



## Geosynthetics Cost/Benefit Analysis for the Development of a Landfill Expansion Module in Monterey, California

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### ABSTRACT

As Part of the design of an expansion cell at the Monterey Peninsula Landfill, several design innovations were developed using geosynthetics. A cost/benefit analysis was then conducted on the proposed innovations to the design to determine the potential cost savings associated with each modification. The modifications to the existing design of previous cells using geosynthetics that were conducted for the Module 4 expansion included the following: 1) Increasing the internal cut slopes of the excavation from 3 horizontal to 1 vertical (3:1) to a slope of 2:1 utilizing a geosynthetic clay liner in place of a compacted clay liner; 2) Increasing the final refuse slopes of the landfill from 4:1 to 3:1 by increasing stability with textured geomembrane, which expanded the waste capacity; and 3) Revising the base-grading plan for the module and leachate collection and removal system layout, which also resulted in increased airspace.

### 1. INTRODUCTION

The Monterey Peninsula Landfill (MPL) is a Class III sanitary landfill covering an area of approximately 127 hectares. The site is located in northern Monterey County of California, approximately 1.6 kilometers east of State Highway 1 and 3.5 kilometers north of the City of Marina. The MPL began accepting waste in 1966 and was constructed in a series of modules. Modules 1 and 2 consisted of unlined cells and Module 3 was lined with a compacted clay and high density polyethylene (HDPE) geomembrane composite liner. As the third phase of Module 3 reached its final fill capacity, the District solicited proposals from qualified engineering consulting firms to prepare the design, technical specifications, and construction drawings for the Module 4 expansion cell.

As part of the solicitation of proposals, the District identified the presence of a perched water zone within the proposed module excavation as being a critical factor in the design of the Module 4 expansion cell. In addition, the District requested that a cost/benefit analysis be conducted in regard to the design parameters developed during previous module designs. Following award of a contract, the selected consultant reviewed the existing design reports and studies from the construction of previous modules.

To supplement the existing information and to determine the geotechnical parameters and subsurface hydrogeology specific to the Module 4 area, a geotechnical investigation was conducted (Vector, 2001). Laboratory testing and engineering analyses were performed on samples obtained from boreholes within the Module 4 cell. From the drilling and analyses, recommended design and construction criteria were determined including the hydrogeology, estimation of capillary rise of the critical aquifer below the cell, the hydraulic conductivity of the *in situ* and remolded native clay, consolidation characteristics of the native soils, and the thickness of the underlying *in situ* clay. Based on these data, several design modifications were identified as having the potential to result in a cost savings to the District.

In order to determine the potential savings of the design modifications, a cost/benefit analysis was conducted and presented to the District. Following District concurrence, the design issues were submitted to the Regional Water Quality Control Board (Regulating Agency) for approval. The design modifications related to geosynthetic components of the liner system that resulted in significant savings to the District include the following:

- Increasing the internal cut slopes of the excavation from 3 horizontal to 1 vertical (3:1) to a slope of 2:1 utilizing a geosynthetic clay liner in place of a compacted clay liner.
- Increasing the final refuse slopes of the landfill from 4:1 to 3:1 using a textured HDPE geomembrane for increased stability, which expanded the waste capacity.
- Revising the base-grading plan for the module and leachate collection and removal system layout, which increased airspace while utilizing the benefits of the *in situ* clay layer below the cell.
- Installation of a perched groundwater removal system to allow for the construction of the module below the existing water level.

The following sections describe each of the above modifications and the associated cost savings.

## 2. INCREASING INTERNAL CUT SLOPES

The existing design for the external cut slopes of expansion modules at the MPL called for a cut slope of 3:1 with an overlying composite liner composed of HDPE geomembrane and compacted clay. The western slope of the proposed Module 4 expansion contains approximately 750 lineal feet of external cut. A cost/benefit analysis was conducted on this external cut slope to determine the feasibility of increasing the cut angle from 3:1 to 2:1. The costs listed within this cost/benefit analysis are an average of the actual bids received from contractors for the Module 4 construction.

### 2.1 Increase in Volume and Airspace

The increase in volume gained by cutting the west slope of Module 4 from 3:1 to 2:1 results in approximately 39,435 cubic meters of airspace. Assuming a waste disposal rate of \$19.62 per cubic meter (MRWMD, verbal communication), the increase in airspace will result in increased waste revenue of \$773,625. In addition to the increase in disposal volume, the sand that is excavated from the cut also has value to the District. The excavated sand has a commercial sales value of \$4.90 per cubic meter per conversations with District's staff. The revenue generated from the sale of the sand removed from the west slope amounts to \$193,406. Therefore, the total revenue generated from cutting the west slope to 2:1 versus 3:1 is \$967,031.

With the increased volume of airspace, there is corresponding increase in the cost of excavation. Using an excavation cost of \$2.05 per cubic meter (average of actual contractor bids), this amounts to \$80,973 in construction costs to increase the west slope cut to 2:1. Subtracting the cost of excavating the 2:1 slope from the increase in airspace and additional sand revenues gives a total cost benefit of \$886,058.

### 2.2 Slope Area and Liner System Modification

By changing the originally proposed external cut slope from 3:1 to 2:1, the surface area of the slope that needs to be lined is significantly reduced.

The 3:1 slope has a surface area of 11,889 square meters. Using the average construction cost for clay liner of \$7.01 per cubic meter (cost for placing and compacting clay on a slope) and assuming the as-built liner thickness averages 0.67 meters, the construction cost for a clay liner on a 3:1 slope would be \$55,872.

However, utilizing a 2:1 slope yields a surface area of 8,314 square meters. With a slope as steep as 2:1, clay liner cannot be placed and compacted in the standard way. For this reason, a geosynthetic clay liner (GCL) was proposed. Utilizing the average cost for installation of a GCL from the Module 4 contractor bids of \$5.70 per square meter results in a construction cost of \$47,414 for the west slope. Thus the decrease in slope area and liner system modification results in a construction cost savings of \$8,458.

The cost-benefit analysis regarding the west slope of Module 4 indicates that modifying the previous design would result in a significant cost savings. The previous design of a 3:1 external cut slope with a clay liner was compared to the proposed 2:1 cut slope with a GCL and 1.5-mm HDPE geomembrane. The total of increased revenue and cost savings associated with the modified west slope design was \$894,516.

## 3. INCREASING THE FINAL SLOPES

The final refuse slopes at the MPL were permitted at a slope of 4:1. By increasing the final slope of the refuse to 3:1, a significant amount of additional waste volume could be realized. This increased waste volume would also significantly extend the site life of the MPL. Based on an analysis of the entire MPL site development, an increase in the final refuse slopes to 3:1 will result in a gain in capacity of 5.9 million cubic meters. Over the life of the site, this will result in additional revenues of approximately \$115 million.

The Module 4 cell is bounded by current or future refuse cells that will provide long-term buttressing of the waste. However, interim waste fill slopes will be unbuttressed. By increasing the interim waste slopes to 3:1, additional waste could be placed in Module 4, extending the period required prior to the development of the next cell. The first geosynthetics lined cell within Module 3 of the MPL was constructed with smooth HDPE geomembrane. Stability analyses conducted by previous consultants on that module indicated that the interim and final refuse fill slopes should be constructed at a slope of 4:1 in order to maintain adequate factors of safety.

In order to gain the approval of the Regulating Agency for the increase in the native cut slopes and the overall refuse fill slopes, analyses were required demonstrating the stability of the slopes. The final build-out and interim conditions of the module were analyzed for stability using a textured HDPE geomembrane. Both failure of the waste mass and failure of

the cut slopes in native soils were analyzed. Failure surfaces were evaluated utilizing limit equilibrium methods to determine the factor of safety for stability. The factor of safety was defined as the ratio of total equilibrium shear stress to available shear strength. Module 4 was evaluated under both static and pseudo-static (earthquake) conditions. The stability analyses conducted at the MPL to demonstrate the stability of the steeper slopes is described in the following subsections.

### 3.1 Configuration

The most critical period for stability of the overall landfill in the Module 4 area exists before the waste mass will be buttressed by perimeter cut slopes or other waste modules. A typical section through the south face of Module 4, where critical conditions exist, was analyzed for overall failure of the waste mass. Similar conditions will also exist on the east side slope of Module 4. Only one typical cross section was analyzed due to the uniform grading of the bottom liner across Module 4.

The waste will be graded at a maximum slope of 3:1 with benches up to a maximum crest elevation of approximately 30 meters above mean sea level. On the north and west sides of Module 4, the waste will always be buttressed by previously placed waste and a cut slope in native material, respectively. To provide additional buttressing of the west slope, the initial waste lifts will be placed in direct contact with the west slope for a width of 9 meters along the entire length of the slope to a height of not less than 3 vertical meters. The condition that was analyzed for the overall landfill failure was considered an interim condition, although the slope will be exposed for an extended time period.

The cut in native soil on the west side will be excavated with 2:1 slopes. The initial waste loads will be placed in 3-meter lifts up against the side slope and extending across Module 4 with a minimum width of 9 meters. The most critical period for stability of the cut slope will be before any waste is placed up against the slope. As waste is placed against the west slope, the slope will be buttressed and stability will not be an issue.

The landfill bottom liner consists of a 1.5-mm HDPE (textured side face down) on a compacted clay liner, overlain by a drainage/operations layer immediately below the waste. On the western side slope, GCL liner will be used instead of a prepared clay liner, due to the difficulties of performing compaction on steep slope faces. Due to the fill sequencing of the landfill, with waste placed across the entire module in lifts, shear failures with sliding along the side slopes with GCL's was not a critical failure mode.

### 3.2 Method of Analysis

The stability of the Module 4 area was evaluated utilizing limit equilibrium slope stability methods. Module 4 was analyzed for stability using the computer program UTEXAS3 (Wright, 1990). For overall landfill stability, the computer program calculated a factor of safety for shear failure surfaces that ran along the bottom liner for some distance before going up through the waste. For cut slopes in native soil, the factor of safety for shear surfaces was evaluated for failures entirely within the sand, and for failures that included the waste mass placed at the toe of the cut slope. The critical shear surfaces were found interactively. Failure surfaces were evaluated using Spencer's method of slices.

Both static and pseudo-static analyses were performed. The pseudo-static analyses subject the two-dimensional sliding mass to a horizontal acceleration equal to an earthquake coefficient multiplied by the acceleration of gravity. The earthquake coefficient, or pseudo-static coefficient, is typically equal to a percentage of the maximum design acceleration in bedrock. A pseudo-static coefficient of 0.18 was used, which was one-half of the mean peak bedrock acceleration specified in the Module 3 Design Report (EMCON, 1992). The pseudo-static analyses provide a preliminary check on whether further analyses should be performed. If the factor of safety was less than one, a displacement analyses would be required.

### 3.3 Material Properties

The material properties used in the stability analyses are summarized in Table 1. Strength parameters are based on effective stress large-strain conditions, except in the case of the native clay layer that was encountered during the Module 4 geotechnical investigation (Vector 2001), where total stress conditions were evaluated for the case of the temporary 2:1 excavation on the west side of Module 4.

The shear strength of the waste was based on data published by Kavazanjian (1995), in which the shear strength of refuse was based on back-calculated shear strengths from case histories and the results from *in situ* testing of waste.

The shear strength between the geomembrane liner and the compacted clay material was based on large scale direct shear tests that were performed between the stockpiled clay and a textured HDPE geomembrane. A check point was performed using the proposed stockpiled clay and HDPE to verify the previous results.

The shear strength of the *in situ* native clay material was a conservative estimate of the clay under total stress conditions, based on the standard penetration numbers between 12 and 29 measured during the Module 4 field investigation (Vector 2001) and correlations in the literature with unconfined compression strength (Das 1990). The total stress conditions were applied to the 2:1 excavation on the west side of Module 4. For overall failure of the landfill, the internal shear strength of the clay would be greater than the shear strength with the geomembrane interface; therefore, effective stress conditions for overall failure were not critical in the evaluation.

The shear strength of the *in situ* native sand material was based on typical values for relatively clean sands with rounded particles. A small cohesion of 0.96 kilo pascals was included to account for the cohesive behavior of unsaturated sand.

Table 1. Summary of shear strength parameters used in the analyses.

Material	Total Moist Unit Weight (g/cc)	Friction Angle (degrees)	Cohesion (kpa)
<i>In Situ</i> Native Clay Soil	1.60	0	71.9
Refuse	1.04	33	4.8
HDPE (text.)/ Compacted Clay	N/A	5	14.4
<i>In Situ</i> Native Sand Materials	1.76	33	0.96

### 3.4 Results of Stability Analysis

The results of the stability analysis conducted on Module 4 are summarized in Table 2. The output from UTEXAS3 contains the cross section information, the material properties, and the locations of the critical shear surfaces with the lowest factors of safety.

Table 2. Results of Module 4 Stability Analyses.

Description	Static Factor of Safety	Pseudo-Static F.O.S.
Overall Landfill to 30 Meter Elev., Typical Cross Section	1.7	<1.0
2H:1V Module 4 Excavation	1.4	<1.0
2H:1V Module 4 Excavation, with 3 Meter Waste Buttress	1.6	1.2

For the case of the 2:1 excavation on the west side of Module 4, the factor of safety was increased from 1.4 to 1.6 against failure when a lift of waste was placed at the base of the slope. With the waste mass in place at the toe of the slope, the factor of safety for failure of the cut slope above the waste mass will gradually increase above 1.4. When the waste within Module 4 reaches an elevation of approximately 50 feet, the factor of safety for failure of the cut slope above the waste will be approximately 1.5. It should be noted that for unsaturated conditions within the sand, the actual shear strength is higher than was used in the analyses due to negative pore pressures, which would cause the actual factor of safety to be higher.

The pseudo-static factor of safety was less than 1.0 for the southern refuse slope in the eastern Module 4 area, and for the temporary excavation on the west side of Module 4. A factor of safety less than 1.0 indicates that some displacement in these areas may occur if the design earthquake event with an acceleration of 0.35g were to occur at the site. Displacement is discussed in the following section.

### 3.5 Dynamic Displacement Analysis

A displacement analysis was performed to estimate the amount of movement along the critical shear surfaces that may be expected during the design earthquake event. A simplified procedure for estimating earthquake-induced deformations was performed in accordance with procedures presented by Makdisi and Seed (1977). An earthquake with a magnitude

of 8.25 was used for the analysis, based on an earthquake originating on the San Andreas Fault, located approximately 24 kilometers to the northeast.

The first step was to determine the yield acceleration for the slopes with a pseudo-static factor of safety less than 1.0. The yield acceleration was defined as the horizontal acceleration that, when applied to the landfill section in the limit equilibrium analyses, resulted in a pseudo-static factor of safety equal to one. In theory, the landfill would displace during a seismic event when the site acceleration exceeded the yield acceleration. The yield acceleration for the landfill typical section through the Module 4 build-out was 0.12g. The yield acceleration for the excavation in native soil was 0.14g.

In the simplified method of Makdisi and Seed, the displacement is dependent upon the relationship of the yield acceleration for the slope to the maximum average acceleration ( $k_{max}$ ) for the sliding mass. Based on dynamic analyses performed for the design of Module 3, the value of  $k_{max}$  for a potential sliding mass extending from the top of the landfill to the toe was 0.16g (EMCON 1992). Additional analyses were performed to verify the accuracy of the value used for Module 3 utilizing the computer program PROSHAKE (EduPro 1998). Using PROSHAKE, the Module 4 configuration was subjected to acceleration time-history records from both a synthetic earthquake and the Loma Prieta Earthquake (Fremont station). The earthquake records were scaled to a maximum acceleration of 0.35g for use as the input ground motion in bedrock beneath the site. The response of the landfill to the input ground motions was evaluated using the simplified method to determine  $k_{max}$  for the Module 4 configuration. The value of  $k_{max}$  was 0.14 to 0.17 for the synthetic earthquake and the Loma Prieta Earthquake, respectively. The value of  $k_{max}$  used for Module 3 was verified to be accurate, and was used for the current analyses.

Based on the simplified procedure and a value of 0.16 for  $k_{max}$ , the estimated displacement of the typical landfill slope would be on the order of 3 to 10 centimeters. The estimated displacement of the 2:1 excavation slope would be on the order of 1 to 2 centimeters. The potential displacement for the west cut slope would not involve the waste mass based on the analyses. The estimated displacements obtained by the simplified procedure by Makdisi and Seed have been found to provide generally conservative results (Seed and Bonaparte 1992). With such low amounts of estimated displacement, further rigorous analyses do not appear to be necessary. A displacement of 10 centimeters is below the typical industry standard of a maximum of 30 centimeters.

#### 4. REVISED BASE GRADING AND LCRS LAYOUT

The existing conceptual grading plan for Module 4 called for a bottom excavation to be cut to an elevation of 3 meters mean sea level (MSL). Following the results of the field investigation (Vector 2001), the bottom of the *in situ* clay lens underlying Module 4 was determined to be at approximately 3 meters MSL. In addition, the highest recorded groundwater elevation from the -2 foot aquifer (permit aquifer) was recorded at 1.7 meters MSL. In order to get groundwater separation (minimum 1.5 meters) and maintain natural containment below the liner, a new grading plan was proposed that reduced the depth of excavation to an elevation of 4.3 meters MSL. While raising the bottom excavation of Module 4 would maintain groundwater separation and natural containment, the loss of disposal volume would occur unless changes were made to the bottom configuration.

The original conceptual grading plan for Module 4 featured two east-west drainage swales that sloped at 1.4 percent from a higher elevation at the east end of the cell to two sumps at the toe of the west slope. A cost/benefit analysis was conducted with a modified grading plan consisting of three swales sloping at 1.0 percent from a higher elevation at the east end of the cell. The leachate collection system design was also modified to have only one leachate collection sump. The following cost/benefit comparisons are relative to the original design basis.

##### 4.1 Reduction in Excavation Depth

The modified grading plan results in a reduction of clay excavation volume of 31,395 cubic meters when compared to the original design. Using an average clay excavation cost from actual contractor bids on Module 4 of \$2.41 per cubic meter, the modified grading plan results in a cost savings of \$75,552. However, the reduced waste volume associated with the proposed grading plan results in a revenue loss of \$615,915 (based on waste disposal fee of \$19.62 per cubic meter). Thus, the modified grading plan results in a net loss of \$540,363.

##### 4.2 Increase in Number of Drainage Swales

The increase in the number of drainage swales proposed in the modified grading plan results in an increase in clay excavation volume of 19,452 cubic meters over the original design. This results in an excavation cost increase of \$46,811 (using \$2.41 per cubic meter). This cost increase is offset by an increase in waste disposal revenue of \$381,615 (using \$19.62 per cubic meter disposal fee). The result of the increase in the number of drainage swales is a net gain of \$334,804.

### 4.3 Reduction in Swale Grade

By reducing the swale grade within the LCRS, an increase in clay excavation volume could be realized. However, the *in situ* clay below the Module 4 area will undergo settlement during refuse loading. In order for the efficient operation of the LCRS, it was determined that a minimum gradient of 0.8 percent must be maintained toward the sump after settlement. The settlement of the *in situ* clay and fine-grained soil layers beneath the Module 4 area was calculated based on the laboratory test results from samples taken during the field investigation (Vector 2001).

The differential settlement at 30-meter intervals along the drainage profile was calculated. The estimated settlement at the west end of the module was approximately 15 cm (resulting in a final separation from groundwater of 2.7 meters). The estimated settlement at the east end of the module was approximately 45 cm. The required gradient was evaluated for both 30-meter incremental gradients and the overall gradient across the module. The results (summarized in Table 3) indicated that a minimum pre-settlement gradient of 0.95 percent was required to end up with a gradient of 0.8 percent after settlement of the foundation soils. A design grade of 1.0 percent was; therefore, selected.

The reduction in swale grade in the modified grading plan from 1.4% to 1.0% resulted in an increase in clay excavation volume of 15,381 cubic meters. This results in an increased excavation cost of \$37,015. This cost increase is offset by an increase in waste disposal revenue of \$301,755. The result in the reduction of swale grade is a savings of \$264,740 over the original design.

Maintaining the base grades of the MPL at a higher elevation to stay within the bottom *in situ* clay layer had important environmental benefits. This would, however, result in a loss of revenue to the District if the same slopes and cut design were used. In order to compensate for the loss in base grade elevation, modifications to the slope and the number of drainage swales within the LCRS were proposed. The change in slope was determined to be acceptable following a settlement evaluation based on the consolidation testing performed during the field investigation (Vector 2001). The modifications to the base grading plan and LCRS swale design resulted in an overall savings of \$59,181 as compared to the original design.

Table 3. Change in Gradient of Bottom Liner due to Settlement of Foundation Soils.

Station	Total Settlement (cm)	100-foot Incremental Settlement (cm)	Required* Pre-settlement Grade %	Overall Differential Settlement (cm)	Required* Grade %
0+00	14.9	-	-	-	-
1+00	18.9	4.0	0.93	4.0	0.93
2+00	22.6	3.7	0.92	7.6	0.92
3+00	24.4	1.8	0.86	9.4	0.90
4+00	25.0	0.6	0.82	10.1	0.88
5+00	26.5	1.5	0.85	11.6	0.88
6+00	27.7	1.2	0.84	12.8	0.87
7+00	30.2	2.4	0.88	15.5	0.87
8+00	33.2	3.0	0.90	18.3	0.88
9+00	36.6	3.3	0.91	21.6	0.88
10+00	40.8	4.3	0.94	26.2	0.89
11+00	45.7	4.6	0.95	30.5	0.89

Required pre-settlement grade based on final minimum grade of 0.8 percent.

## 5. PERCHED GROUNDWATER DEWATERING SYSTEM

The grading plan discussed within this paper called for the floor of Module 4 to be positioned within the 3.3 to 5.5-meter thick *in situ* clay layer beneath the overlying red sand and below the 35-foot perched aquifer. The 35-foot perched aquifer consisted of a perched groundwater zone above the *in situ* clay layer. Therefore, it was necessary to dewater the construction area in order to excavate the module to the proposed depth. The additional airspace gained by excavating the module below the perched groundwater was considerable. The dewatering of the module incorporated several improvements to each of the perimeter slopes.

Because the 35-foot aquifer within the red sand flows from south of the landfill into Module 4, the construction of a dewatering trench along the southern side of the module was necessary to intercept this flow and direct it to a withdrawal point. The trench was constructed as a trapezoidal channel with a 0.5% slope to the east. The trench was positioned on top of the *in situ* clay at elevations ranging from 5.7 meters MSL down to 4.3 meters MSL at the discharge into the holding pond. Perched water flows across the top of the clay and into the trench, at which point it flows to the east into the holding pond. A dedicated pumping operation at the east end of the pond lifts the water from the holding pond to either an above ground storage tank or to the existing surface drainage channel which flows to the east end of the property.

Although the primary groundwater flow direction of the 35-foot aquifer is from the south, residual groundwater may drain into the excavated module from the western cut wall. To control this flow, a side-drain system was constructed along the top of the *in situ* clay layer along the west slope that provided a flow conduit for any incoming groundwater. This system was constructed by temporarily over-steepening the sands directly above the clay layer and cutting a "v" ditch along the top of the clay at approximately a 1.0% flow line to the south.

This cut and underdrain were designed to extend approximately 1.5 meters above the top of the clay layer to ensure adequate drainage of the aquifer. A perforated HDPE pipe wrapped in drainage gravel and geotextile were installed in the "v" ditch located approximately 1.8 meters inside the final limits of the composite liner. The drain system was capped with almost a meter of compacted clay along the entire exposed face of the drain system in order to rebuild the slope to its final configuration. At the southern perimeter of the system, the perforated pipe was connected to a solid walled pipe that was sealed in a clay plug to assist in directing the flow into the perforated pipe. The pipe discharges into the southern trapezoidal channel at elevation 5.7 meters MSL. The installation of this side-drain prevents groundwater from backing up against the liner creating pore pressure build-up and potential instability.

The stability of the temporary cut along the western wall for the installation of the underdrain was evaluated for both an overall failure of the slope and for minor sloughing of the sand where temporary over-steepening is performed. The same methods described previously in this paper were used to evaluate the stability of the temporary cut. The factor of safety for the overall slope was not significantly affected by the excavation of the temporary cut near the toe. The factor of safety was equal to 1.4 for the overall slope with the temporary cut (prior to buttressing with waste). The angle of repose for the sand when dry is approximately equal to 33 degrees, or 1.5:1, thus the factor of safety for the temporary cut is near 1.0, if the beneficial effects of negative pore pressure within the moist sand are ignored. As observed during the field investigation, temporary cuts will remain almost vertical when the sand is moist but not saturated.

## 6. CONCLUSION

The cost/benefit analysis conducted during the design and construction of the Module 4 expansion cell at the MPL resulted in a savings to the District of over one million dollars. The result of this cost savings was a cooperative effort between the District, their design consultant, and the regulating agency. By working together as a team, modifications to the existing permitted design were evaluated and approved resulting in a project that was protective of the environment while at the same time being cost effective for the District.

Landfill owners and operators often select consultants based on the lowest design costs provided in a proposal. This practice results in proposals that only follow the work that was done before in order to keep costs low. During the consultant selection process for the Module 4 design, the District did not select the lowest cost proposal. The winning proposal was 40% higher in overall costs than the lowest cost proposal. While the District paid more in initial design costs, their investment paid off over 25 fold in construction savings and a gain in refuse capacity.

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