CHAPTER 1 - INTRODUCTION

Intermountain Regional Landfill (IRL) is interested in pursuing a more efficient design of their Class V landfill by achieving a better balance between total required cut and fill. With this goal in mind, IRL requested that Hansen, Allen & Luce, Inc. (HAL) perform a complete redesign of the floor and closure cap within the previously approved landfill footprint while maintaining the same maximum height requirement of 100 feet above the existing topography. This report provides a summary of background information associated with the design of the landfill floor and closure cap, proposed design modifications to other associated features, and results from the engineering calculations for the proposed modifications. The engineering calculations are included to provide a basis for approval of a permit modification from the Utah Division of Waste Management and Radiation Control (DWMRC).

The proposed permit modification includes the following design modifications:

1. Floor elevations and leachate collection system, including sumps and leachate withdrawl pipes.
2. Closure cap final grade, including access, benches for runoff and erosion control, downspout drainage piping to remove runoff from the top of the closure cap and benches.

Locations and configurations of some other on-site facilities to support landfill operations were also modified to provide a general concept layout plan regarding the types of facilities needed. These facilities include a potential leachate management pond, parking areas, and soil stockpile areas. The locations, sizes and configurations of these facilities are not critical to the design requirements associated with the landfill and its closure. Therefore, it is understood that the types and locations of proposed support facilities may be modified from those presented herein.

CHAPTER 2 – MODIFIED LANDFILL FLOOR DESIGN

This chapter presents the general layout and design concept of the landfill floor systems, which includes more specific information for the leachate collection and removal system components and runoff/run-on containment. References to the permit drawings in Appendix A, geotechnical report in Appendix B, the original slope stability and settlement analysis completed by HDR in Appendix C, and calculations provided in Appendices D and E should be noted throughout this chapter.

PREVIOUS DESIGN AND CURRENT LANDFILL STATUS

The original design of the landfill was completed by HDR, Inc. and presented in a design engineering report dated November 2010 that was included in the permit application as Part 3. The original capacity of the facility was listed as 27,000,000 cubic yards. As of June of 2016, several cell construction projects covering approximately 20 acres have been completed in the western portion of what was previously referred to as Cell 1 and Cell 2 in the original permit drawings.

GENERAL LAYOUT AND DESIGN

The redesign of the facility consists of a landfill area formed by incised embankments along all sides of the facility with a floor system concept very similar to the previous design. The facility has three main cells with each containing its own leachate collection system, sump and leachate withdrawal system. In order to improve on the previous design concept, the floor elevations were raised to reduce the amount of excavation required. The floor slope was reversed for the undeveloped portion of Cell 1 and taken to the north in order to minimize excavation and provide an accessible location for a leachate sump and withdrawal system. The other two cells function very similarly to the previous design with the only major change being the elevations. Cells 2 and 3 are both similar in design which drain down the center of the cell to a sump located on the far east end of the cell floor. Cell 1 differs because it includes the 20 acre area that has already been developed. Cell 1 needed to be altered because the previous design placed the floor trajectory toward the east which would have made the excavation much deeper.

The overall capacity above the protective soil cover material placed above the lining system is about 28.9 million cubic yards, slightly more than the previous design. This does not include the final cover system. The design modification maintains the previous horizontal footprint and makes grade adjustments to the floor and closure cap that do not increase the overall height above existing topography (100 feet) and actually decreases the maximum height from floor to the top of the closure cap. The slight increase in capacity was not the intent of this modification and also cannot be attributed to either horizontal or vertical expansion. The increase is a byproduct of a change in the geometry of the waste mound that produces a more favorable cut/fill balance while maintaining the original design constraints.

FLOOR ELEVATIONS AND SLOPES

The floor elevations were raised significantly from the previous design to decrease the amount of cut required for the facility. This resulted in the distance to ground water being much greater than the minimum 5 feet as required in R315-302-2(e). The bottom liner has a 2% slope, consistent with the previously approved design and with R315-303-3-3a(ii) that specifies minimum liner slopes. The slope of the leachate collection system piping was set at a minimum of 1% but ranged to as high as 2% in some areas.

As part of the original Technical and Engineering Report completed by HDR in 2010, a slope stability analysis and settlement analysis entitled “Slope Stability and Settlement Evaluation” was completed and included as Appendix C. This evaluation included a review of the site conditions, static, pseudo-static stability and deformation analyses, and settlement and liner strain. The modified design has maximum cut slopes and maximum fill slopes that are equal to or less than those considered for the stability and seismic analyses completed by HDR. Therefore, the geometry of the design is considered acceptable from a stability standpoint based on the previous evaluation, which is found in this report in Appendix C.

The differential settlement and liner strain calculations were updated due to the modification of the overall closure geometry and changes to the leachate collection piping slopes. This update was necessary to ensure that the leachate collection system would maintain a positive slope toward the sump given differential settlement once final waste grade has been reached. A summary of the results are found in Appendix D.

LINING SYSTEM

A composite lining system is proposed for the landfill disposal area consisting of a Geosynthetic Clay Liner (GCL) overlain by a 60-mil HDPE geomembrane liner. This system was approved as part of the permit issued in 2011 and has been used on each landfill construction project to date.

An extra GCL and 60-mil HDPE geomembrane are proposed for placement in the sump areas directly above the GCL and HDPE geomembrane placed across the rest of the cell area. This extra GCL and geomembrane provide added protection against leakage in the sump areas which is the most vulnerable area for leakage to occur. Geosynthetic materials placed on interior slopes of the landfill area will consist of reinforced GCL and textured HDPE geomembrane liner. Geosynthetic materials placed across the cell floor may be unreinforced GCL with a smooth HDPE geomembrane.

Geosynthetic Clay Liner (GCL)

Hydraulic equivalency of the geosynthetic clay liner was included as part of the original permit application and was approved by the Executive Secretary as required by UAC R315-503-3(3)(a)(ii) as stated in the current permit.

HDPE Geomembrane Liner

HDPE geomembrane is proposed for use as the synthetic liner system above the GCL. The floor area will consist of 60-mil smooth HDPE geomembrane and the interior slopes will consist of 60-mil textured HDPE geomembrane to increase slope stability for materials placed on the side slopes above the HDPE geomembrane.

LEACHATE COLLECTION AND REMOVAL SYSTEM (LCRS)

A leachate collection and removal system (LCRS) will be constructed consisting of a geocomposite (geonet bonded to a geotextile) placed directly over the HDPE geomembrane liner. The geocomposite on the floor will be a single sided geocomposite on the floor and a double sided geocomposite on the side slope. Perforated leachate conveyance pipes surrounded by gravel will be placed along the collection areas where the floor surfaces come together with additional conveyance collection pipes provided at specified intervals. The leachate will then be directed to the leachate sumps for removal.

HELP Model

The Environmental Protection Agency's (EPA) Hydrologic Evaluation of Landfill Performance (HELP) model is a quasi-two-dimensional hydrologic computer model used for conducting water balance analyses of landfills, cover systems and other solid waste containment systems. The model accepts weather, soil and design data, and uses solution techniques that account for the effects of surface storage, snowmelt, runoff, infiltration, evapotranspiration, vegetative growth, soil moisture storage, lateral subsurface drainage, leachate recirculation, unsaturated vertical drainage, and leakage through soil, geomembrane and/or composite liners.

The evaporation and solar radiation values for the modeling effort were obtained from default data contained within the HELP model software corresponding to the Salt Lake area. The precipitation and average temperature data used in the model came from the data reported in the Western Regional Climate Center database for Fairfield, Utah.

Five layers were defined in the HELP model corresponding to municipal waste material, soil cover, geocomposite, HDPE geomembrane and GCL to represent the open cell area. An additional three layers were added above the waste consisting of HDPE geomembrane, soil cover material and the erosion protection layer to represent closed portions of the landfill. Model default data were used to define the physical properties of the individual design layers. Leachate quantities were generated for the landfill under the following conditions: no waste, waste thicknesses of 10 feet, 50 feet, 100 feet, 121 feet and at closure. These different depths of waste were used to simulate leachate production at various stages of landfill development. Table 2-1 provides the leachate quantity values generated by the HELP model that were the basis for the LCRS design.

TABLE 2-1
HELP MODEL GENERATED LEACHATE RATES

Waste Height (feet)	Peak Daily Leachate	Annual Average Leachate
	(inch)	(inches)
No Waste	0.204	0.134
10	0.161	0.571
50	0.157	0.571
100	0.160	0.571
121	0.158	0.571
Closure	0.001	0.013

Geocomposite

A geocomposite will be placed above the HDPE liner to collect and convey leachate from the floor area to the leachate conveyance pipes that convey the leachate to the sumps for removal. The geonet component of the geocomposite was designed based on the peak daily leachate rate of 0.162 inches/day. The design of the geonet was completed based upon a one-foot wide section of geonet over the longest flow path for the facility. The longest one-foot wide flow path is 460 feet from the leachate conveyance pipe to the upper end of the surface at its longest reach. This will provide the longest flow path and a typical design that can be applied to all areas of the floor.

The longest flow path for each cell is presented in Table 2-2. “Designing with Geosynthetics”, by Robert Koerner, suggest several safety factors that should be applied to the leachate rate to obtain a design capacity for the geocomposite. These safety factors include: 1) creep deformation of the geonet 2) biological clogging of the geonet and 3) chemical clogging of the geonet. Therefore, a total safety factor of 4.5 was used for the design of the geocomposite. Applying this resulting safety factor to the leachate rate gives the design transmissivity requirement presented in Table 2-2.

TABLE 2-2
REQUIRED PROPERTIES FOR GEOCOMPOSITE

Cell	Longest Flow Path	Peak Daily Flow	Transmissivity Requirement
1	426 ft	5.7 ft^3/ft-day	1.38 x 10^{-3} m^2/sec
2	418 ft	5.6 ft^3/ft-day	1.36 x 10^{-3} m^2/sec
3	460 ft	6.2 ft^3/ft-day	1.49 x 10^{-3} m^2/sec

Geotextile Filter Fabric

Geotextile will be used as part of the geocomposite above the HDPE liner and around the leachate conveyance piping on the cell floor in order to provide a filter layer between the soil cover and the LCRS. Gradation properties of the native soil were provided by Applied Geotechnical Engineering Consultants (AGEC) and used for the calculations.

TABLE 2-3
REQUIRED PROPERTIES FOR GEOTEXTILE FILTER FABRIC

Property	Standard
Equivalent Opening	≤ 0.22 mm
Permeability	≥ 1.35x10^{-2} cm/sec
Grab Tensile Strength	≥ 90 lbs

Leachate Conveyance Pipes

The leachate conveyance pipes are designed to be placed along the valley of the cell floors that are formed by the intersection of the planar surfaces of the floor. Additional leachate pipes along the toes at the north and eastern ends as well as along the planar surfaces of the floor at specified spacing are also included to provide for adequate drainage given the assumed limitations of the geocomposite. These leachate collection pipes receive leachate from the geocomposite and convey the leachate to the sumps for removal.

The maximum leachate rate calculated using the HELP model was applied to the maximum width and length of floor area where leachate will be collected in the geocomposite for each planned leachate pipe. The contributing area for each leachate pipe varies due to different factors, including cell layout, especially the difference between Cell 1 and Cells 2 and 3. The peak leachate rate of 0.161 inches/day for the cell floor and 0.124 inches/day for the slopes was applied to each contributing area.

Eighty percent of the maximum flow depth in the pipe was assumed for the actual capacity for each pipe using Manning's equation with a Manning's n roughness value of 0.016. A detailed breakdown of the capacity calculations for each leachate pipe is provided in Appendix E.

Landfill Leachate Withdrawal Pipes

Leachate withdrawal pipes were evaluated for wall crushing, wall buckling, and ring deflection using published procedures. Overburden loadings were determined based on the loading over the low point at the sump at closure. The calculations for the determination of pipe's ability to withstand wall crushing, wall buckling, and deflection are found in Appendix E. It was found that the 24-inch HDPE leachate withdrawal pipes specified in the drawings provide sufficient strength under the ultimate load.

Leachate Pond

Leachate will generally be contained and managed within the landfill where the sumps will be pumped when necessary and the leachate will be either used for dust control or placed in active phases of the landfill where leachate containment is provided. Currently, leachate production levels are far below the levels that would be anticipated according to the HELP model predictions. Leachate production rates will be tracked in order to provide a basis for the sizing of the leachate pond. When management of leachate production levels approaches the capacity of the facility to contain the leachate within lined areas, a leachate pond will be designed to be located in the northeast corner of the property as depicted in the permit drawings.

The leachate pond lining system will include a composite secondary (bottom) lining system constructed of GCL overlain by a 60-mil HDPE membrane. A leak detection and removal system consisting of a geonet, a sump and a leachate withdrawal pipe will be placed above the secondary lining system. A primary (upper) lining system consisting of a 60-mil HDPE geomembrane will be placed above the leak detection system above which the leachate will be stored.

RUNOFF CONTAINMENT

Precipitation runoff from the waste material in open areas of the landfill will be contained and managed within the lined landfill area. Containment areas may be formed on waste surfaces and/or by maintaining waste set-back areas where runoff water will be contained between the top of lined embankment and the waste mound. Sufficient capacity will be maintained in these areas to contain runoff from the 25-year 24-hour precipitation event as required by the regulations.

GROUNDWATER MONITORING WELLS

Groundwater monitoring wells were installed prior to the first construction phase at the facility. This permit modification does not include any modifications to the existing groundwater monitoring wells. Since the landfill footprint has not be modified, no changes to the groundwater monitoring well locations should be required.

GEOTECHNICAL INVESTIGATION

A geotechnical study was completed by Earthtec Testing and Engineering, P.C. dated October 13, 2006. Based on that study, HDR Engineering, Inc., who were responsible for the original permit application submitted in 2010, completed the slope stability, seismicity and settlement evaluations. The proposed design modifications do not go outside of the parameters considered as part of HDR's slope stability and settlement evaluation. The maximum cut slope evaluated by HDR was based on a maximum of 50 feet whereas the modified design has a maximum cut slope of 37 feet. The maximum fill slope evaluation completed by HDR was based on a maximum waste fill height of 130 feet and 100 feet above existing grade (although the design only had 80 feet of height over existing grade) at the toe whereas the modified design has a maximum height of 126 feet and maintains the 100 feet above grade with the same 4H:1V slopes as the original design. The maximum operational fill slope evaluated by HDR was 130 feet at a 3H:1V slope which is consistent with the height of 126 feet achieved under the modified design.

The differential settlement calculations were revised due to the change in the geometry of the closure cap and floor which was completed using the same methodology used in the original evaluation completed by HDR. Those calculations are found in Appendix D.

CHAPTER 3 – MODIFIED CLOSURE DESIGN

This section presents the general layout and design concept for the landfill closure system. The geometry of the closure design was modified in order to offset the airspace capacity lost from below existing grade due to the reconfiguration of the floor elevations. This allows for a more efficient use of the available footprint.

GENERAL LAYOUT AND DESIGN

The final waste mound with the overlying layers of daily cover material provides the subgrade for the final closure system. The final cover system for the modified design is consistent with the final cover system that was approved as part of the original permit application in 2010. The cover system consists of 18 inches of intermediate cover, a textured 60-mil HDPE liner followed by 2 feet of final soil cover. The two feet of cover material includes soil fill and an erosion protection layer consisting of native vegetation. A discussion of the erosion protection measures is provided in Chapter 4.

Closure Slopes

Waste mounding and the overlying closure cap extends up on a 4H:1V slope from the top of the floor embankment slopes around the perimeter of the landfill area. An intermediate bench (25 feet wide) is designed into the 4H:1V slopes to provide for intermediate storm water collection and conveyance necessary for erosion protection on the slopes around the facility. The waste mound and closure cap rise to an elevation of about 100 feet above the top of the west cut slope. The waste mound and closure cap then break grade to a 2 percent slope extending to the east. The north, south and east slopes extend upward on 4H:1V slopes from the top of the incised embankments to intersect with the top surface as it extends east on the 2 percent slope.

Sub-Surface Drainage

Some storm water may infiltrate through the cover system and collect on the surface of the HDPE membrane. A drainage system consisting of a perforated drain pipe will be installed underneath the storm water containment berm on the east side of the top of the waste mound. The drain pipes are placed in drain rock with a geotextile wrap around the drain rock. These pipes are provided to drain water that is conveyed along the HDPE liner before it reaches the side slope. Additional perforated drain pipes will be placed under the intermediate bench located on the 4H:1V slopes and will be conveyed to either the downspout pipes directly or run parallel to the downspouts in a separate solid pipe to the exterior toe of the landfill on the east side.

STORM WATER MANAGEMENT

The storm water management system consists of a 2 percent slope at the top of the landfill that directs precipitation runoff from the top surface of the closure cap toward the east. Runoff is then collected and directed to storm water downspouts using a storm water containment berm that directs water to inlet boxes and into parallel 18-inch storm drain pipes. The downspouts convey the storm water from the top of the closure cap to the exterior toe that discharges into an energy dissipation basin where it will then exit the property to the east.

The intermediate bench is located on the 4H:1V perimeter slopes of the closure cap primarily to shorten the length of the 4H:1V slope for erosion control purposes. The intermediate bench also provides storm water conveyance that is collected at inlet boxes and to parallel 18-inch diameter downspout pipes located at the northeast and southeast corners. The storm water management system associated with the closure cap is designed for the 25-year 24-hour precipitation event. Design of the storm water management system, including the hydrology, hydraulic design of the downspout pipes and erosion control is presented in more detail in Chapter 4.

STABILITY

The stability of the closure cap design was originally evaluated by HDR based on information provided in a geotechnical investigation completed by EarthTec Engineering. Although the geometry of the closure cap is changed from the original design, the design parameters are all within the values used for the prior evaluation, including maintaining the exterior slopes at 4H:1V and keeping the overall height of the slope within 100 feet vertical from the base of the slope. The maximum cut slope was also maintained below the height contemplated during the previous evaluation. Because the current design is not outside of the parameters used in the original slope stability evaluation, the previous evaluation is incorporated by reference for use in the modified design.

CHAPTER 4 – STORM WATER MANAGEMENT

A diversion channel/berm will be constructed to manage storm water from the tributary area to the west of the facility. A berm on the top of the closure cap as well as an intermediate bench will convey storm water to downspouts that will take the water off the closure cap. A hydrologic analysis was completed in order to determine peak flow rates to use for the design of the channels, downspouts and erosion control.

HYDROLOGY

Hydrologic calculations were completed for the tributary area to the landfill and the closure cap to determine peak runoff for design purposes. The Soil Conservation Service (SCS) curve number methodology was used in conjunction with the Army Corps of Engineers HEC-HMS hydrology model to predict the peak flows.

Off-Site Run-On Storm Water

Storm water that originates from outside of the landfill facility will need to be diverted in order to prevent water from entering the facility and from eroding the base of the closure cap.

Methodology. Storm drainage diversions extending to the north and south on the western edge of the landfill property will collect and convey storm flows around the facility. Tributary areas to the diversions were delineated based on USGS topographical maps.

Curve numbers were determined based on the hydrologic soil type and soil vegetative cover. The hydrologic soil type is a general indication of the soil's infiltration capacity. Soils are assigned a hydrologic soil type of A, B, C or D by the Natural Resource Conservation Service (NRCS). Soils of hydrologic soil type A have the highest infiltration rate, and therefore produce the least amount of runoff. Soils of hydrologic soil type D have the lowest infiltration rate, and therefore produce the highest amount of runoff. Most of the soils within the tributary area are hydrologic soil type C or D with smaller portions of B and A type soils. The soil vegetation cover and conditions were assumed based on information provided from a custom soil resource report for Fairfield-Nephi Area, Utah and Tooele Area, Utah and verified with a field visit. The cover conditions were combined with the hydrologic soil type to produce a curve number based on Table 2-2d of Technical Release 55 "Urban Hydrology of Small Watersheds" (TR-55). The entire tributary area was combined in one subbasin where an area weighted curve number was applied to the total area.

The lag times (T_L), defined as the time to the hydrograph peak, were calculated by using the time of concerntration (T_C) and the equation T_L – $0.6T_C$. The time of concentration was calculated using the criteria found in Worksheet 3 in TR-55.

The SCS Type II distribution was used to model a 24-hour 25-year storm, consistent with the requirement of R315-305-4-3(a). The rainfall amount was taken toward the higher elevations of the tributary area from the "Point Precipitation Frequency Estimates from NOAA Atlas 14".

The magnitude of the area tributary to the landfill site is large enough to warrant the use of a reduction of the precipitation value due to the likelihood of the full amount of the storm affecting the whole region decreases with an increase of tributary area. The factor is based on information from the Salt Lake City Hydrology Manual. According to the manual, a 24-hour event has an areal reduction factor of:

$$ARF = 0.01(100 - 2*Area^{0.46}) \text{ where}$$

$$Area = \text{Total Tributary Area, } 6.69 \text{ mi}^2$$

$$ARF = 0.95$$

This reduction factor was applied to the tributary area for run-on calculation purposes.

Peak Design Flow. Hydrologic calculations presented above were used to generate a peak flow of 61 cubic feet per second (cfs) for the run-on from the tributary area to the west of the facility. The calculations and summary of methodology and results are presented in Appendix F.

On-Site Run-Off Storm Water

Storm water will need to be conveyed off the landfill facility in order to protect the integrity of the closure cap.

Methodology. Delineation of the subbasins from the closure cap, shown in the figure included in Appendix F, was based on the cell closure cap design discussed in Chapter 3. Each subbasin is designed to drain either directly off of the closure cap or to a downspout that conveys flow away from the facility.

A curve number was determined based on the hydrologic soil type C found at the facility and assumed to be used as part of the cover system. The cover type was assumed to be similar to a dirt road in order to account for the period before vegetation becomes fully established. The cover conditions were combined with the hydrologic soil type to produce a curve number of 87 based on Table 2-2a of TR-55.

The lag times for each subbasin were calculated using Worksheet 3 in TR-55 with a minimum lag time of 5 minutes being applied to subbasins where the calculated value was less than 5 minutes.

The SCS Type II Distribution was used with the 25-year 24-hour storm, consistent with the requirement of R315-305-4-3(b). The rainfall amount was taken from the "Point Precipitation Frequency Estimates from NOAA Atlas 14" based on a location defined at the center of the landfill facility. The value of the 25-year 24-year event is 1.74 inches.

Peak Design Flows. The hydrologic analysis presented above was used to generate peak design flows for each of the subbasins defined for the closure cap and for the downspout piping located at points along the east side of the closure cap. The calculations and summary of methodology and results are presented in Appendix F.

HYDRAULIC DESIGN OF CHANNELS

The peak flow rates based on the hydrology discussed above provided the basis for the design of the drainage conveyances. An existing run-on diversion channel on the west side of the property has been constructed based on the design provided in the original design. This channel/berm will be extended to the south as construction of the landfill continues to ensure that run-on is directed around the facility.

DOWNSPOUT DESIGN

Hydrologic calculations for run-off described above were used to design the downspouts. The design is based on a combined peak flows ranging from 23 cfs for the northeast bench downspout to 28.5 cfs for the northeast top downspout.

Downspout sizes were determined used inlet control conditions and selecting the size and head water depth requirement using the orifice equation. Inlet control conditions were assumed because at peak flow supercritical flow in the system on the 4H:1V slopes and the elevation differences between the inlet and outlet ends of the downspout pipes will not allow for outlet conditions to control.

The downspout pipes were sized based on calculations provided in Appendix F. Two 18-inch pipes in parallel are to be installed for each of the four downspout locations. The headwater depth requirements are provided within the inlet boxes below the grating with additional depth and freeboard provided by the grading of the benches and the berm height on the top of the closure cap.

EROSION PROTECTION

Long term options to provide erosion protection generally consist of establishing vegetation or by placing a stone mulch. In this case, the establishment of vegetation is the selected erosion protection method. Procedures presented in “Erosion and Sedimentation in Utah – A Guide for Control”, a Utah-specific publication, were used to determine the adequacy of the vegetative system. The detailed calculations are found in Appendix G. According to the calculations, a native vegetative cover of approximately 30% on the 4H:1V slopes will provide adequate protection against erosion and the top of the closure cap with the 2% slope provides adequate protection due to the minimal slope, although vegetation will be established there as well.

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