = No Public Hazard Dam. This table was taken from Table 5.2 of the Office of the State Engineer Dam Safety Branch's "Rules and Regulations for Dam Safety and Dam Construction," dated January 1, 2007.

For Utah, design precipitation is selected based upon hazard classification as determined by the State Engineer. Design precipitation for all low hazard dams is the 100-year event, whereas significant and high hazard dams must use the Spillway Evaluation Flood (SEF). The SEF is defined as the most critical flood of either the 100-year event applied to a saturated watershed or one of the PMP events.

For Montana, all dams with a potential for loss of life due to failure are classified as high hazard and the minimum design precipitation considered for any impoundment greater than 50 acre-feet is the 500-year event. Design precipitation for all high hazard dams is determined following a loss of life analysis using the requirements summarized in **Table 2**.

Table 2: Montana Design Flood Requirements

Loss of Life (LOL)	Design Flood
Less than or Equal to 0.5	500 YR
0.5 to 5	LOL x 1000
5 to 1000	$P_s = P_{5,000}\ (10^{rd})$ Where: r = -0.304 + .435 log10 (lol) d = log10 (PMP) - log10 (P5,000) lol = estimated loss of life PMP = probable maximum precipitation $P_{5,000}$ = 5,000-year recurrence interval precipitation P_s = design precipitation to meet spillway standard
Greater Than 1000	Probable Maximum Precipitation (PMP)

This table was taken from Montana's Rules and Regulations, Rule 36.14.502 entitled, "Hydrologic Standard for Emergency and Principal Spillways".

For Wyoming, determination of design precipitation and dam hazard classification is at the discretion of the State Engineer. Additionally, all reservoirs with a dam height greater than 20 feet, storage capacity greater than 50 acre-feet, and/or a reservoir located in an area where extensive property damage or loss of life might result, are required to have a minimum design precipitation of the 100-year event.

State rules and regulations typically prescribe the minimum criteria and not necessarily the method for satisfying the criteria. For example, a common requirement for low hazard dams is the 100-year event. This design criterion is typically the 100-year, 24-hour rainfall with a specific temporal distribution of hourly rainfall. Hydrological guidelines are then followed to determine the Design Flood. Alternately, the 100-year flood can be derived from actual stream gauge data collected within the drainage area or a similar nearby drainage area. The reader is cautioned to work with each state's dam regulatory agency to gain an understanding which methods and guidelines are acceptable for meeting the state's minimum criteria.

Precipitation Frequency Events

Since 2004, NOAA, National Weather Service (NWS), and HDSC have been working on updating and posting online precipitation frequency estimates, such as the 100-year event, as part of NOAA Atlas 14 for various parts of the United States. Funding is the largest impediment to the updating process, and is typically pooled from a variety of Federal, State, and local agencies. **Figure 3** presents where NOAA Atlas 14 is currently available in blue.

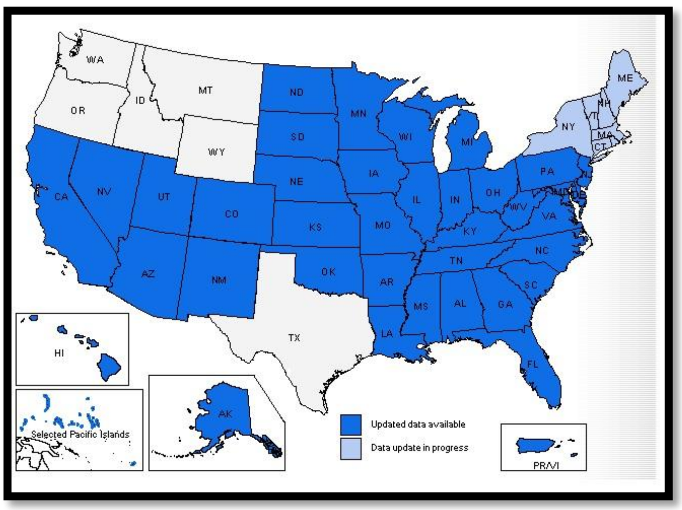

Figure 3: NOAA Atlas 14 Availability

As of 2013, Colorado and Utah have been updated to NOAA Atlas 14, while Wyoming and Montana still use NOAA Atlas 2 for storm durations of 1-hour to 24-hours. In addition to NOAA Atlas 2, Montana also uses the USGS WRI Report 97-4004 "Regional Analysis of Annual Precipitation Maxima in Montana" (Parent, 1997). This document is used to produce large recurrence intervals for 2-, 6-, and 24-hour storm durations specifically for dam design purposes. The typical duration used for dam design is the 24-hour duration.

While NOAA's goal is to update all states to NOAA 14, as of this publication, no funding has been received by NOAA and no plans are currently in place for updating Montana and Wyoming to NOAA Atlas 14.

The durations for NOAA Atlas 14 range from 5-minutes to 60 days and have an average recurrence interval ranging from 1 to 1,000 years. The updated analysis is different from NOAA Atlas 2 because it uses a longer period of record and a denser network of rain gauge stations, along with more robust and accepted statistical techniques. The precipitation magnitude-frequency relationships at individual rain gauge stations were based on regional frequency analysis approach based on L-moment statistics. The frequency analyses were carried out on annual maximum series (AMS) across a range of durations. Detailed information and discussion for deriving the estimates from rain gauge station data is provided in the NOAA Atlas 14 Document.

The Precipitation Frequency Data Server (PFDS) is an online point-and-click interface developed to deliver NOAA Atlas 14 precipitation frequency estimates and associated information. Upon clicking a state on the map or selecting a state name from the drop-down menu, an interactive map of that state will be displayed. From there, a user can identify a location from the map or enter the latitude and longitude for which precipitation frequency estimates are needed. The PFDS is shown in **Figure 4**.

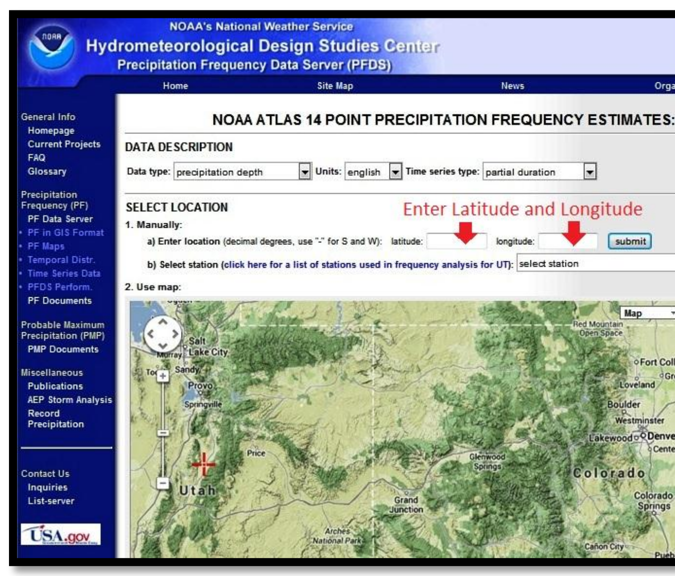

Figure 4: Precipitation Frequency Data Server

Estimates and their confidence intervals can be displayed directly as tables or graphs via separate tabs. Links to supplementary information (such as ASCII grids of estimates, associated temporal distributions of heavy rainfall, time series data at observation sites, cartographic maps, etc.) are also available. The ASCII grids of point estimates are the basis of the PFDS interface results and are available to be downloaded in a GIS compatible format. **Figure 5** presents the precipitation frequency table. The 24-hour duration, 100-year recurrence interval is highlighted in red. The numbers in parentheses are the upper and lower bound of the 90% confidence limits.

PF tabular | PF graphical | Supplementary information | Print Page

PDS-based precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.085 (0.065–0.113)	0.106 (0.080–0.143)	0.134 (0.100–0.184)	0.158 (0.116–0.220)	0.190 (0.137–0.270)	0.215 (0.152–0.310)	[illegible]	0.270 (0.186–0.401)	0.310 (0.209–0.470)	0.340 (0.226–0.522)
10-min	0.115 (0.088–0.153)	0.142 (0.108–0.191)	0.180 (0.134–0.247)	0.212 (0.156–0.295)	0.256 (0.184–0.364)	0.289 (0.205–0.417)	[illegible]	0.363 (0.250–0.539)	0.416 (0.280–0.630)	0.456 (0.303–0.701)
15-min	0.134 (0.103–0.178)	0.166 (0.126–0.224)	0.211 (0.157–0.289)	0.248 (0.182–0.345)	0.299 (0.215–0.425)	0.338 (0.239–0.487)	[illegible]	0.425 (0.292–0.631)	0.487 (0.328–0.738)	0.534 (0.355–0.820)
30-min	0.178 (0.136–0.237)	0.220 (0.167–0.296)	0.280 (0.208–0.384)	0.329 (0.241–0.458)	0.397 (0.285–0.564)	0.449 (0.318–0.647)	[illegible]	0.564 (0.388–0.838)	0.646 (0.435–0.978)	0.708 (0.470–1.09)
60-min	0.244 (0.187–0.325)	0.302 (0.229–0.407)	0.383 (0.285–0.525)	0.451 (0.331–0.628)	0.544 (0.391–0.772)	0.615 (0.436–0.887)	[illegible]	0.772 (0.531–1.15)	0.885 (0.597–1.34)	0.970 (0.644–1.49)
2-hr	0.340 (0.261–0.453)	0.420 (0.318–0.566)	0.534 (0.398–0.732)	0.628 (0.461–0.874)	0.758 (0.545–1.08)	0.858 (0.608–1.24)	[illegible]	1.08 (0.740–1.60)	1.23 (0.831–1.87)	1.35 (0.898–2.08)
3-hr	0.396 (0.304–0.527)	0.490 (0.371–0.660)	0.622 (0.463–0.853)	0.732 (0.537–1.02)	0.884 (0.636–1.26)	1.00 (0.708–1.44)	[illegible]	1.25 (0.862–1.86)	1.44 (0.969–2.18)	1.58 (1.05–2.42)
6-hr	0.578 (0.443–0.770)	0.715 (0.542–0.963)	0.908 (0.676–1.25)	1.07 (0.783–1.49)	1.29 (0.926–1.83)	1.46 (1.03–2.10)	[illegible]	1.83 (1.26–2.72)	2.10 (1.41–3.18)	2.30 (1.53–3.55)
12-hr	0.792 (0.607–1.05)	0.981 (0.744–1.32)	1.25 (0.928–1.71)	1.46 (1.07–2.03)	1.76 (1.27–2.50)	[illegible]	[illegible]	[illegible]	2.87 (1.94–4.35)	3.15 (2.09–4.84)
24-hr	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.97 (2.30–3.88)	[illegible]	3.83 (2.86–5.19)	4.20 (3.08–5.78)
2-day	1.35 (1.16–1.59)	1.65 (1.40–1.97)	2.08 (1.73–2.53)	2.43 (1.99–3.00)	2.93 (2.35–3.70)	[illegible]	[illegible]	[illegible]	4.97 (3.71–6.74)	5.49 (4.03–7.55)
3-day	1.55 (1.33–1.83)	1.88 (1.60–2.24)	2.36 (1.96–2.87)	2.76 (2.26–3.40)	3.33 (2.67–4.21)	3.81 (3.00–4.89)	4.33 (3.35–5.64)	4.95 (3.77–6.56)	5.77 (4.31–7.82)	6.40 (4.70–8.80)
4-day	1.71 (1.47–2.01)	2.06 (1.75–2.45)	2.57 (2.14–3.13)	3.00 (2.46–3.71)	3.63 (2.90–4.58)	4.15 (3.27–5.33)	4.71 (3.65–6.15)	5.40 (4.11–7.16)	6.30 (4.70–8.54)	6.99 (5.13–9.62)
7-day	2.06 (1.77–2.43)	2.48 (2.10–2.95)	3.08 (2.56–3.74)	3.57 (2.92–4.41)	4.29 (3.43–5.41)	4.88 (3.84–6.26)	5.51 (4.27–7.19)	6.27 (4.78–8.31)	7.27 (5.42–9.85)	8.02 (5.89–11.0)
10-day	2.33 (2.00–2.74)	2.79 (2.37–3.33)	3.45 (2.87–4.20)	3.99 (3.27–4.93)	4.76 (3.81–6.01)	5.39 (4.25–6.92)	6.06 (4.70–7.91)	6.84 (5.21–9.08)	7.88 (5.88–10.7)	8.66 (6.36–11.9)
20-day	3.19 (2.74–3.75)	3.82 (3.24–4.55)	4.68 (3.89–5.68)	5.35 (4.38–6.61)	6.29 (5.03–7.93)	7.02 (5.53–9.01)	7.78 (6.03–10.2)	8.60 (6.56–11.4)	9.69 (7.23–13.1)	10.5 (7.72–14.5)
30-day	3.99 (3.43–4.70)	4.78 (4.05–5.69)	5.83 (4.85–7.08)	6.63 (5.42–8.18)	7.70 (6.16–9.72)	8.53 (6.72–11.0)	9.37 (7.25–12.2)	10.2 (7.79–13.6)	11.3 (8.46–15.4)	12.2 (8.96–16.8)
45-day	5.03 (4.33–5.92)	6.03 (5.12–7.18)	7.32 (6.09–8.89)	8.26 (6.76–10.2)	9.49 (7.60–12.0)	10.4 (8.19–13.4)	11.3 (8.75–14.7)	12.1 (9.25–16.1)	13.3 (9.88–18.0)	14.1 (10.3–19.4)
60-day	5.78 (4.97–6.80)	6.96 (5.91–8.29)	8.41 (6.99–10.2)	9.43 (7.71–11.6)	10.7 (8.55–13.5)	11.6 (9.11–14.8)	12.4 (9.60–16.2)	13.1 (10.0–17.4)	14.1 (10.5–19.1)	14.8 (10.9–20.4)

Figure 5: Precipitation Frequency Table

The precipitation frequency data available in graphical format includes depth-duration-frequency (DDF) curves and precipitation frequency curves with 90% confidence limits. **Figure 6** presents the precipitation frequency curves.

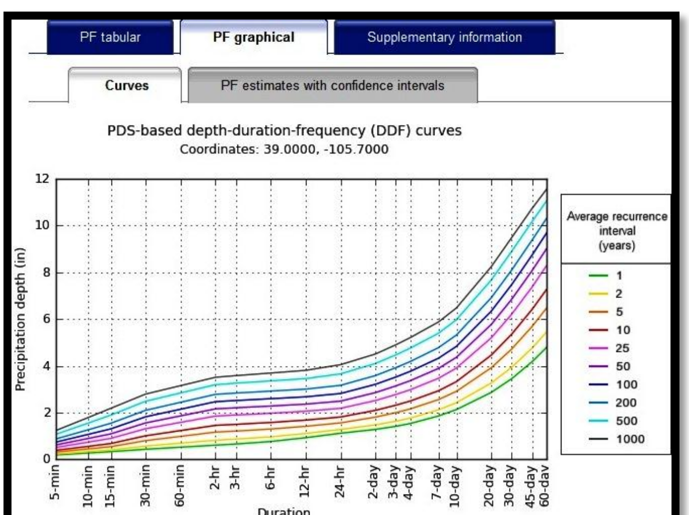

Figure 6: Graphical Precipitation Frequency Curves

Precipitation frequency estimates from NOAA Atlas 14 are point estimates, and reductions should be applied when used for areas. The conversion of a point estimate to an areal estimate is usually done by applying an areal reduction factor, obtained from a depth-area-duration curve, to the average point estimates within the subject area. Currently, the depth-area-duration curves from the U.S. Weather Bureau's Technical Paper No. 29 can be used for this purpose and is recommended by NOAA Atlas 14. The NWS is investigating the areal reduction factors for NOAA Atlas 14 and may issue new areal reduction factors in the future.

Probable Maximum Precipitation Events

The PMP, as defined in the HMR documents, is "theoretically, the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location at a certain time of the year". No recurrence interval is assigned to the PMP.

For Colorado, Montana, Utah, and Wyoming, the PMP east of the Continental Divide is derived using the methodology in HMR 55A; the PMP west of the Continental Divide is derived using HMR 49 or HMR 57. The PMP studies developed by NOAA and NWS are shown in **Figure 7** by geographical location in the United States and are available online through NOAA.

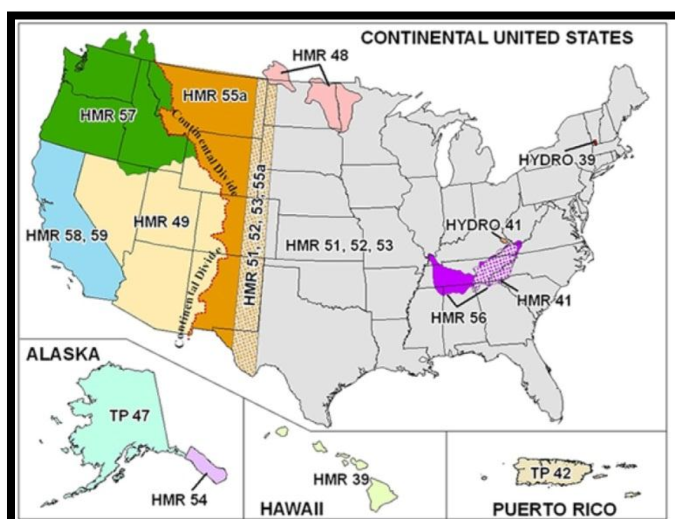

Figure 7: Available PMP Studies

Utah has published two updates to HMR 49:

- "2002 Update for Probable Maximum Precipitation, Utah 72 Hour Estimates to 5,000 sq. mi. - March 2003" (USUL)
- "Probable Maximum Precipitation Estimates for Short Duration, Small Area Storms in Utah - October 1995" (USUS)

These augment, not supersede, HMR 49 and are intended only for use in the state of Utah.

HMRs 55a, 49, and 57 provide precipitation values for a local storm (thunderstorm) with 6 hours of duration and a general storm with 72 hours of duration. The results from both the general and local storm should be used in hydrologic trials to determine the critical design values.

The HMR methods require obtaining index precipitation from maps, and then adjusting precipitation depths for drainage area size, elevation, and orographic effects specific to the watershed being studied.

An alternative to the HMR documents for PMP estimates is a site-specific analysis. Colorado has developed an Extreme Precipitation Analysis Tool (EPAT) and is currently conducting a formal 3rd party meteorological peer review set for completion in April 2014. In general, a site-specific analysis is not readily achievable for small dam owners and because it typically requires a custom analysis by a consultant engineer/meteorologist.

Conclusions

To determine design precipitation depths for precipitation frequency events, NOAA Atlas 2 and/or Atlas 14 are available (Montana also uses USGS WRI Report 97-4004). NOAA Atlas 14 is an update and supersedes NOAA Atlas 2 in Utah and Colorado, while no update is currently planned for Montana or Wyoming. One advantage of NOAA Atlas 14, the precipitation depths can be easily obtained online using NOAA's Precipitation Frequency Data Server.

PMP estimates can be estimated using HMR methods. Precipitation for both the local and general storms is derived for hydrological evaluation. An alternative to the HMR methods is a site-specific extreme precipitation analysis. Site-specific analysis is not easily achieved and typically requires a custom analysis by a consultant engineer/meteorologist.

Selecting the design precipitation is the first step in the hydrologic analysis used to size the dam and spillway. If the dam and spillway are not sized appropriately, an extreme precipitation event can lead to dam overtopping and failure. As the recent precipitation event in Colorado shows, large or rare precipitation events can occur and when they do, the importance of appropriately selecting the design precipitation for a given dam is reinforced.

References (with Links where available)

Colorado Rules and Regulations for Dam Safety and Dam Construction, January 01, 2007.

Colorado Launching Massive Emergency Dam Inspection Program, The Denver Post, by David Olinger, September 23, 2013.

Montana Dam Safety Rules and Regulations, Rule 36.14.502 - Hydrologic Standard for Emergency and Principal Spillways, November 5, 1999.

Regional Analysis of Annual Precipitation Maxima in Montana, USGS Water-Resources Investigations Report: 97-4004, by Charles Parent, 1997.

Utah Requirements for the Design, Construction and Abandonment of Dams, R655-11-4 Hydrologic Design, August 01, 2013.

Wyoming Surface Water Regulations and Instructions, Chapter 5 - Reservoirs, 1977.

NOAA Data Links

Current NWS Precipitation Frequency Documents (Including NOAA Atlas 2 and NOAA Atlas 14):
http://www.nws.noaa.gov/oh/hdsc/currentpf.htm

Current NWS Probable Maximum Precipitation Documents (Including HMR 55A, HMR 49, and HMR 47):
http://www.nws.noaa.gov/oh/hdsc/studies/pmp.html

HDSC Exceedance Probability Analysis for Select Storm Events:
http://www.nws.noaa.gov/oh/hdsc/aep_storm_analysis/index.html

Areal Reduction (Technical Paper No. 29):
http://www.nws.noaa.gov/oh/hdsc/Technical_papers/TP29P4.pdf

Special Series: What the Heck Should Be in My Spec?

Part 1: Earthwork Considerations

A thorough set of technical specifications for any dam construction project helps ensure the owner and regulator that the desired product is attained, provides the contractor with a clear understanding of requirements for bidding, and helps reduce risks for construction claims. There are many considerations for technical specifications that are unique for dam construction projects. In this special series we will present some of the key topics that are distinctive and important for dam specifications through a series of 3 articles:

1. Earthwork Considerations
2. Dewatering and Diversion – Writing "team effort" specifications.
3. The Devil is in the Details – specification tips to make your construction project move ahead smoothly

Purpose of Technical Specifications

Technical specifications, along with the design drawings, are the guiding documents which enable the project to be constructed according to the intent of the design engineer. They provide a roadmap of sorts for the appropriate procedures and processes to be used to achieve the desired end result. Technical specifications, when properly tailored specifically to the project for which they were written, will provide an enveloping description of how the works shown on the construction drawings are to be assembled and any special considerations and conditions which are not readily shown on the drawings. The technical specifications serve the purpose of explaining the drawings, and ensure that a detailed set of instructions are documented for the purpose of implementing dam construction projects in accordance with the current state of practice for civil engineering work involving dams.

It is oftentimes true that both design engineers and contractors will devote the vast majority of their attention to the development and understanding of the detailed construction drawings for a project, while the technical specifications are seemingly relegated to a lesser position of importance. However, construction contracts are almost invariably written stating that, in the event of inconsistency or disagreement between the drawings and specifications, the written specifications take precedence over the drawings. For this reason alone, it is imperative that the technical specifications be written specifically and accurately for the project at hand, and be unambiguous in their content and meaning. The use of broadly-based, standard earthwork specifications which may be based on other forms of heavy civil construction, such as highways or support of structures, may result in rejection by the regulatory agency having jurisdiction, conflicts during construction, or worse yet, a constructed project which utilizes inappropriate construction techniques and methodologies rather than the original intentions of the designer.

This first article of the series will focus on specifications requirements common for earth materials in dam construction.

Filter Placement

Filters and drains are placed in embankment dams to provide for the safe transmission of seepage water through the dam and out the downstream side. As such, they are placed on the downstream side of the impervious portion of the dam, which is constructed of fine grained soils and is referred to as the core. Filters are used to protect the core from movement of soil particles due to seepage forces, while providing some measure of drainage ability. Drains are designed for the removal of water, so must be relatively free-flowing and designed to prevent the migration of granular filter particles into the drain. Design and construction considerations of filters were discussed in Issue 1, Volume 1 of this Technical Note publication (March 2013).

Four primary items are generally specified in contract documents that relate to construction of filter and drainage zones:

1. In-place material gradation, including material quality and durability specifications;
2. Moisture (wetting requirements) – generally requiring the addition of water to the material during handling and prior to compaction;
3. Compaction effort (number of passes with specified equipment) for a method specification or % compaction/relative density for an end-result specification (see discussion later in this article concerning method and end-result specifications); and
4. Geometry (alignment, width, and vertical continuity).

Filters used to protect the core are generally specified to be constructed of sand-sized materials. Gradations should be designed in accordance with current dam practice to provide for both filtration of the base soil and for drainage of collected water. To accomplish this, more uniformly graded sand is preferable to broadly graded materials. In practice, commercially available concrete sand produced in accordance with ASTM C33 is applicable in most cases for protection of a fine-grained base soil. However, this information should always be verified by analysis.

Aggregate quality and durability are other requirements which should be specified for granular filter materials, and those in ASTM C33 are applicable for filters, as well. Specifications for filter sands typically require that filter aggregates shall be "sound, strong, durable, clean, and minimally affected by chemical alteration and physical breakdown, meeting durability requirements for concrete sand." These requirements can be verified by use of the various testing methods for friability, clay lumps, soundness, and impact resistance listed in ASTM C33.

To ensure permeability and the self-healing nature of filters, the presence of fines (-#200 sieve size materials) in filter sand should always be limited to no more than about 3-4% at the source, and the presence of plastic fines should be prohibited altogether. Typical practice requires that particle breakdown during handling and compaction should result in no more than about 5% fines in place. This limit on break down is generally achievable with the typical durability and compaction requirements discussed herein.

Filter and drain materials are not particularly amenable to conventional earthwork compaction density control. Typical filter sand materials do not exhibit the "standard" compaction curve shape, with a clear maximum dry density at specific optimum moisture content. Rather, these materials exhibit their maximum dry densities when compacted either completely dry or nearly saturated. Drain materials, usually uniform gravels, are not influenced in their compactability by the presence of water, and are not suitable for conventional compaction testing or conventional field density testing.

Conventional end-result compaction specifications (e.g., percent compaction specifications such as ASTM D698) are sometimes used for filter and drain materials, but they can be difficult to apply in the field. End-result compaction specifications based on relative density requirements (e.g., ASTM D4254) are also sometimes used, but the relative density test is notoriously difficult to apply in the field. Consequently, method specifications are often used for filter and drain materials. The difference between method and end-results compaction specifications are discussed in a subsequent section of this article.

Photo 1. Placement of a 3-stage filter showing use of hand-held plywood shield to limit cross contamination.

For most applications, the desired degree of compaction of filter and drain materials is such that sufficient strength is attained and settlement is limited. In locations subject to seismic loading, it is also necessary that filter materials be sufficiently dense to avoid the potential for liquefaction if saturated. All of these requirements can be met by compacting to around 70% relative density (ASTM D4254), which is not particularly difficult to accomplish. Overcompaction beyond this point should be avoided, as this can lead to excessive particle breakage and increased fines content, which can negatively affect permeability and the desired self-healing nature of filters.

In general, it is easier to use a method specification for filter and drain materials, in which a minimum size and weight of compaction equipment, and a minimum compaction effort (e.g. number of passes with the required equipment), are specified. In addition to the compaction equipment and effort, it is also recommended that the placement specification for the filter include wetting the material both during handling, which may help prevent segregation, and prior to compaction. Compaction is most effectively achieved when water is added to the filter material as it is placed to produce a moisture content near saturation. This can be effectively accomplished with a water bar mounted on the compactor or by applying water with a water truck or hose just in advance of the compactor. The filter material should not be oversaturated in locations where the water cannot readily flow away during compaction. Vibratory compaction equipment, such as smooth drum vibratory rollers, should be specified for compacting granular filter and drain materials in order to achieve uniform, complete compaction.

Construction of filters and drains within dams generally requires that the designed width and alignment of these features conform to the types and methods of construction to be used. As a practical issue, alignments of filters and drains should be kept as reasonably straight as possible across the width of the dam section to ensure that continuity of those features is maintained, and thicknesses/widths of filters and drains should be specified to match up with the types of construction equipment to be used. Maintaining alignment, width and vertical continuity of filter and drain zones is of vital importance, and should be covered in detail within the specifications.

Placement specifications should be written to provide for accurate surveys of filter and drain locations during construction, so that these locations are reasonably certain during fill placement operations. The correct geometry must be maintained at all times to ensure vertical continuity of filter and drain zones. Accurate and precise placement of filter material, in lifts of limited thickness, can help prevent the development of “Christmas tree” shaped filter zones within the embankment, thus minimizing the expense of placing excess filter material while ensuring that the design width of the filter is maintained. Some degree of variation in the filter boundaries will occur despite the best efforts of the contractor and specified widths should be sufficient to maintain continuity with an expected variation in these boundaries.

To prevent the potential for contamination of filter and drain zones, placement and compaction of materials in those zones should be advanced one lift thickness ahead of materials in surrounding core and shell zones, to ensure that surface drainage is away from the filter/drain. Also, traffic of construction equipment across the filter and drain zones should be eliminated or very carefully controlled to prevent contamination of the surface. Those areas where traffic over the filter/drain zone is allowed require special treatment to remove contaminated granular materials before the next lift is placed.

Core Placement

Specifications for low permeability core materials generally need to consider the following:

1. Minimum fines content and plasticity
2. Moisture requirements
3. Compaction requirements
4. Special compaction
5. Protection from drying or overwetting

Embankment core sections are generally constructed of the most fine-grained, highest fines content soils available on or near site, although there are some exceptions which may arise due to unworkable materials. Specifications should require that core materials possess a certain minimum content of fines (minus #200 sieve size fraction). It is also desirable that the fines maintain a required minimum plasticity, as measured by Atterberg limits; however, in some locations soils with plastic fines may not be available. Embankment cores can be successfully constructed with soils with very low or non-plastic fines, but precautions such as wider cores, more robust filters, and material test pads are appropriate in these cases. There is no absolute limit on any of these criteria, but the goal is to obtain a relatively watertight core which maintains some measure of flexibility under loading. Core sections can be constructed with a broad range of material; ranging from material comprised of nearly all fines to material containing as little as 20-30%. The amount of preferred fines will depend on the plasticity and coarseness of the remaining material gradation. Generally, clay materials of low to moderate plasticity are preferred, as they are quite impermeable and maintain good workability characteristics. Sandy clay soils and clayey sands can also provide a very desirable core section, of both high strength and impermeability. However, as noted above silty sands, silty sands and gravels, and even low plasticity silts can potentially be used with appropriate precautions.

Problems of workability can arise if fine grained materials having liquid limits in excess of 50% (CH and MH soils) are allowed.

Moisture contents for compacted core material should be specified over the range at which optimal compaction can be best achieved, while still maintaining satisfactory plasticity of the fill. For clay materials, this will generally be between 2% below and 2% above optimum moisture content, as defined by ASTM D698. For silty, lower plasticity materials, somewhat lower moisture content is desirable, in the range of 3% below to 1% above optimum moisture, per ASTM D698.

Compaction requirements for fine-grained embankment fill materials such as clay core materials are almost universally defined by end-result based specifications rather than method specifications, due to the well-established relationship between moisture content and compacted density under a given compaction effort, and the relatively straight forward means by which the state of compaction is measured. Generally, compaction specifications will be defined by requiring 95% of standard Proctor maximum dry density (relative compaction), as measured by ASTM D698. Under some conditions, such as fill under rigid structures, greater density and resistance to settlement may be desirable, and a higher percentage of relative compaction, such as 98%, may be specified. Alternatively, modified Proctor (ASTM D1557) standards may be used for structural support fill, in which case the required percentage of compaction should be decreased a few points, to 95%. Control of embankment core material using modified Proctor is not commonly used for embankments, due to the shifting of the lower moisture contents required to achieve the higher modified Proctor densities, which has an undesirable effect on core ductility. In addition, experience has shown that the greater compactive effort required to achieve modified Proctor compaction is not generally required for acceptable embankment performance.

In addition to the required density and moisture content, acceptable compaction equipment and methodology should be specified. For core materials, this would appropriately involve the kneading action of sheepsfoot or pad foot rollers for mass fill areas. Use of a sheepsfoot or pad foot roller will result in a more homogeneous fill which is compacted from the bottom up, leaving a rough surface for the next layer to adhere to, with less tendency to produce laminations in the fill.

In all cases, the specifications should require that placement and compaction of core materials be done in the longitudinal direction parallel to the dam axis rather than across the axis, to avoid the potential for non-uniform fill materials or laminations creating preferential seepage paths through the embankment. Each succeeding lift must be well-bonded with the preceding lift by ensuring the proper fill placement moisture content, and, where necessary, scarifying the preceding lift to prevent slick surfaces which may cause laminations in the fill. Core fill placement specifications also typically require that fill placement shall advance relatively evenly along the length of the core zone, to help prevent the potential for transverse shear surfaces or poorly compacted zones within the fill.

Photo 2. Core Compaction. *Note compacting in paths parallel to dam axis and the hand tamper in photo that will be used to compact zone immediately next to wall.*

Almost invariably, there will be areas within the core section, such as at contacts with outlet conduits or other structures and at the contact areas with rock foundations and abutments, where the equipment used for compacting the mass fill areas is not suitable. These areas are referred to as special compaction areas, and should be addressed with their own specification. Compacted density requirements should not be compromised in these areas, but it may be desirable to maintain soil moisture contents on the wet side of optimum to ensure the plasticity of the fill, so that it readily deforms to the shape of the surface contacted. Rubber-tired compaction equipment, such as heavy front-end loaders, should be used where possible in these areas, rather than sheepsfoot rollers, to avoid damaging foundation and abutment surfaces and to permit compaction of soils directly against structures. Smaller, hand-operated compaction equipment may be necessary in more confined areas, but their use should be minimized as much as possible, and lift thicknesses should be reduced accordingly to allow for full effectiveness and uniformity of the compactive effort.

The specifications should also provide for protecting the placed core material from excessive drying, overwetting, and freezing. Any areas that are allowed to dry excessively should be scarified, watered and recompacted to ensure that subsequent lifts can bond adequately. Similarly, if the core zone is exposed to excessive rainfall or ponding of water on the surface, it may be necessary to scarify the wet material and allow it to air dry to an acceptable moisture content prior to recompaction, or, in some cases, completely strip the overwet material prior to proceeding with subsequent fill placement. The specifications should also address preventing the incorporation of frozen materials within the embankment, and the protection of placed fills from freezing.

Method Specifications versus End-Result Specifications

For earthwork projects, specifications may be written to require either a specific methodology to achieve a desired result (“method specification”) or to require a certain specific outcome which is verified by testing (“end-result specification”).

A method specification may be appropriate if limited material is being placed, or if testing of materials is difficult or too time-consuming for real-time test results. A typical usage of a method specification would be to control the placement and compaction of granular materials, such as would be used for filters and drains within dams. Since these types of materials do not exhibit the type of moisture/density compaction behavior typical of fine-grained soils, test procedures developed for fine-grained soils are not generally applicable, and the types of tests which have been developed to determine placed densities can be somewhat problematic. Method specifications are, therefore, often more appropriate for controlling the placement of these materials. A method specification would typically specify a required type and amount of effort to be expended to achieve the desired result, without necessarily testing for the result.

Method specifications are usually verified by requiring the contractor to perform a scaled test pad using proposed source materials and equipment. The test pad places the material in accordance with the proposed method specification. Testing of the in-place compacted materials including gradation (for particle breakdown) and in-place density are performed to confirm the method specification achieves the desired results.

Photo 3. Test Pad for Granular Filter Materials

Below is an example excerpt from a specification for placement and compaction of a sand filter. It is not all encompassing, and only provides an example portion of typical requirements, and would be tailored for project specific requirements by the design engineer.

"Place, spread, and compact Zone 2 material parallel to the embankment axis and in such a manner as to avoid cross contamination of adjacent zones…Place and spread Zone 2 material in level, continuous, approximately horizontal lifts that do not exceed 12 inches in thickness before compaction….Thoroughly wet Zone 2 material at the time of compaction in such a manner as to achieve uniform moisture throughout the lift…compact each lift on Zone 2 with 4 coverage's, as is defined in these specifications, of an approved 20-ton smooth drum vibratory roller, or with the number of passes based on the test pad program result…"

End-result (or QC-based) specifications are more typically and effectively used to control the compacted density of fine-grained, especially cohesive, embankment materials. Verification of the desired outcome is obtained by QA/QC testing of the completed product against a required minimum standard, such as a percentage of standard Proctor maximum dry density within a range of acceptable moisture contents. This is the most common type of earthwork control specification for clay and silt core materials where in-place density testing can be readily performed with real-time results using nuclear density testing gauges.

Below is an example excerpt from a specification for placement and compaction of a core material. It is not all encompassing, only provides an example portion of typical requirements, and would be tailored for project specific requirements by the design engineer. It does not present associated requirements including, but perhaps not limited to, protection from cold and wet weather, limits on exposure time of unworked surfaces, discing of clumps, scarifying for adequate tie-in of layers, special compaction at contacts, etc.

"Place, spread and compact Zone 1 material parallel to the embankment axis… Place and spread Zone 1 material in level, continuous, approximately horizontal lifts that do not exceed 8 inches in thickness before compaction…compaction water content of Zone 1 shall be between minus 1 and plus 3 of the optimum water content in accordance with ASTM D698… moisture conditioning shall be performed in the borrow area or at the stockpile to the extent possible…Zone 1 material shall be compacted to at least 98 percent of the maximum dry density (unit weight) as determined by ASTM D698.

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