A Review and Evaluation of
Proposed Sunrise Mountain Landfill Covers

Prepared for
Science Applications International Corporation
11251 Roger Bacon Drive, MS R4
Reston, VA 20190

SAIC Subcontract Agreement
Contract 68-C-00-179
Task Order 4, Work Order 2

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I. Introduction and background

Purpose of report

This report was prepared by Anderson-Hydro under contract to Science Applications International Corporation (SAIC) to support the United States Environmental Protection Agency (EPA). The report consists of a review and evaluation of two proposed covers for the Sunrise Mountain Landfill near Las Vegas, Nevada. One cover was proposed by Republic Services of Southern Nevada (Republic), and one by EPA. The report focuses on our opinion as to the adequacy of the cover designs to limit water flux through the cover and erosion of the cover surface.

The report is based largely on review of existing documentation provided to Anderson-Hydro by the EPA. In addition, a visit to the Sunrise Mountain Landfill was made on June 24, 2004. At that time, there was an opportunity to discuss the Sunrise Mountain Landfill and the cover plans with representatives from both EPA and Republic. In addition, there were conference calls with EPA personnel regarding cover related issues on May 21, June 14, June 29, July 13 and July 23, 2004.

Status of Sunrise Mountain Landfill

The Sunrise Mountain Landfill received approximately 25 million tons of municipal waste from perhaps as early as 1951 to 1993. The site was leased by Clark County from the USDI Bureau of Land Management (BLM). The site was operated from 1975 to 1997 by Disposal Urban Maintenance Processing Company (DUMPCO) and then by Republic. In 1994, DUMPCO, Clark County and the BLM agreed to a closure plan for the site that was to be in agreement with applicable laws and regulations.

In 1995, the BLM identified concerns about the closure of the landfill, specifically the thickness of the final cover and the functioning of some surface water structures. In 1997, Dwyer issued a report for the BLM concerning the adequacy of the closure. They concluded that: the closure was inadequate with respect to surface water control; erosion and infiltration were not minimized by the cover; the cover did not meet regulatory requirements (specifically 40 CFR Part 258); methane gas exceeded the explosive limit at one location; and the gas and groundwater monitoring were inadequate. Cracking and erosion of the surface were observed. In 1998, the EPA documented uncovered waste, cracks, gullies and gas vents on the landfill surface as well as very high concentrations of hydrogen sulfide gas in some soil gas samples. Subsequent sampling of hydrogen sulfide by Clark County in ambient air also detected unacceptably high values.

On September 11, 1998, a series of storms resulted in significant damage to the landfill and the discharge of waste from the landfill. Subsequent inspections by the EPA, the Nevada Department of Environmental Protection, and the Desert Research Institute documented damage to the landfill cover and storm water control system. The Nevada Department of Environmental Protection ordered Republic to assess and improve the
storm water control system at the landfill and to improve the groundwater monitoring. Subsequent plans by the responsible parties were deemed inadequate.

In 1999, the EPA issued two Administrative Orders (referred to as the RCRA Order and the CWA Order) to require additional corrective actions with respect to the landfill cover. A Notice of Violation was issued by the EPA to Republic on January 18, 2002, regarding submissions of landfill cover assessment reports including data collection and modeling procedures. In 2004, a Scope of Work was proposed by the EPA as an amendment to the Administrative Orders. In the Scope of Work, design requirements are given for the construction of the landfill cover, gas monitoring, groundwater monitoring, storm water control, and long-term operation and maintenance.

There are a number of issues and differences between the cover design required by the Scope of Work (the EPA’s design given in EPA, 2004) and proposed cover put forth by Exponent on Republic’s behalf (Exponent, 2003a). The two principal issues that are the subject of this report concern the adequacy of the two designs to limit erosion and infiltration.

Alternative covers

Regulations relevant to the covers for solid waste landfills are contained in 40 CFR Part 258. These regulations require the cover to be “designed to minimize infiltration and erosion.” The EPA has issued design guidance for covers to meet these regulations, which if followed result in what are commonly referred to as prescriptive cover designs. Alternative designs are permitted if they can be shown to be comparable, with respect to infiltration and erosion, to the prescriptive design. Discussions of alternative design concepts can be found in the literature (e.g., ITRC, 2003).

The most prevalent type of alternative cover is one that is designed to temporarily store precipitation until it is eventually removed by the combined effects of evaporation and transpiration. These types of covers are most applicable in dry, warm climates such as the western US. The simplest design of this type is just a layer of soil that, due to its properties and thickness, has sufficient moisture storage capacity with respect to the climatic conditions at a particular site. A review of soil cover performance by Anderson (1997) led him to conclude: “Past failures of earthen barriers as final caps on landfills in arid or semi-arid regions likely result from insufficient depths of soil to store precipitation and support healthy stands of perennial plants.”

A large number of soil covers have been or are being tested as alternative landfill covers under a wide range of climatic conditions (e.g., see summaries in Roesler et al., 2002; Dwyer, 2003). In no case was a soil cover constructed with a thickness less than 3 ft. Erosion protection is usually afforded by a combination of surface vegetation and limited slopes angles (typically between 2 and 5%) essentially similar to that designed for conventional covers. Gravel mulches and gravel admixtures have been used in a few locations in relatively arid climates as a means of limiting erosion of the surface layer (e.g., Caldwell and Reith, 1993; Anderson and Stormont, 1998; Dwyer, 2003).
II. Observations of existing conditions and design assumptions and principles relevant to the design and performance of the Sunrise Mountain Landfill cover system

A. Observations of existing conditions

Some important observations regarding existing conditions relevant to cover performance at the Sunrise Mountain Landfill are discussed in this section.

Waste is actively decomposing, resulting in settlement and gas production.

The waste in the landfill has a significant organic component, which is decomposing over time. The rate of decomposition varies with the waste content and moisture content. Without moisture, the waste would not decompose. Decomposition of waste is accompanied by volume reduction of the waste and the production of gas (methane). The amount of volume reduction can be over 50% of the initial volume of fresh refuse (e.g., Oweis and Khera, 1998; Jaros 1991).

The settlement rate at the Sunrise Mountain Landfill has been estimated from changes of the surveyed position of a gabion structure and the deformation of settlement monuments on site (Exponent, 2003b). The maximum settlement rate corresponds to an estimated annual strain rate of about 0.45 percent of the waste depth. This rate is on the low end of typical values for municipal waste landfills (Sharma and Lewis, 1994). It should be noted that the few discrete point measurements that have been made do not necessarily represent the maximum or even average settlements that may be ongoing at the site.

Settlement can also be observed in a more qualitative manner. Surface depressions have been observed at different times on the cover surface (Dwyer, 1997; Dwyer, 2000a), including some apparent slight depressions during our site visit (Figure 1). The presence of vegetation in locations of local water collection (i.e., low spots) provides a visual indicator of differential settlement.

Decomposition of the waste is also accompanied by gas production. Gas odor and surface staining from gas has been detected during various inspections (Dwyer, 1997; Dwyer, 2000a), including during our site visit. Sufficient gas was being produced that a gas extraction system was installed, and continues to extract gas from the decomposing waste.

Borings into the waste reveal that only a portion of the waste has degraded (Exponent, 2003b), and much of the waste is still relatively intact and subject to future decomposition and accompanying settlement. The waste has been variously described at different locations as moist, slightly moist and dry (Exponent, 2003b).
The cover is cracking in response to deformation of underlying waste.

The cover surface has experienced substantial cracking in response to differential settlement of the underlying decomposing waste (Dwyer, 1997; Dwyer, 2000a). Some cracking was visible during our site visit as well even though there was an indication of recent grading that may have covered some cracks (Figure 2). These cracks were long (at least several feet), generally linear and more than two inches deep. In contrast, desiccation cracking (which was also observed) will have a "checked" or "alligator type" pattern and be rather shallow.

The response of the cover to the settlement of the underlying waste will depend on characteristics of the soil used to construct the cover. In particular, the extent to which the cover material has a brittle response will dictate whether or not the soil cracks in response to deformation, and if so, the size, extent and frequency of the cracks. A brittle response can be related to the tensile strength of the soil. If the soil has some tensile strength, it will eventually crack in response to differential settlement. If the soil has no tensile strength, it will not crack but rather readily move or "flow" with the underlying deforming waste. Soils often derive tensile strength from cohesion due to clay minerals or
cementation of soluble salts. A soil without clay or without cementing minerals will not likely crack substantially in response to deformation.

Figure 2 - Surface-exposed crack on Top Deck (Area D). Photograph taken June 24, 2004.

The cracking on the cover surface of the landfill indicates that the soil has some tensile strength. Another indication of tensile strength is that vertical walls of trenches can stand unsupported. Exponent back-calculated a cohesive strength of 100 pounds per square foot for an on-site borrow source with a near vertical slope (Exponent, 2003b, appendix F).

Cementing from precipitates, perhaps including calcium carbonate and/or gypsum, is apparently the source of the soil's tensile strength and brittle behavior (e.g., Dwyer, 1997;
Dwyer, 2000a; Dwyer, 2000b; Exponent, 2003a). The soil has a light color, consistent with calcium carbonate and/or gypsum, and some locations of discrete light-colored precipitate were observed during the site visit. Some laboratory geotechnical testing forms described the material as "granular gypsum" (Exponent, 2003b, appendix C). A petrographic analysis of on-site borrow for aggregate reported the gypsum content to be 11% (Exponent, 2003b, appendix D). However, there are no measured amounts of calcium carbonate, gypsum or other salts for the existing cover soils. This information would provide additional insight for design.

Cementation will also affect the soil's hydraulic properties, generally by making the soil less conductive. Further, cementing produces an indurated (hard) soil that is generally difficult for plants to grow in.

There is limited vegetation on the cover surface.

During the site visit, there was a small amount of volunteer vegetation on the cover in a few locations, primarily Russian thistle. Refer to Figure 1. The estimated cover vegetation density was below 10%. Vegetation appeared to be located only in local depressions. Similar observations with respect to vegetation were made in previous site inspections (Dwyer, 1997; Dwyer, 2000a).

The water balance modeling that has been conducted is insufficient for design purposes.

Water balance modeling has become an accepted method to compare various cover designs. In particular, modeling is often used to compare the performance of an alternative cover design and the prescriptive design, and to evaluate cover performance sensitivities to design parameters (e.g., cover thickness). Given the complexity and spatial and temporal variability of the processes involved in the near-surface water balance, it is not surprising that modeling has had limited success in forward predictions of water balance values (e.g., Roesler et al., 2002; Dwyer, 2003).

Modeling of the Sunrise Mountain Landfill cover has been conducted with the UNSAT-H computer program (Benson, 2002). Some issues regarding climate input files for this modeling have been raised previously (EPA, 2002a). Additional fundamental concerns regarding the modeling are given below. We concur that the modeling that was conducted should not be used for design purposes, and that additional modeling would not likely be useful unless issues raised below were addressed.

The cover soil at the Sunrise Mountain Landfill is cemented and cracked. Such a soil is referred to as a "structured" soil. The cracks, under some conditions, will serve as a conduit to flow. Not explicitly including them in the modeling ignores the most significant pathway for infiltration into the waste. Including the cracks, however, is not an easy task as unsaturated flow under these conditions cannot readily be included in a model such as UNSAT-H (see CGER, 2001).
The soil properties used in the modeling were derived from re-packed samples (SCS, 2000). Re-packing the samples means that the cementing of the soil was destroyed, and the subsequent hydraulic properties would not reflect the properties of much of the in place soil. Eighteen of the nineteen hydraulic conductivities from re-packed samples used in the modeling were greater than $10^{-5}$ cm/sec (Benson, 2002). However, a single hydraulic conductivity test on a cemented chunk of soil from the site was found to be less than $10^{-5}$ cm/sec (apparently the measurement resolution) (Dwyer, 1997). This suggests the significance of the cementing to the hydraulic conductivity of the soil.

Another issue with the soil properties was that the soil was re-packed in the laboratory at a density that corresponded to 85% of the maximum dry density in a proctor test (SCS, 2000). However, subsequent in situ densities revealed significantly higher densities in the field than used in the laboratory (SCS, 2001). Because hydraulic properties are a strong function of density, the properties derived from the laboratory tests would be expected to be significantly different than the field based on density differences alone. It is also noted that the modeling did not consider the range of reported unsaturated hydrologic properties that defined the 95% confidence interval (SCS, 2000).

Another observation is that groundwater from off-site areas is likely to move to landfill areas and significantly change the water balance conditions. One-dimensional water balance modeling cannot represent this condition. This potential source of water suggests that some water can enter the waste regardless of the cover performance.

Slopes are highly variable within defined drainage areas

The Exponent Cover Plan (Exponent, 2003a) contains a drawing titled “Cover Plan Proposed Improvements” that is dated June 17, 2003. This drawing identifies nine areas of the Sunrise Mountain Landfill as:

Area A: Eastern Perimeter (with drainage area A1). This area includes Area A1: Optional Plan, Hardened Surface Treatment (with drainage areas A11, A12, A13 and A14)

Area B: Eastern Side of Lower Southern Flat Area (with drainage areas B1, B2, B3, B4, B5, B6, B7, B8 and B9)

Area C: Western Side of Lower Southern Flat Area (with drainage areas C1, C2, C3 and C4)

Area D: Upper Deck of the Top Deck Area (with drainage areas D1, D2, D3, D4 D5 and D6)

Area E: Side Slopes of the Top Deck Area (with drainage areas E1, E2, E3, E4, E5, E6, E7, E8, E9, E10, E11, E12, E13, E14, E15, E16 and E17)

Area F: Construction Debris Area (with drainage area F1)
Area H: Dead Animal Area (with the area within drainage area E5)

Area I: Asbestos Waste Area (with drainage area I1)

Area J: Northeast Canyon (with no additional drainage areas identified)

During the site visit on June 24, we were able to observe the Top Deck Area D and the top deck and side slopes of Areas A, B, C, E, F, H and I and the Northeast Canyon. The 5-foot contour interval topographic mapping on the “Cover Plan Proposed Improvements” map allows measurement of the slopes at the top deck and side slopes. We examined small areas in the drainage basins, and determined the following:

The slopes are highly variable within identified drainage areas. For example: Area B6 has slopes of 3.6% to 19%, Area C1 has slopes of 1.5% to 16%, Area D4 has slopes of 1.8% to 19%, Area E11 has slopes of 4.5% to 28%, Area E13 has slopes of 5% to 29%, Area I1 has slopes of 2.7% to 28%, and the Northeast Canyon has slopes of 4.5% to 21%. The variability makes identifying an appropriate runoff condition or surface treatment more difficult. Many of the identified drainage areas contain both top and side-slopes. Additionally, it is not possible to determine a specific line of transition between a top deck and a side slope in many of the areas because the contours show a gradual transition from a flatter top deck to a steeper side slope.

Slopes of top deck areas vary from about 1.5% to 6%. There are areas within drainage areas B2, C1, C2, D3, D4, D5 and D6 that are flatter than a 2% slope. There are slopes between 2% and 3% within drainage basins A1, B1, B2, B3, B4, C1, C2, D2, D3, D4, D5, D6 and I1 and at the Northeast Canyon. Top deck areas at drainage basins C1, C3 and C4 and at the Northern Canyon have slopes above 5%. There are many localized zones within top deck areas with slopes greater than 10%.

Slopes of side-slope areas vary from 13% to 31%. There are side slopes between 23% and 26% at drainage basins E5, E6, E8, E10, E11, E13, E14, E15, F1 and I1. There are side slopes between 26 and 29% at drainage basins E10, E11, E13, E14 and I1. Drainage basin E13 has an area with a side slope of 31%.

Potential sheet flow areas appear to be of limited extent.

Based on review of the 5-foot contour interval mapping on the “Cover Plan Proposed Improvements” map, the areas of sheet flow appear to be of limited extent. The contour mapping can be used to discern regions of sheet flow. A plane surface, where the contours are parallel and linear, will have sheet flow. There will also be sheet flow on convex surfaces. A convex surface exists when two points on the surface can be joined by a line that is interior to the surface. Additionally, the configuration of the contour...
mapping can be used to determine locations of concentrated flow. Some conditions that indicate flow concentration include the following:

- Contours that are linear but not parallel. Flow concentration occurs because sheet flow is diverted to one direction until it intersects another plane of flow. This is illustrated in Figure 3.

![Figure 3 - Flow concentration with sheet flow diverted to one direction](image)

- The intersection of two plane surfaces that have sheet flow when a concave intersection is formed. This is illustrated in Figure 4.

![Figure 4 - Flow concentration with sheet flow at a concave intersection](image)
- An irregular surface that is concave. This is illustrated in Figure 5.

![Figure 5 - Flow concentration with irregular concave surface](image)

- A bump or bubble in an otherwise smooth series of contours. When this shape creates a local concave surface it indicates the presence of a swale or channelized flow. With 5-foot contour intervals, even a small divergence from smooth contours can represent a significant flow conveyance. This condition is illustrated in Figure 6.

![Figure 6 - Flow concentration with contour bump or bubble (Example: 2-foot deep channelized flow at 5-foot contour interval)](image)
There are numerous locations within the top deck areas where the conditions described by Figures 3 through 6 are observed by examining the contour mapping. To illustrate this, drainage basin C1 was examined to identify possible flow concentration areas. Figure 7 shows the flow concentration locations that were identified from the existing 5-foot contour mapping. It is recognized that more detailed information is required to precisely identify the location and number of flow concentration areas. For example, 2-foot contour mapping can greatly improve identification of existing drainage features. Additionally, field surveys can be used to trace the location of drainage watercourses. As an example, Figure 8 shows concentrated flow on the Area D top deck observed during our site visit.

Figure 7 - Possible Flow Concentration Areas in Drainage Basin C1 (Based on examination of 5 ft contour mapping)
Figure 8 – Evidence of concentrated flow on the Area D top deck. Photograph taken June 24, 2004

In many areas there are numerous relatively shallow flow concentration areas. Re-grading would provide maintainable sheet flow areas and reduce the number of flow concentrations throughout the Sunrise Mountain Landfill. The design of top cover erosion protection will require precise determination of sheet flow lengths and slopes. Long-term maintenance will require monitoring of sheet flow areas and will likely require filling to restore uniform slopes when segments of the landfill experience settling. The location, slope and initial elevations of top deck sheet flow areas need to be established to prepare designs, construct cover protection and implement maintenance.
Throughout the existing landfill, surface water from the sheet flow areas and concentrated flows are conveyed directly to the steeper side slope areas. This condition occurs within drainage basins B5, B6, B7, B8, B9, C1, C4, D4, E11, E13, E15, F1, II, and the Northeast Canyon, and at the basin boundary between drainage basins D1 to E1, D2 to E1, D3 to E2, D4 to E2, D4 to E15, D5 to E7, D5 to E12, D5 to E16, D6 to E3, D6 to E4, and D6 to E6. Because of the high probability of “nick point” erosion at these areas, treatment of the top surface flows is needed. The use of top deck perimeter channelization will be needed in most of these locations unless the top surface slopes are changed so that sheet flow is directed away from the steep slopes. In some areas, special treatment in the top deck to side slope transition could prevent flow concentration and “nick point” erosion.

Some side-slopes appear to be draining onto top deck areas. This occurs within drainage basins D4, E5, E9, E16, II and the Northeast Canyon, and at the boundary from drainage basins E8 to E9, E10 to B4, E11 to E17, and E15 to I1. The flow velocity of sheet flow and the sediment transport capacity will change rapidly at the slope transition. At this location, sediment can accumulate at the lower surface and this can result in flow concentration as water moves around sediment deposits.

Off-site concentrated flows are conveyed directly to the landfill

Off-site concentrated flows are conveyed directly to the landfill at the Northeast Canyon drainage basin. South of coordinate N20500, there are seven flow concentration points at the west side of the Northeast Canyon drainage basin. These seven flow concentration points receive water from a combined off-site drainage area of approximately 110 acres. One of the seven west side flow concentrations is directed to a ponding area identified as the “NE Canyon Lagoon;” this feature is at the westerly edge of the Northeast Canyon drainage basin. Storm water retention at this location may reduce the peak surface flow but can add subsurface water to waste zones.

South of coordinate N20500, there are at least seven flow concentration points at the east side of the Northeast Canyon drainage basin. At the east side, off-site basin slopes are much steeper than at the west side, and the precise location of flow concentration is difficult to determine from the existing 5-foot contour interval topographic mapping. Some additional ground surveys will likely be needed to locate the flow concentration points at the east side of the Northeast Canyon. The total area that flows to the east side of the Northeast Canyon and south of coordinate N20500 is approximately 40 acres. The east side flows are very steep with the high potential for rock slides and debris flows. Concentrated flows are likely to cause scour holes at the confluence with the top deck of the landfill.

Off-site areas immediately east of the eastern perimeter, at drainage basin A, flow directly into the landfill. These flow conditions are similar to the conditions at the east side of the Northeast Canyon. There are at least 6 flow concentration points from the eastern off-site area, but field surveys are likely to identify additional concentration.
points. The off-site watershed at the largest flow concentration area is approximately 20 acres. There is a large sediment and loose rock deposit at the lower end of this 20 acre watershed. The remaining off-site drainage basins have a total area of approximately 30 acres. Landslide and debris flow onto the landfill area are a significant concern in this area. Scour-hole formation is probable at locations with flow concentration from off-site areas.

Offsite flows at the west side of the canyon in this area are significantly smaller (with slope lengths of 50 to 100 feet and slopes of approximately 30%). These off-site areas flow to the upper deck of the top deck area (drainage basins D1, D2, D3 and D4) and to drainage basin E15. Some local treatment from these off-site flows may be required.

B. Design assumptions and principles

In this section, some assumptions and principles relative to the cover design are presented.

The cover will continue to deform as waste decomposes.

Design of the cover system must recognize that the cover will deform as the waste continues to experience volume reduction from ongoing biological decomposition. It should be assumed there will be sufficient moisture for decomposition to continue for the foreseeable future. A well-designed and constructed landfill cover will not completely exclude moisture from the waste for a number of reasons including:

- Water may be entering from outside of landfill cover. It is possible that some of the water that falls on the adjacent steep mountainous slopes is directed into the waste from the sides of the partially filled canyon, that is, not through the top surface.
- A soil-based cover is not impermeable, and can periodically permit a small flux to enter the underlying waste.
- Moisture is contained with the waste itself.
- Condensation of water vapor.

It is inevitable and should be anticipated that the Sunrise Mountain Landfill will continue to settle due to ongoing waste decomposition. The amount of total settlement is not known, but an additional settlement of at least 10% would not be unreasonable given much of the waste has not yet fully decomposed, suggesting that settlements in excess of 10 feet over portions of the landfill are possible. Due to variability in waste thickness, waste composition, and waste moisture contents, settlements will undoubtedly be non-uniform, that is, there will be differential settlements.

Continued settlement will (1) crack cover soils that are brittle or cohesive, and (2) impact control of surface water. Surface-exposed cracking will allow rapid infiltration, as discussed below. Control of surface water is predicated on maintaining a designed
surface grade, which will change in response to underlying settlement. This suggests that any surface water control plan must recognize and account for this process.

**Surface-exposed cracks allow rapid infiltration.**

Because cracking is usually associated with differential settlement, it should be expected that there will be surface water directed over the cracked soil. If the crack is exposed on the surface, then a vertical crack represents a potential preferential flow path for water to move downward and potentially infiltrate into the waste. Directed run-off into a crack exposed on the surface must be avoided through placement of non-cracking surface soils and/or committed maintenance.

Surface-exposed cracks may fill from sediment transported in surface water or wind-blown material, but this does not represent a restoration or healing of the crack. The infill material will likely be different from the adjacent cover soils, and the filled cracks may continue to serve as a preferential flow path.

If a crack is not exposed on the surface, such as the case where there is a layer of intact soil on the surface that prevents direct run-on into the crack, then the crack may not transmit significant water downward. The role of macropores such as cracks and root holes on sub-surface flow is the subject of current research and debate (CGER, 2001). The simplest conceptual model is that as long as the crack is not saturated, it will transmit little if any water. If it is saturated, then it will transmit a relatively large amount of water quickly. There are some studies that indicate that macropores do not substantially affect water movement until the soil is relatively wet (Hawke and McConchie, 2003). On the other hand, there are other studies that indicate that some macropores are apparently quite transmissive well below saturation possibly due to film flow or droplet movement (CGER, 2001).

Regardless of the exact conditions under which they become active, there is sufficient evidence to suggest that macropores such as cracks will have a substantial role in any infiltration that may reach the waste.

**Vegetation should not be relied on to manage the cover water balance.**

Vegetation can be an important component in the water balance of a landfill cover, even in relatively dry conditions. However, the conditions at the Sunrise Mountain Landfill are sufficiently harsh with respect to the establishment of vegetation that any design should not rely upon the presence of vegetation and should not take any credit for impacts vegetation may have on the water balance or erosion stability.

Conditions working against the establishment of a reliable stand of vegetation include: the hard (indurated) nature of much of the cover soil, the high density of much of the cover soil (SCS, 2001), the apparently high salt content of the soil, as well as the very dry conditions.
Volunteer vegetation should not be discouraged as it will aid in the removal of water that infiltrates into the cover soil, and may help identify local depressions that form.

Use of gravel admixtures to protect cover surface.

An effective erosion barrier for conditions associated with semi-arid climates can be designed by combining gravel with native soils into a gravel admixture layer (Anderson, 1999). As finer portions of the soil are removed by erosive forces, the larger particles remain behind and form an "armored" layer that inhibits the formation of deep rills and gullies. The area of armored layer formation is restricted to zones where erosive flows are concentrated. This process is observed in nature in the formation of armored layers in sand and gravel bed arroyos. In contrast to some other treatments, an admixture will have little impact on vegetation or the soil-water balance.

A procedure for the design of gravel admixture that has been applied to landfills in semi-arid conditions (Anderson, 1999) is explicitly only for sheet flow conditions, that is, flow that has not been concentrated. For locations where flow is concentrated, additional treatment may be required.

A gravel admixture will produce sediment as part of its normal function. The amount of sediment will generally increase with the steepness of the slope and the slope length. The impact of this sediment on downstream conveyances should be considered in their design.

The gravel included in a gravel admixture must be environmentally stable (that is, stable under freeze-thaw, wet-dry cycles) in a manner similar to that used to evaluate rip-rap. Specifications for the gravel should include reference to ASTM standards related to stability (e.g., ASTM D 5312 and D 5313).

Because of the brittle nature of the on-site soils, if they are used in a gravel admixture it is very likely to crack in response to differential settlement.

Use of gravel veneer or mulch to protect cover surface.

In dry climates a surface gravel veneer or mulch layer can be utilized to provide erosion protection. A gravel veneer is constructed by placing a one or two-inch thick layer on the soil surface. The gravel must be of sufficient size that it will not be substantially displaced during a major storm event. A gravel layer will reduce surface erosion due to runoff and wind erosion, hold seed in place until it can germinate, and moderate the temperature of the underlying soil. Experimental studies have shown that gravel mulch can significantly reduce sediment yield from a cover (Finley et al., 1985; Wischmeier and Smith, 1978). At the same time, a gravel veneer will significantly reduce evaporation and increase the moisture of the soil immediately beneath the layer. This is a particular concern for a site where only evaporation (not transpiration) is relied upon to remove moisture. For steep slopes, where a gravel veneer is placed on a rather low hydraulic
conductivity layer, there is significant lateral drainage within the gravel and the problems with increased moisture are significantly reduced.

If gravel veneer is placed on top of a cracked soil, it may permit a fairly direct hydraulic connection to surface cracks. A gravel veneer may make it more difficult to detect the formation of surface cracks and make required maintenance less likely to occur. When grade adjustment maintenance is required, the gravel veneer will need to be removed. Following the re-grading, the gravel veneer must then be replaced.

**Drainage design criteria.**

A 200-year 6-hour precipitation event has been recommended for use for hydrologic analysis and hydraulic design of major drainage structures including the detention basin (an earth fill dam) and channels. It is reasonable that all drainage structures be designed with that criterion. The USDA NRCS (formerly SCS) curve number (CN) procedure has been proposed for computation of runoff from the precipitation.

A 100-year event and the NRCS CN procedure have commonly been used for design of flood protection structures. However, the historic development of the CN procedure has been based on 24-hour precipitation events. The computation of peak flow from a watershed is greatly dependent on the assumed distribution of precipitation over time, and the amount of rainfall in the time period that precedes the most critical runoff period. Therefore, the peak flow associated with a 24-hour distribution will normally be greater than the peak flow for a 6-hour distribution. Additionally, the volume of runoff will be greater for a 24-hour distribution. If a 6-hour precipitation distribution is used with the NRCS CN procedure, a somewhat greater rainfall than a 100-year event is appropriate for use and a 200-year event represents a reasonable adjustment to allow use of the CN procedure with a 6-hour distribution.

The design of erosion protection and capacity of conveyances that carry sediment should be based on consideration of single severe events, and not just on average annual sediment yields or sediment yields from commonly occurring rainfall. For a site evaluation in New Mexico, a 10-year storm produced erosion equal to three times the average annual erosion, and a 100-year storm produced five times the average annual erosion (Anderson and Stormont, 1997). Similar or more severe conditions are likely at Sunrise Mountain Landfill. In arid climates, a landfill may not experience erosion or sediment transport problems for many years, but be suddenly impacted by a single rainfall event.

An additional adjustment should be applied to the event duration when evaluating the capacity of ponds and detention dams. The critical precipitation event should be based on the precipitation during the time period that is most critical for the function of the structure. Pond and dam structures that have a very small discharge so that water storage occurs for a very long time need to be evaluated for longer events. It may be that a 24-hour, 48-hour or longer event will need to be evaluated to design a structure that retains or detains water.
The Nevada State Engineer criteria (Nevada, 2004) should be used for detention basins or dams that have embankments higher than 20 feet (the elevation difference between the crest and the toe of the structure) or impound more than 20 acre-feet of water. Nevada State Engineer approval of design may be required. Based on discussions during the meeting on June 24, 2004, there is apparently some possibility of urbanized development immediately south (downstream) of the landfill. Urbanized development in that area would require that detention basins or dams have a high downstream hazard classification because of the need to protect downstream property from failure of the structure during very severe events.

Criteria for all drainage structures must provide for the capacity of structures to convey runoff from the design precipitation including freeboard, and sediment transport. Sediment transport particularly includes a substantial portion of bed load material because of the fairly coarse soils and steep slopes at natural and constructed watercourses. Sediment transport can result in sediment bulking, channel bed degradation, lateral channel migration, head cutting and scour hole formation. When the sediment transport capacity of a water conveyance is less than the sediment transport from contributing areas, deposition or aggradation will occur. Aggradation can cause loss of channel capacity, flow blockages and changes in the channel roughness. The potential for sediment to alter flow conditions is particularly critical when large amounts of gravel and rock are available for transport during a precipitation event. The off-site and on-site conditions at the Sunrise Mountain Landfill have a high potential for gravel and rock transport.

Design of storm water conveyance structures must include entrance conditions, conveyance capacity and exit conditions. The design of entrance structures must direct flow to the conveyance without overtopping and consider the reduced flow velocities that are likely to occur. In many cases, entrance structures will need to be designed for weir flow conditions. Sediment deposition at entrance structures is a special concern because low entrance velocities can cause deposition of most of the sediment bed load. Entrance structures can be designed that maintain sediment transport capacity but the common application of simple headwalls does not normally meet this requirement.

The design of conveyance structures (channels, swales, pipes, etc) must consider the quantity, velocity and depth of flow. The slopes of existing flow paths indicate that supercritical flows are likely to occur at many locations. Supercritical flows are of special concern because of the potential for formation of standing waves and creation of hydraulic jump conditions. Standing waves can occur at curved alignments and at alignment bends. Hydraulic jumps can occur when there is change in the channel slope or shape. They can also occur when there is a sudden change in the vertical grade. The US Army Corps of Engineers Hydraulic Design of Flood Control Channels (EM 1110-2-1601, 1991) provides some guidance for considering supercritical flows.

The design of exit structures must consider the potential for scour-hole formation and the safe conveyance of flows to downstream conveyances. When supercritical flow
conditions exist, hydraulic jumps will commonly occur at outlet structures. Local scour will be a major design factor when this occurs. Another important consideration is the vertical stability of the downstream conveyance from sediment transport. If the downstream conveyance has a higher sediment transport capacity than the quantity of sediment that can be delivered, the downstream conveyance may be subject to degradation and increase the size of scour-hole formation. For example, if a conveyance has an entrance condition that causes most sediment to be deposited upstream of the structure, a downstream earth channel is likely to experience degradation equal to the downstream sediment transport capacity. In some cases the formation of downstream scour-holes can be incorporated into designs by implementing scour-hole armoring. The USDA NRCS has used this approach for protection of outlet structures at dams.

The consideration of appropriate freeboard in the design of storm water conveyance structures is almost as important as the basic hydraulic design. The Clark County Regional Flood Control District, Hydrologic Criteria and Drainage Design Manual (CCRFCD, HCDDM) Section 706.1.3 and 706.2.4 (1999) provides appropriate freeboard criteria. Freeboard is in addition to any standing waves, waves associated with hydraulic jump conditions and sediment bulking. Where side channel velocities can be accommodated by the erosion protection of the landfill cover that is adjacent to the conveyance, it may be possible to use the landfill surface to achieve the height for freeboard requirements.

All new and existing structures should be evaluated with respect to the design criteria outlined in this section. This includes the existing drainage swales and half-pipe conveyances that are currently visible at the Sunrise Mountain Landfill. The half-pipe metal channels that have been used have apparently experienced failure at the entrance structures, but there is additional concern about the available capacity of the conveyance, the irregular flow at joint sections, the potential for water loss at open joints and the scour conditions at the half-pipe outlets. All flow concentration locations should be considered conveyance structures and need to be appropriately designed. This includes concentration of flow from off-site areas and irregular top deck areas that create channels and swales. Every structure, new and existing, should be explicitly and uniquely identified so it can be designed, inspected, and maintained.

Construction QA/QC is critical to success.

There should be a strong emphasis on ensuring that the cover is constructed as designed. This will require two distinct efforts. First, a quality control (QC) program should be implemented by those responsible for construction. A QC program refers to measures taken by the contractor to determine compliance with plans and specifications through inspections, tests, and a system of inspections.

The second element to ensuring proper construction is a quality assurance (QA) program. A QA program refers to measures taken by the regulatory agency to confirm that the facility was constructed as designed. This will include audits, verifications, inspections, tests, and evaluations of the materials used in construction.
QC and QA programs are state of the practice for landfill cover construction (e.g., Daniel and Koerner, 1995) and should be included as part of any cover implementation. Particular areas of importance with respect to QA/QC include:

- Soil properties.
- Thickness of layers.
- Grades and slopes.
- Geomembrane construction.
- Storm water conveyance structures.

Monitoring and maintenance ultimately will be required, and will be a critical factor in the long-term success of the cover.

Regardless of particular design details, the cover will require monitoring and maintenance. An independent entity should periodically inspect and document the condition of the cover system. Maintenance action levels should be pre-determined and included in a maintenance plan.

Monitoring activities should include confirmation of grades by field surveys, identification of rill and gully formation, water balance measurements, monitoring sediment and debris accumulation in drainage control structures and detention basins, embankment and detention basin inspection, pipe inspection, characterizing vegetation, and collecting meteorological data.

Expected maintenance activities include:
- Sediment removal in drainage control structures and detention basins.
- Gully repair.
- Grade restoration of sheet flow areas.
- Grade restoration of drainage control structures.
- Crack identification and repair.
- Repair of embankment, emergency spillway, pipe, inlet, and riprap at detention basin.
- Cleaning of the pipe downstream of the detention basin
- Repair of riprap and gabions at conveyance structures

A reasonable approach for filling local depressions may be to use a local soil to bring the surface to grade as long as the topmost layer includes erosion protection consistent with the remainder of the cover.

It will be important to identify surface-exposed cracks and repair them as soon as possible. Any visible cracking on the surface should be repaired, even if the crack extends only a short distance into the cover. Because cracking is likely to occur where there is ongoing differential settlement, locations of repaired cracks should be documented and monitored because it is likely additional cracking will occur.
III. Review of the proposed cover by Exponent

Some key elements of the cover plans proposed by Exponent (Exponent, 2003a; Exponent, 2003b; Exponent, 2003c; Exponent, 2003d; Shaw EMCON/OWT, 2003) are reviewed below.

Soil thickness of 2 feet.

The cover plan for the Sunrise Mountain Landfill (Exponent, 2003a) indicates that a minimum 2-foot cover thickness would be placed on top of the waste. This soil cover thickness is unacceptably thin for a number of reasons.

The proposed cover does not meet the prescriptive requirements detailed in EPA guidance and apparently required by the State of Nevada (consistent with requirements in 40 CFR 258). In order to be considered an acceptable alternative, it should be demonstrated to be equivalent to the prescriptive requirements. There has been no demonstration of the equivalency of the 2-foot soil cover to the prescriptive design. Modeling has been discounted as a means of design comparison for a number of reasons (EPA, 2002a; EPA, 2002b; Exponent, 2003a). Further fundamental issues with the water balance modeling were raised in this document. An alternative to modeling for demonstrating equivalency can be comparison of the performance of field test plots. There has been no monitored test plot or cell of the proposed cover to demonstrate its performance.

There is no known precedent for an effective unvegetated 2-foot thick soil cover on a landfill. All of the known test plots include at least 3 feet of soil above the waste layer. Nevada’s regulations for alternative covers at small risk sites require 3 feet of soil cover.

While the climate is very dry, it is still possible for water on occasion to infiltrate well below a 2-foot depth. For example, a tracer study near the Nevada Test Site indicates that infiltrating water periodically reaches a depth of 7 feet below the ground surface (Cochran et al., 2001). Thus, it cannot be argued that a 2-foot soil layer would be sufficient to eliminate flux into the waste based on the analogy with undisturbed soil under similar climatic conditions.

The site has some unique conditions that are generally unfavorable with respect to a thin soil layer preventing a substantial flux, notably the cracking of the surface soil as previously discussed. Further, this cover will only rely on evaporation, so any redundancy and benefit attributable to transpiration cannot be counted on.

The existing 2-foot cover does not include any surface erosion protection layer that will be necessary to satisfy the requirement to minimize erosion.
Cover soil material

The cover plan indicates that material to increase the thickness of the cover would be processed from on-site sources ... “in order to achieve properties similar to those of the existing cover soil.” (Exponent, 2003a, p. 27). What these properties are, and why they are desirable, is not presented. It is necessary to specify material properties in order to assess whether material can perform its intended function.

Protection from erosion on top deck locations

The cover plan does not provide for a surface layer that will explicitly resist erosion. Exponent concluded rill erosion was unlikely to occur on the top deck areas because it was too flat (Exponent, 2003a, p. 8). An examination of the top deck slopes from the available topographic map (Section II.A) indicates that there are slopes that are in excess of 5%. Previous studies, including geomorphological observations, have shown that rills and eventually gullies can form on slopes as flat as 2 or 3% (Anderson and Stormont, 1997). Thus, some erosion protection on top deck areas is indicated.

Based on an examination of the available topographic maps, not all flow on top deck locations will be sheet flow (see Section II.A). There are locations where flow will concentrate. These areas also should be addressed with respect to erosion mitigation measures and designed as conveyances.

Detention Basin (Ref: Shaw EMCON/OWT 2003) Drawing Nos. 10, 11, 13 and 14

The proposed detention basin (flood water detention dam) is a reasonable conceptual approach for dealing with water from the watershed above the landfill. The structure will collect water and sediment and allow the water to be released slowly through a principal spillway pipe. An engineered earth fill embankment and a concrete channel emergency spillway over the embankment is proposed. Based on the height of the proposed embankment at 40 feet (from crest at elevation 2340 and toe at elevation 2300) and the volume of the impoundment, this structure appears to meet the conditions that require approval by the Nevada State Engineer. The following summarizes our concerns and comments.

- We have concern about the incorporation of sediment accumulation into the design. Sediment accumulates over time due to a series of events. A single design precipitation event can also produce a substantial quantity of sediment. The detention basin should be designed to accommodate both of these conditions. Because a substantial portion of the sediment will be bed material, it is expected that sediment will accumulate near the principal spillway outlet. The design of the pipe inlet (Detail 9/10, Drawing No. 13) does not appear to be able to accommodate sediment accumulation because of the limited number of inlet orifices and the grating elevation. In addition, floatable debris from the natural watershed can impact the capacity of the pipe inlet.
should provide for expected sediment conditions. The USDA NRCS and others have produced designs that accommodate varying sediment levels while maintaining required design discharge. We have not seen an analysis to quantify the sediment accumulation to determine the sediment produced from a single event or from normal sediment accumulation over time.

- A geomembrane is proposed at the upstream face of the detention basin and a portion of the flood pool. This feature will require special design and construction because it is apparently intended to protect the detention dam from failure due to seepage through the earth embankment, the base of the dam and the abutments. The location of the geomembrane in the flood pool is apparently based on soil borings at a limited number of test holes. Soil or rock criteria to establish the basis for locating the geomembrane in the flood pool need to be established. Stability for one foot of fill on the geomembrane for 40% slope at the embankment and 25% slope at the upstream basin is questionable. It appears that additional fill depth and erosion resistant fill will be required to protect the geomembrane from local and off-site flows, including several areas with concentrated flows. Measures to protect the geomembrane from puncture due to rock in surrounding base or cover soils need to be identified.

- We have concern about seepage/piping and the structural stability of HDPE pipe principal spillway through the embankment of the dam. The HDPE pipe is relatively flexible and subject to deformation even when the pipe is not experiencing collapse. This condition can lead to creation of voids in the surrounding soil embankment because there may be less elasticity in the soil than in the pipe. The pipe will have over 40 feet of embankment fill and there is concern about overall strength and buckling stability under these conditions. Additionally, compaction of the embankment during construction of the dam is a concern. It is noted that no cutoff walls or other seepage barriers are proposed at the pipe. There is no apparent method to connect the HDPE pipe to the geomembrane to prevent seepage at this location. The pipe is on a proposed curved horizontal alignment which may make inspection of the pipe segment under the embankment more difficult. The pipe may need to be placed on a vertical camber to accommodate settlement. The pipe appears to be under positive pressure for much of the flow conditions of the dam; this condition can cause embankment failure at areas with minor pipe leaks.

- We are concerned about seepage through the soil and rock layer immediately below the geomembrane at the dam embankment. Horizontal hydraulic conductivity in the rock layers is a critical design factor and is not addressed. Also, any soil layer immediately below the geomembrane at the upstream side of the dam should be designed to prevent seepage at the rock-embankment interface or at the principal spillway pipe.

- We are concerned about the structural and hydraulic design of the emergency spillway. It is apparently designed for a 500-year precipitation event. However, if
the downstream land use is likely to be urbanized, this will require a high hazard classification for the dam. Thus, a larger design event would need to be considered. The outlet of the emergency spillway is apparently designed as a SAF or similar US Bureau of Reclamation hydraulic jump structure. The modeling assumptions for this type of structure assume that a minimum downstream tailwater exists in the area immediately below the structure. It is not apparent that the design currently provides this condition. The plans provide a detail to show how the base slab of the emergency spillway will be connected to the geomembrane but a similar detail is not indicated for the side walls of the spillway. Also the flows at the approach to the emergency spillway will have a relatively high velocity and no measures to protect the earth embankment and geomembrane are indicated. The top portion of the emergency spillway and the dam embankment will be subject to initial and long term settlement, and the bottom end of the spillway is at a location with little settlement. This condition can cause structural failure in the spillway base slab and side walls for a spillway placed on a dam embankment. It is possible to design a concrete emergency spillway that can accommodate this differential movement, but such details are not indicated on the plans. It may be appropriate to consider alternative placement of the emergency spillway in the undisturbed soil area that is west of the west abutment of the dam. This location could reduce the potential for settlement at the spillway, simplify the construction of the dam embankment and allow for consideration of alternative spillway linings.

- The grading plan for the dam shows relatively sharp transitions between the constructed embankment and the existing abutments. These locations will need treatment to prevent concentrated flow erosion.

- The Geotechnical Engineering and Engineering Geologic Report Proposed Stormwater Detention Basin Sunrise Mountain Landfill (Exponent, 2003b) calls for “Removal of sediment from the basin following each winter season.” Additional monitoring of sediment accumulation following significant rainfall events is appropriate. One problem with such sediment removal is the potential for damage to the geomembrane during the maintenance. Additionally, it may be difficult to determine if any sediment has accumulated in the basin. Sediment monitoring points should be established throughout the flood pool to aid in sediment removal. Levels of safe sediment accumulation can be established and the bottom elevation for excavation can be clearly identified. Sediment level indicator posts are commonly placed for this purpose. The quantity of any sediment removal should be measured and reported.

**HDPE pipeline downstream of the Detention Basin**

The HDPE pipe outlet downstream of the detention dam is a reasonable concept, although we have some concerns about the design details. HDPE pipe is subject to ultraviolet and thermal deterioration and the proposed construction will permanently expose the pipe to the atmosphere. The pipe should be certified by the pipe manufacturer
for permanent installation that is exposed to the atmosphere. The structural integrity of
the pipe should be monitored for long term deterioration. The Geotechnical Engineering
and Engineering Geologic Report Proposed Stormwater Detention Basin Sunrise
Mountain Landfill (Exponent, 2003b) calls for “Annual closed-circuit television
inspection of the outlet pipe through the dam.” All of the HDPE pipe downstream of the
dam should be monitored. There appears to be a need to construct manholes on the HDPE
pipeline to accommodate television inspection and to allow maintenance access should
the inspection find problems within the pipe. There is the potential for sediment
accumulation within the pipe during the period that flows are decreasing. Cementing of
accumulated sediment can greatly restrict flows. One manhole should be placed near the
downstream toe of the detention dam to allow inspection of the principal spillway pipe
below the dam embankment. If an alternative pipe material is to be used within the dam
embankment, a downstream manhole can provide a location for a material transition.

We have concerns about the adequacy of the 12-inch pipes for drainage under the HDPE
dam outlet pipe. The design of these crossing pipes should follow CCRFCD, HDDDM
Section 1000 for culvert design (including 18-inch minimum size). The design should
consider the concentration of flow at the entrance and outlet, and sediment transport and
accumulation to protect the dam outlet pipe and the landfill cover. These designs should
follow the criteria for all storm water structures.

We note that if the upper channel was a hard lined channel, it would not be necessary to
have such a long dam outlet pipeline. If an alternative for a hard lined channel is
considered, it appears that additional removal of waste below the channel would be
required.

An access road for maintenance and inspection of the pipeline needs to be provided along
the full length of the pipeline. There are several locations along the pipeline where
existing contours do not appear to allow vehicular access. At Northing 19800 and
Northing 19200, there are locations where an existing storm water channel is very close
to the pipeline alignment. It appears that protection of the pipeline will be required in
these locations.

Local Drainage Swales on Eastern Perimeter (Area A)

This is a reasonable design element, but there is a lack of design analysis and detail, so
we cannot address the adequacy of this concept. If an unlined depression on the landfill
cover with unlined collection embankments are proposed, concentrated flow can result in
erosion of the landfill top deck. All the flow from the eastern steep slopes off-site from
Area A needs to be intercepted by structures that convey flow to the drainage swales. The
swale (channel) entrances and conveyances need to be designed as drainage structures.

It is anticipated that off-site flows at Area A will carry a large percentage of sediment and
rock. Scour-hole formation at the eastern edge of Area A may result from this flow. It is
appropriate to provide scour protection for the landfill cover at this location. The
transition structures that convey flows from the off-site areas to the drainage swales
(channels) need to be designed to accommodate sediment including rock. If this area will be designed for sediment deposition, provisions to protect the landfill cover during maintenance need to be provided. It may possible to provide some hardened surface treatment near the eastern edge of Area A to provide scour-hole protection and a definitive surface for maintenance.

**Hardened Surface Treatment as an option on eastern perimeter, Area A**

This alternative is a reasonable design element to reduce scour potential and facilitate debris removal at Area A. “Soil-crete” hardening is proposed by the Exponent plan. Hardening may prevent scour-hole formation at flow concentration points. A much reduced zone of hardening may effectively prevent scour-hole formation, but additional flow treatment on Area A would be required. The construction of surface hardening will not prevent cracking because of landfill settlement, and special crack repair maintenance would need to be established for any areas with “soil-crete” or soil-cement hardening.

**Hybrid (Eastern) perimeter channel**

A channel at the eastern edge of Area A was considered by Exponent but not recommended principally because of potential for rock slides and debris flows. We concur that this channel may not be effective or maintainable. The probable conveyance of flows directly from steep slope off-site areas creates sediment transport problems within the proposed channel that may not be possible to resolve. We note that if the channel was moved away from the eastern edge of the landfill, concerns about debris accumulation could be mitigated. Treatment at the edge of the landfill would need to be similar to that recommended for the drainage swales at the eastern perimeter.

**Terrace drains**

Flow path lengths into the terrace drains are highly variable, and in excess of 200 feet in numerous locations. The terrace drain spacing shown on the available plans is inconsistent with the WEPP modeling results (SCS, 2003) which suggested much shorter spacing was required. These results indicate that sediment yields from the proposed terrace drains from the 25-yr 6-hr storm will not satisfy the 2 tons/acre/yr criterion based on the WEPP modeling.

The terrace drain design in the cover report is purported to account for long-term settlement of the landfill because it is comprised of “flexible” components (Exponent, 2003a,p. 26). These components may be less likely to crack than some more brittle materials but, given enough deformation, they will fail. Also, differential settlement may change the grade of the drain sufficiently to alter its performance. Because of the probable channel slopes, supercritical flow conditions may be common.

Each of the terrace drains needs to be designed for the design precipitation and the contributing watershed. Design of terrace drains must include freeboard. The upstream side of terrace drains can provide freeboard provided that the adjacent slopes have
protection for the expected velocity. At the downstream side of terrace drains, additional side-channel depth will be required to accommodate sedimentation and freeboard. The typical section of the terrace drains does not appear to provide structural stability to support the proposed channel treatment. An appropriate support embankment will need to be provided.

**Drainage of off-site runoff in Northeast Canyon, Area J**

The off-site flows from the east and the west at Area J were not addressed. There are significant concentrated flows onto Area J. The flow within the Northeast Canyon also needs to be addressed. It will be necessary to construct channels to convey the off-site and on-site concentrated flows.

**Drainage crossings of landfill gas headers and laterals**

Five locations where there is a conflict between the landfill gas collection system and surface drainage are identified on the Exponent plan. Based on the existing contour mapping, there are at least seven additional locations within the waste filled areas where concentrated flows appear to cross the gas collection system. Additionally, there are at least eight locations outside of the waste areas where concentrated flows appear to cross the gas collection system. There are also locations where the gas collection system intercepts sheet flow areas so that the gas pipeline will cause concentrated flows. All of the drainage crossings will need to have designed culvert crossing structures. In some locations it may be possible to direct sheet flow away from the gas collection pipeline so that flow concentration does not occur. In other locations, conveyance structures parallel with the gas collection pipeline may be required.

**Channel/Embankment protection for Construction Debris Area, Area F1**

There is an existing concrete structure that is intended to provide protection to Area F1 from off-site flows. The off-site area draining to this area contains approximately 30 acres. There is no existing design analysis to show that the existing structure can carry the expected flow including sediment transport. Appropriate freeboard is also required. It is noted that the Asbestos Waste Area (Area II) could be impacted if the structure at Area F1 fails.

**Need for design elements at slope transitions**

In many locations there is no perimeter swale proposed at the transition between waste area top deck areas and side slopes. Locations of particular concern include Areas B5, B6, B7, B8, B9, C1, C4, E5, E13, F1, II and the Northeast Canyon (Area J). There are additional areas where sheet flow areas seem to drain directly to steep slope areas with no perimeter protection structure. In many areas the transition from the top deck to the side slope is very gradual and the identification of appropriate treatment will require detailed evaluation and appropriate design for each slope segment. In some cases, re-grading could simplify design and construction. Perimeter swales need to be designed to carry the
expected concentrated flows including sediment. The downstream edges of the swales need to include additional height for freeboard.

Many identified drainage areas have a transition from a relatively steep to a shallow slope with no identified bottom slope treatment. Protection is needed at these locations to prevent downstream scour and convey concentrated flows.

The Exponent plans indicate that four water quality settling basins are to be constructed. The design parameters and configuration for these basins need to be specified. It is assumed that the basins will hold water and cause deposition of sediment and contaminants during the design flow event. It is noted that some waste areas do not flow to water quality stilling basins, and the basis for establishing protection for a few areas needs to be established. Since water quality settling basins will cause sediment deposition, relatively clean “hungry water” will be discharged from the basins. This can cause downstream channel degradation unless the discharge is carefully controlled. Degradation can also cause formation of scour holes immediately downstream of the basin. Even though the settling basins structures do not present the risk that comes from overtopping of a dam, an appropriate overflow spillway needs to be included in the design of these structures.

Channel No. 1 and 3

We believe that these two channels are needed structures. Design details and plans should include bottom grade, water surface profile, freeboard, and top of design embankment. Information on the Froude number and flow velocity should be provided so that the design of freeboard can be verified. It is noted that the design report evaluated rip-rap but not the gabions shown on the plans. Sediment transport is a design concern with these channels because of the flows that come from the waste areas and the steeper off-site flow areas. Sediment transport capacity of the channels should be able to accommodate the expected sediment inflow. The Exponent report contains summaries of HEC-HMS (hydrologic) and HEC-RAS (hydraulic) computer models of the detention dam and the major channel segments. The information to verify the specified input conditions and quantify the numerical values of the output is not provided. Therefore, we could not provide any detailed review of these items.

At two locations along channel 3 “rockfall protection fencing” is proposed. This appears to be a needed provision. The height of the fencing is not indicated on the plans. The material for the fencing net is not specified, but we believe that it should be a permanent material that will withstand the expected loading from the rock-fall areas. No analysis of the expected volume of rock-fall has been located. The strength of the fencing configuration needs to be evaluated to confirm that it can support the expected rock loading.

There are several natural drainages at the eastern side of channel No. 3 where flows directly enter the side of the channel. Additionally, there are other swales, channels and
pipes that enter the channel from the landfill. Appropriately designed side inlets need to be provided.

Provisions to facilitate maintenance of the channels need to be in incorporated into the design. A continuous vehicular access needs to be provided on one side of the channel. At some segments of the channel, side slopes adjacent to the channel are at 14% or more.

Channel #2

We believe that this is a necessary structure. The Exponent report references a design capacity of 150 cfs, but the details of the hydrologic analysis to determine this value are not provided. There is no sediment/debris analysis for design of this channel and the upstream inlet/debris control fence. We note that the proposed debris control fence includes “8 oz U.V. stabilized, non-woven filter fabric.” We question whether this material can be considered a permanent material for exterior exposure. The strength of the debris control fence to withstand the water flow and sediment/debris loading is uncertain. We note the existing debris deposit in the upstream natural channel. If the debris control fence plugs with sediment, it appears that some of the flow could bypass the channel entrance because the top of the fence appears to be 0 to 0.5 foot above the top of the control berm. The entrance control berm needs to provide confinement of the flow plus required freeboard. If sediment/debris is trapped to the top of the control fence, the material would be expected to slope upstream from the control elevation. If most of the sediment/debris is to be trapped by the control fence, the storage volume for this material should be accommodated by the area above the fence.

Pipeline and Channel Junction

We believe that this is a necessary structure and the concrete lining is appropriate. The design capacity is a concern with the supercritical flow conditions in the channels and the confluence of two channels and the HDPE pipeline at this location. There is the potential for hydraulic jump and standing wave conditions at this location, and a detailed hydraulic analysis of the junction is needed. The extent of downstream channel protection could be affected by this analysis.

Rockfall Channel (Existing)

We believe that this is an appropriate use of an existing feature. It appears that hydraulic jump conditions with scour-hole formation are likely near Northing 13000 (Station 19+00). Waste at drainage basin E5 appears to be very close to this location and the formation of a scour-hole could cause removal of waste during a single precipitation event. It is possible to protect the waste by construction of armored scour-hole protection at the landfill side of the channel. The USDA NRCS has used armored scour-hole designs for protection of embankments at the downstream end of spillways.

Additional shot-crete erosion protection is proposed at one side of the transition between channel 3 and the rockfall channel. The criteria for locating additional erosion protection
need to be established. The protection of the channel from undercutting scour should be one primary criterion.

**Lack of design analysis and details for flow concentrations and existing drainage facilities**

There is no design analysis to indicate that existing storm water facilities are adequate for the design precipitation event. Also, there are numerous flow concentration areas indicated on the topographic mapping with no design analysis or construction details indicated.

**Crack repair**

The cover plan suggests crack repair should be conducted on an annual basis. Until it is shown how frequently cracks will appear, a shorter inspection and repair cycle is appropriate, perhaps initially every two months.

**IV. Review of EPA design elements**

The design elements required by EPA are principally described in the Proposed Scope of Work (EPA, 2004).

**3-foot thick cover soil**

The EPA is proposing a minimum 3-foot cover, including a 6-inch gravel admixture layer for areas with a slope less than 6% and 18-inch gravel admixture for slopes steeper than 6% (EPA, 2004). The 3-foot thickness is consistent with the minimum thickness of other soil cover layers. Three feet is the minimum thickness recommended by the State of Nevada for alternative covers. Three feet is about the minimum thickness of test covers installed in the western US.

To completely prevent any flux, the cover would probably have to be over 6 feet thick and vegetated. A 3-foot thick soil cover will undoubtedly allow some small flux to enter the underlying waste. Thus, some small flux through the cover should be anticipated. The amount of flux anticipated for a cover is usually predicted by modeling; however, modeling is not being considered for design purposes for this site.

The 3-foot thickness is reasonable as a minimum prescriptive requirement provided that water balance monitoring is conducted at major fill areas. In this way, the performance of the cover can be monitored. If there are indications of unacceptable infiltration through the cover and into the waste, remedial action could be taken (e.g., increase soil thickness, include other design elements, etc.).
There are many ways to conduct water balance monitoring, each with its own strengths and limitations. Some common approaches include:

- **Pan lysimeter**: Pan lysimeters are what have been used in many test covers. The principal advantage of this type of monitoring is that it captures downward moving water over the lateral extent of the lysimeter. In this way, the lysimeter can capture the effect of macropores such as cracks. One limitation of a pan type lysimeter is that the amount of water it collects tends to underestimate the flux through the cover. Another significant consideration is how the lysimeter will function in the presence of significant differential settlements which could disrupt the controlled bottom grade and access pipe connections.

- **Water content monitoring**: Water content monitoring using in place probes is a proven method of monitoring conditions in the sub-surface. TDRs or similar probes can be installed at numerous depths and locations and be monitored remotely, providing a nearly continuous record of soil moisture. Changes in soil moisture, in turn, can be used to infer soil water flux. Such a system may not detect or account for the presence of cracks.

- **Suction monitoring**: The use of suction measuring probes (such as heat dissipation probes) is a relatively new means of monitoring fluxes in near-surface soils including landfill covers. The benefit of these measurements is that they provide the hydraulic gradient, so the direction of water movement is known. Coupled with known hydraulic conductivity functions for the cover soil, these data can be interpreted in terms of soil water flux. This system may not be effective in detecting flow through cracked soil.

- **Periodic sampling**: Another monitoring approach is to periodically (say quarterly) obtain soil samples from the different depths and locations on the cover. These samples can be tested for water content and suction. This information will indicate the soil moisture status of the soil as well as the hydraulic gradient, in other words, which direction water is moving. Advantages of this method include: no field instrumentation is required, and there is flexibility as to the location and number of samples that are tested. A disadvantage of this method is that it does not provide continuous measurements.

Another condition on the acceptability of the 3-foot cover is that the erosion resistant layer on the top deck areas is kept largely free of surface-exposed cracks. This requirement emphasizes the need for a prescriptive maintenance program.

**Soil properties for soil barrier layer**

Soil will need to be added to the top of the existing cover to bring it to the required 2.5-foot thickness (3 feet total minus the 6-inch erosion layer). The EPA refers to this material as the soil barrier layer, and specifies the allowable grain size distribution as 30
to 50% fines with no gravel-sized or larger particles. We understand that this in not a specification for materials that are already in place.

It would be reasonable to adjust this specification to allow some limited amount of larger sized particles, say about 10% maximum 1-inch size particles. With 30 to 50% fines the soil would normally be expected to have a substantial storage capacity. However, unless storage measurements are made on borrow soils that are in the condition expected on the cover (i.e., cemented), the benefit of this specification cannot be quantified with respect to storage capacity. In light of this, changing the specification to 20 to 50% fines may have no detrimental effect and may be more readily achieved with on-site soils.

The plasticity parameters of this material should be limited to reduce volume change and cracking potential. Limiting the liquid limit to less than 40 and the plasticity index to less than 10 are reasonable specifications in this regard.

The specifications provided in the Proposed Scope of Work will not preclude the use of on-site soils screened to meet the particle size distribution requirement. This is the likely intent of Republic as stated in the cover plan (Exponent, 2003a, p. 27). Grain size distributions obtained from the on-site soils suggest many of them can satisfy the specifications by removing the material coarser than sand (Exponent, 2003c; SCS, 2000). The use of on-site soils would likely mean the soils will cement and crack in response to differential settlement. It may be acceptable to use these soils if there is diligent monitoring and repair of any surface-exposed cracks in the erosion layer along with water balance monitoring.

**Slope 3% on Top Deck area (Area D)**

A 3% minimum slope on the Top Deck (identified as drainage area D on the cover plan drawings) is reasonable. A 3% minimum slope will aid in minimizing areas of local ponding and maintaining positive drainage. This minimum slope is reasonable for this area because it is the location of the deepest waste and is expected to experience the greatest settlements with time. Additionally, a 3% slope is consistent with the State of Nevada regulations for minimum slopes (refer to EPA, 2004). Any consideration of a shallower slope at this location should be based on analyses that include detailed evaluations of construction grade control, time dependent settlement, frequency of grading and crack repair, spacing and design of flow conveyances, diversion of flow from adjacent areas, and accommodation of the gas collection system.

An approach to minimizing fill volumes on the Top Deck to achieve the 3% grade is to interrupt long slope lengths with drainage conveyances (e.g., swales) to collect and divert surface water. For the remainder of the cover, a minimum slope of 2% is reasonable and is consistent with the recent draft EPA guidance for alternative covers. Much of the flatter slopes on the remainder of the cover are between 2% and 3%. It is noted that slopes at 2% may require more frequent maintenance at areas experiencing landfill settlement and sediment deposition.
Gravel admixture

A gravel admixture can be an effective surface treatment for mitigating erosion problems on areas that experience sheet flow. As indicated earlier, there are many apparent or potential areas of flow concentrations where a gravel admixture may not be appropriate, and additional or alternative treatments are required.

The calculations (attachment 3 to EPA, 2003) for design of a gravel admixture layer are not consistent with the design methodology (Anderson, 1999). The sizing of the gravel in the admixture and the admixture depth is substantially dependent on the estimation of an effective average annual discharge or dominant discharge. The dominant discharge is a geomorphic concept that describes the flow that creates the geometry of channel formation. While the use of 10 to 20% of the 100-year runoff can represent a reasonable value to estimate the dominant discharge, the use of 10 to 20% of the peak 25-year flow rate does not represent an appropriate adjustment. The computation of dominant discharge should consider all the sediment/erosion producing events from the 50% probability event to the 1% probability event (i.e.: 50%, 20%, 10%, 4%, 2% 1% probability events), and then compute the statistically weighted average annual sediment yield. The effective average annual discharge is the discharge that produces a similar sediment yield. A reasonable simplification of this procedure uses the peak flow from a series of events and statistical weighting to compute the dominant discharge.

In previous applications a dominant discharge at 20% of the 100-year discharge has been used to size the gravel admixtures. This value represents an approximation of results from a statistically weighted analysis of a range of events. While the 100-year discharge is used as the basis for computing a percentage that represents the dominant discharge, this does not mean that designs using the dominant discharge can be called a 100-year design. Based on the idealized hydrologic conditions at the Sunrise Mountain Landfill top cover areas (0.03 ft/ft slope, CN=88), a dominant discharge at 20% of the 100-year flow would represent flow between a 2-year and 5-year event. If a gravel admixture is designed using 10% of the 100-year peak flow this would be representative of an event somewhat smaller than a 2-year event. It may be an appropriate accommodation to use a 2-year event as the basis for design of the gravel admixture, but it is not apparent that using a much smaller runoff (such as 10% of the 25-year design discharge) provides appropriate erosion protection.

There are some additional issues related to the spreadsheet used to compute the gravel admixture that need to be corrected, in addition to the use of a 100-year runoff rate in column B. Column B is labeled “Peak flow (Op in mm/hr)” but appears to be a peak runoff rate, and the peak flow can be computed by multiplying the peak runoff rate by the area that contributes to runoff. We cannot reconcile the values that are computed in column C with the tabulated runoff rates. The values in column F will be 20% or 10% of column C, if column C is correctly computed. The specific weight of the particles (column J) should be equal to the specific weight of the water times the specific gravity of the particles; the dry bulk density of the admixture was apparently used. The required
thickness in column T should be the armor layer thickness (column R) plus the scour depth (column S).

The application of the gravel admixture design procedure to steep slopes (such as those above 10%) may be inappropriate. We are aware of no applications that used the procedure for steep side slopes such as those at the Sunrise Landfill. It has been recently noted that the computed flow depth is much less than the computed particle size which is inconsistent with the application of Shields equation in the procedure. This would tend to over-estimate the movement of particles. Conversely, the equations do not consider the mechanics of the steep slopes so that the direction of potential particle movement is not in the direction of gravity. This would cause under-estimation of the movement of particles. These two conditions are not necessarily compensating, and the design procedure would require significant modifications to be applied to steep slopes.

One method for sizing erosion protection at steep slope areas is found in “Stability of stones on downstream face” (Chapter 3, Stephenson 1979). Another method is described by Abt and Johnson (1991). If a gravel veneer or riprap protection is used at the steep side slope areas, the soil beneath the rock layer should have a low hydraulic conductivity.

The Proposed Scope of Work contains values of 25-year event peak runoff (reported as peak flow) that come from output of WEPP modeling. Additional values for 200-year and 100-year events are contained on the spreadsheet (page 7) that is included in the Proposed Scope of Work. We have not located the WEPP model input and output files that generated these values so we could not directly review the details of that analysis. However, we did perform an independent evaluation of peak runoff rates for the landfill cover and side slope areas and find a substantial variance with the values that are in the Proposed Scope of Work.

We evaluated the top cover conditions assuming 600-foot flow control spacing and 3% slopes for the top deck areas, and 100-foot flow control spacing and 20% slopes for the side slope areas. We computed a time of concentration of 0.097 hours (approximately 6 minutes) for the 600-foot top deck area and 0.010 hours (approximately 1 minute) for the side slope areas using procedures from NRCS Technical Release No. 55 (Natural Resources Conservation Service, 1985). We used 100- and 25-year precipitation events as reported by EMCON (2000), with application of the adjustment ratios in table 501 of the Clark County Regional Flood Control District, Hydrologic Criteria and Drainage Manual (1999). We used the 200-year precipitation event as reported in Table 1 by Exponent (2003d). We used a NRCS curve number (CN) of 88 as reported in Table 2 by Exponent (2003d). A 6-hour rainfall distribution was used with the peak rainfall intensity assumed to occur at the middle of the 6-hour time and rainfall quantity. With this information it is possible to compute the peak rainfall intensity and the peak runoff intensity. A peak flow rate can be computed by multiplying the runoff intensity by the area of runoff. The peak 10-minute rainfall was associated with the peak runoff from the 600-foot top deck area, and the peak 5-minute rainfall was associated with the peak runoff from the 100-foot side slope areas. While it is possible to use a computer program to compute the peak runoff, it is not difficult to use manual computation to determine the
peak runoff rates from the peak rainfall, the rainfall distribution and the CN. The results of a manual computation by Anderson-Hydro are summarized in Table A.

Table A – Rainfall and Runoff Rates for Top Deck and Side Slope Areas

<table>
<thead>
<tr>
<th>Precipitation Event</th>
<th>25-year</th>
<th>100-year</th>
<th>200-year</th>
</tr>
</thead>
<tbody>
<tr>
<td>6-hour precipitation (inches)</td>
<td>2.53</td>
<td>3.62</td>
<td>4.20</td>
</tr>
<tr>
<td>Peak 1-hour precipitation (inches)</td>
<td>2.06</td>
<td>2.79</td>
<td>3.42</td>
</tr>
<tr>
<td>Peak 10-minute precipitation (inches)</td>
<td>0.93</td>
<td>1.26</td>
<td>1.54</td>
</tr>
<tr>
<td>Peak 5-minute precipitation (inches)</td>
<td>0.60</td>
<td>0.81</td>
<td>0.99</td>
</tr>
<tr>
<td>Peak 10-minute rainfall rate (mm/hour)</td>
<td>142</td>
<td>192</td>
<td>235</td>
</tr>
<tr>
<td>Peak 5-minute rainfall rate (mm/hr)</td>
<td>183</td>
<td>247</td>
<td>302</td>
</tr>
<tr>
<td>Peak top deck (10-minute) runoff rate (mm/hr)</td>
<td>92</td>
<td>147</td>
<td>189</td>
</tr>
<tr>
<td>Peak side slope (5-minute) runoff rate (mm/hr)</td>
<td>121</td>
<td>191</td>
<td>245</td>
</tr>
<tr>
<td>Peak top deck loss/infiltration (mm/hour)</td>
<td>50</td>
<td>45</td>
<td>46</td>
</tr>
<tr>
<td>Peak side slope loss/infiltration (mm/hr)</td>
<td>62</td>
<td>56</td>
<td>57</td>
</tr>
</tbody>
</table>

The peak runoff rates in Table A can be compared with the runoff rates tabulated on the spreadsheet in the Proposed Scope of Work; the values in Table A are substantially higher than the Proposed Scope of Work. A notable example is the runoff rate for the top deck 25-year event where Table A has 92 mm/hour and the Proposed Scope of Work has 19 mm/hour. One method to determine if a reasonable solution exists is to examine the computed loss or infiltration rates in Table A. At the middle of these rainfall events, when the peak rainfall is assumed to occur, the loss/infiltration rates would be expected to approach the saturated hydraulic conductivity of the soil. Existing cover soil laboratory hydraulic conductivity testing using remolded samples is summarized by Benson (2002). These values have an algebraic mean of 24 mm/hour and a geometric mean of 3 mm/hour. These values indicate that the CN, assumed to be 88, could be even higher for these soils. A higher CN would have somewhat higher runoff rates than are shown in Table A. Another method of comparison examines the ratio of the peak runoff to the peak rainfall, to determine if the results are consistent with observed conditions. For the top deck area at a 25-year event, the values in Table A show that 65% of the peak period rainfall will become runoff with the remaining 35% infiltrating into the soil. With the Proposed Scope of Work value (at 19 mm/hour), 13% of the peak period rainfall is assumed to be runoff and the remaining 87% is assumed to be loss/infiltration. We suggest that the values in Table A are more appropriate for a CN of 88.

While a CN of 88 was used to compute the values in Table A, the infiltration computed appears to be much greater than is indicated by the very limited hydraulic conductivity undisturbed field sample testing. The CN used for final design should be based on the infiltration of the final landfill cover, including any gravel admixture and cover layer soil, and not on a published value for an undisturbed natural area (rangeland, desert shrub-poor condition, hydrologic soil group D). Steep side slopes will likely have a larger CN than top desk areas.
The slopes assumed for the side slope are not consistent with the existing side slopes as deduced from the available maps. For the side slope areas, the calculations assumed a maximum slope of 20% and a flow length of 100 feet. As noted earlier, substantial portions of the side slopes are at angles in excess of 20% (see Section II.A). The analysis for slopes less than 6% assumed a maximum 600-foot flow length. This is appropriate for the Top Deck (area D) where diversion berms are specified, but many locations, on areas C and B for example, may have flow lengths in excess of 600 feet. It is emphasized that these analyses are only applicable to areas that experience sheet flow and not for areas of flow concentration.

The gravel admixture will produce sediment as part of its normal function. The impact of this sediment must be considered on downstream drainage structures.

The gravel to be used for the admixture must be specified to ensure that it is environmentally stable. Tests similar to those used to assess the suitability of rip rap, gabions and other stone applications may include:

- Petrographic analysis similar to that reported for aggregate suitability in doc #51
- Bulk specific gravity (saturated surface dry), typical specification>2.48, tested using ASTM C 127
- Adsorption, typical specification <2%, tested using ASTM C 127
- Durability under wetting and drying conditions, ASTM D5313
- Durability under freezing and thawing conditions, ASTM D5312

Additional specifications must be provided for the gravel to ensure that once blended with the soil, the required fraction of particles above the critical size is consistent with the design. This will require limiting the amount of fines and sand in the gravel.

The EPA specifications are based on an admixture with 33% gravel. Other percentages of gravel can provide an acceptable design. A gravel percentage between 20% and 50% is a reasonable range.

The soil specified by the EPA (2004) to be used in the admixture is identified as “topsoil” and is to be classified by the USDA method as a loam. Topsoil may be a misleading name as it implies the soil will be used to sustain vegetation; typically, topsoil has properties conducive for vegetation such as a particular pH and organic content range. Without quantifying the required water holding capacity of a soil or other critical properties, it is not possible to exclude the acceptability of soils with other grain size distributions. The required grain size distribution called for in the Proposed Scope of Work probably contains too many fines rendering it susceptible to volume change and cracking. Our preference would be a material with a maximum of 45% fines, of which the clay content was a maximum of 20%. The justification for requiring a minimum plasticity index of 7 should be explained or removed, that is, set a minimum plasticity of zero. It may be desirable to limit the maximum plasticity to perhaps 5 or 10 instead of 30 to reduce the possibility of volume changes that can lead to cracks.
The specifications preclude any gravel size particles in the soil used for the admixture. The inclusion of some gravel would not necessarily be detrimental, and if the gravel was of the proper size, it could contribute to the gravel fraction required for the gravel admixture design.

If any hydraulic property tests are to be required (moisture characteristic curve, hydraulic conductivity, water holding capacity), they should be conducted on samples that retain the structure anticipated for the in place soil. Specifically, if the soil is expected to cement to some degree, then the soil in a cemented state should be tested. Cemented soil could be evaluated with field tests (e.g., infiltrometers, tension infiltrometers) or by laboratory tests of intact field samples or possibly reconstituted lab samples.

The best choice for a soil for the admixture is a non-cohesive, non-cementing soil if available. The use of such a soil would minimize cracking and maintenance of the topmost layer. Limiting cracking of this layer maximizes its ability to attenuate infiltration into the cover system. Limiting the plasticity index of the soil to a small value will reduce the potential for cracking due to cohesive behavior. It is more difficult to develop a specification to reduce cementing as the exact cementing agents and their contribution to cementing are not well known. It is likely that most soils in the vicinity of the landfill would cement to some degree; as little as a few percent calcium carbonate can cement soils, albeit weakly. Thus, a specification for a non-cementing soil may require a very low cementing agent content (perhaps as little as 1%) be present. As a practical matter, an experienced geologist or engineer could assess the cementing nature of potential borrow soils during their excavation.

If on-site soils are to be used, cementing and resulting cracking appears inevitable. The evaluation of borrow sources (Exponent, 2003a, appendix C; Exponent, 2003b; SCS, 2000) suggests extensive screening may be required to create a soil which satisfies the grain size distribution specification in the Scope of Work using on-site soils. An alternative with fewer fines may be more practical to create using on-site soils.

Regardless of the soil that is used in the gravel admixture, it is imperative that cracking of the erosion-resistant layer be identified and repaired as soon as possible.

**Use of a gravel veneer**

It is suggested that consideration be given to the use of a gravel veneer on the side slopes. A gravel veneer on these slopes will produce run-off, but little sediment. The existing cover on the side slopes should not be very coarse, high conductivity material (i.e., solely gravel and/or stone). The hydraulic conductivity of the existing side slopes needs to be verified if a gravel veneer is used.

**100-foot spacing of terrace drains**

A 100-foot spacing of terrace drains is reasonable, but this length is not necessarily the practical maximum spacing. Typical spacing of terrace drains of one per 50 vertical feet...
is common (Sharma and Lewis, 1994); on a 25% slope, this would correspond to a terrace drain spacing of about 200 feet.

600-foot maximum flow length on Top Deck

The 600-foot flow length for top deck sheet flow is a reasonable maximum, but shorter lengths may be required for analyses that consider varying slope conditions and erosion layer configuration.

Terrace drains

The conceptual terrace drain design suggested by the EPA (EPA, 2004 attachment 4) addresses some concerns about maintaining the thickness of the underlying cover soil, capacity of the drain, and stability and support of the sidewall of the drain.

Other Storm Water Drainage Structures

The structures identified by Exponent and Shaw EMCON/OWT as Channels 1, 2 and 3, rock outfall channel, pipeline and detention basin will be required. Design of other structures to convey concentrated flows (including perimeter drains, terrace drains and drainage swales) is required. See previous comments on the Exponent design in section III.

Maintenance and monitoring

Repair of surface-exposed cracks is important to the integrity and functioning of the cover, and has been identified as an expected maintenance activity (EPA, 2004; Exponent, 2003b). Criteria for the identification of cracks that require repair should be provided. A crack should not be required to be “wide open and deep” before it is repaired, as a very thin crack can transmit a significant amount of water and it is not obvious how deep a crack penetrates from its surface expression. A crack identification criterion could be as simple as any surface-exposed crack at least 1 mm wide, 1 m long requires repair. Obvious shallow alligator type cracking due to desiccation could be excluded.

Periodic monitoring of landfill settlement needs to be implemented. Elevation monitoring stations need to be established throughout the waste containing areas. Grades of top deck areas need to be verified. A historical record of settlement needs to be maintained and periodically reported.

Maintenance in response to settlement will be necessary for the proper functioning of the cover. Local depressions should be filled to restore the intended local grade. The material used to fill these depressions should be consistent with the intended cover profile so that the topmost layer is the erosion layer. If more fill depth than the specified erosion layer thickness is required, then soil consistent with the soil barrier layer specifications should
be placed beneath the erosion layer. It should be anticipated that enough settlement will eventually occur so that the surface water control plan may need to be modified.

Water balance monitoring of the cover should be implemented. Monitoring may reveal areas of the cover that may require additional cover material to reduce percolation into the underlying waste. Depending on how the monitoring is done, it may be possible to quantify effective fluxes through the cover and/or identify the relative contribution of percolation through cracks versus the intact soil.
REFERENCES (documents provided to Anderson-Hydro are identified by a document number in parentheses)


Clark County Regional Flood Control District, 1999, Hydrologic Criteria and Drainage Manual, Las Vegas, NV


Dwyer, S.F., 2000a, *Sunrise Mountain Landfill Cover Characterization, Field observations for April 17 to 21, 2000 and May 1 to 5, 2000*. (document 11)

Dwyer, S.F., 2000b, letter dated February 1. (document 13)


EPA, 2002b, email: “FYI Sunrise Mt. cover modeling,” December 12. (document 35)


SCS, 2000, *Draft Evaluation of Final Cover including laboratory analyses backup*, October and November. (document 14)


