REVISION
TO
VOLUME IV OF IV REPLACEMENT
AMENDMENT TO FINAL CLOSURE PLAN
LOPEZ CANYON SANITARY LANDFILL
LAKEVIEW TERRACE, CALIFORNIA

By
Bureau of Sanitation

Department of Public Works
City of Los Angeles
419 South Spring Street, Suite 800
Los Angeles, California

March 1997

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ADDRESSING CIWMB AND LEA'S COMMENTS TO FINAL CLOSURE PLAN

CITY OF LOS ANGELES

CALIFORNIA

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MAR 12 1997

Mr. Peter Janicki
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Permitting and Enforcement Division
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BOARD'S COMMENTS ON REPLACEMENT VOLUME IV OF THE FINAL CLOSURE PLAN FOR LOPEZ CANYON SANITARY LANDFILL

In response to your letter dated October 01, 1996, the Bureau of Sanitation (Bureau) would like to address the comments that both the California Integrated Waste Management Board (Board) and the Local Enforcement Agency (LEA) have regarding the revised closure plan for Lopez Canyon Landfill (Attachment A), prior to final approval being granted by both agencies. The Bureau acknowledges that the Regional Water Quality Control Board (RWQCB) also reserves the right to comment on the revised closure plan should significant changes occur.

The attached revisions to the final closure plan replace in full all prior pages within Volume IV of IV Replacement Amendment to Final Closure Plan, June 1996. The preceding Table of Contents addresses the attachments under this submittal, and the attached Summary Table of Revisions summarizes all the revisions to Volume IV of IV Replacement Amendment to the Final Closure Plan.

Comment No. 1:

"There are several inconsistencies in the closure estimate submitted with this closure plan revision. Specifically, on page 9-2, the text mentions that decrease in final elevations of the Disposal Area "C" will result in material and construction savings totaling $1,535,386. However, on the bottom of the same page, the final cover construction costs are shown to be reduced from $10,687,998 to $10,278,252 (difference of only $409,746). Also, it is unclear if the cost of demolishing and reconstructing the access road and perimeter drainage channel are additional construction costs induced by the decrease in the final elevation of Area "C".

In addition, Table 9-2 should include a statement explaining that the final construction costs for Area "C" are incorporated under "Other Activities" and that the total final cover costs is a sum of the "Final Cover" and a part of "Other Activities" items.

These issues have been already discussed with Mr. Jeff Dobrowolski of your staff during several recent telephone conversations. Mr. Dobrowolski has agreed verbally to revise both the relevant portions of text and the closure cost estimate to address Board staff concerns."

Response:

The Closure Estimate: Section 9.2.1, page 9-2, third paragraph, has been revised to reflect the correct estimated cost of the geotextile cushion and VFPE geomembrane for the deck and bench areas of Disposal Area C of $785,740, and the modified construction savings of $1,466,586, with a final cover construction cost reduction from $10,687,998 to $9,221,412. Attachment B replaces Section 9 of the closure plan in its entirety. These text modifications do not affect the closure cost estimate.

The cost of demolishing and reconstructing the haul road and drainage channel is necessary since it was determined that trash was found beneath both areas.

Table 9-1 of the closure plan has been revised to clarify the cost summary. See attachment C.

Additionally, it should be noted that in a conversation with you and Reina Pereira of my staff on November 18, 1996, Ms. Pereira informed you that we would not be replacing the two abandoned lysimeters, since the Regional Water Quality Control Board (RWQCB) concurred that the gas collection indicator probes located around the site are adequate for vadose zone monitoring. Therefore, the City is requesting reimbursement for the lysimeter abandonment work which was estimated to be $8,400. Attachment D includes Section 2.6.3 of the Monitoring Systems Report submitted to the RWQCB in August 1994, along with a followup letter from the City to the RWQCB dated November 04, 1994, that discusses the City's intentions with respect to vadose zone monitoring.

Comment No. 2:

"The revised plan indicates that the top deck and benches of Area "C" will incorporate a 40-mil very flexible polyethylene (VFPE) synthetic membrane (smooth on the top deck and textured on the benches). However, Section 2.3, Revised Final Cover Configuration, does not include detailed design information or any design justification which is expected from a final closure plan. Specifically:

a. No technical specifications are provided for the VFPE to be used on Area "C". The plan must include a set of minimum specifications for the synthetic membrane which are acceptable for the proposed design.

b. The plan must include calculations supporting use of VFPE (shear stress, VFPE elongation vs. differential settlement, anchorage, etc.).

c. The plan must include design drawings showing synthetic membrane system key points (anchors [if present], key points, pipe intercepts, etc.).

d. The plan must provide supporting documentation used to establish the minimum design yield point and its interpretation as "the point on the stress-strain curve at which the tangent modulus first becomes 290 psi."

e. The plan must provide design drawings for portions of the access road which is to be constructed over disposal areas (over final cover). Please include culvert details and final cover protection features.

While the text of the closure plan refers to the synthetic membrane to be used in the final cover as very flexible polyethylene (VFPE), Appendix I (Revised Construction Quality Assurance Plan) addresses the synthetic membrane as Very Low Density Polyethylene (VLDPE). It is our understanding that the VLDPE material is either no longer available or very difficult to obtain in large quantities. Thus, we request that the synthetic membrane terminology remain consistent throughout the closure documents."

Response:

(a) Technical specifications for the 40-mil very flexible polyethylene (VFPE) synthetic membrane to be used on Area C are included under attachment E and should be inserted into the Tables Section, Table 2-1 of the final closure plan.

(b) GeoSyntec has provided an analysis of the VFPE geomembrane to be used on the deck and benches of Area C. See Attachment F. This is to be included under Appendix III of Appendix H of the final closure plan. Section 3.2 of Appendix H has been revised to reflect this reference.

(c) Drawings showing the extent of the synthetic membrane and corresponding key points in Area C are included in the Figures Section, Figure 2-1(a) through 2-1(e).

(d) Appendix I, "Revised Construction Quality Assurance Plan," of the closure plan has been revised to include an appendix that provides supporting documentation used to establish the minimum design yield point. See attachment G.

(e) Design drawings for the haul road which is to be reconstructed over refuse to the north of area AB+ have been included in the Figures Section, Figures 2-4 and 2-4(a) of the closure plan. See attachment E.

VFPE geomembranes include very low density polyethylene (VLDPE) and linear low density polyethylene (LLDPE). Since the appendices of the closure plan were previously approved by your office on October 10, 1995, the City is requesting that any reference made to VLDPE of LLDPE in the Appendices be assumed to fall under the general VFPE geomembrane definition as stated in the text. Section 2.3.1 of the closure plan has been revised to clarify this issue. See attachment E.

Comment No. 3:

Section 2.3.2, Revised Final Cover Configuration, Disposal Area A, B, and AB+ Deck Areas, states that the geotextile between the vegetative layer and low permeability layer had been deleted. Please provide an explanation why this change occurred.

Also, the same section of text states that a geosynthetic clay liner (GCL) may be used as a barrier layer in the event a low permeability source is not available. Since the current submittal is identified as the final closure plan, the issue of securing sufficient volumes of cover material should be already resolved. Since the current cost estimate accounts for a clay low permeability material at a specific cost, this statement raises concern about the accuracy of the cost estimate.

Board staff indicated the above concern to Bureau of Sanitation (BOS) staff and was informed that the low permeability material for remaining portions of the landfill will be handled under a separate bid and the choice of the material will depend not only on its availability but also on economical conditions within the BOS (utilizing existing BOS work force, agreement with labor union, etc.) Thus, we request that the plan include *an explanation of this approach along with the anticipated time frames.*

As agreed during the meeting, should the low permeability type and/or its source differ from the current one, appropriate steps will be taken to update the current plan.

These will include:

a. A new test pad and permeability tests conducted prior to implementation of low permeability layer installation. This requirement shall be enforced in the event that a GCL is proposed instead of clay as a low permeability barrier.

b. An updated grading plan for the affected areas and supplemental QA/QC plan along with updated postclosure maintenance plan. This requirement shall be enforced in the event that a GCL is proposed instead of clay as a low permeability barrier.

For the purpose of the current plan revision, the text should include a section stating an intent to comply with the above conditions. The current closure plan should also acknowledge that all changes will be submitted as an amendment to the current plan and include updated cost estimated.

Finally, it must be acknowledged that in the event the costs of a changed design exceed the costs provided with the current plan, these additional costs will be absorbed by the BOS using additional funds.

Response:

The geotextile on the decks of areas A, B and AB+ has been deleted since it was originally intended as a barrier layer between the vegetative layer and low permeability layer. However, it has been determined by GeoSyntec Consultants, that it does not serve any additional purpose, it is not required, and it accounts for an additional cost savings.

The Bureau would like to reserve the option of using a geosynthetic clay liner (GCL) on the decks of Areas A, B AB+ and C. Final cover drawings using this option are shown in Figures 2-1(a) and 2-2(a), and technical specifications for GCL are shown in Table 2-2. Sections 2.3.1 and 2.3.2 of the closure plan have been modified to include the above revisions, (see attachment E). The Bureau will notify the Board, LEA and RWQCB, and update the grading plan, and postclosure maintenance plan, if this option is chosen. The QA/QC plan has been revised to include the GCL option, (see attachment G).

The GCL option will provide an easier, faster, less labor intensive, and more economical installation of the final cover on the decks as compared to the one foot of low permeability clay layer.

The Bureau acknowledges that in the event the costs of a changed design exceed the costs provided within the current plan, these additional costs will be paid by the City.

Should the low permeability type and/or its source differ from the current one, appropriate steps will be taken with respect to additional testing and updating the current plan.

Comment No. 4:

The limits of the refuse must be clearly shown on all appropriate drawings.

Response:

Drawing No. 5 has been added to the final closure plan to show the limits of refuse. See attachment H.

Comment No. 5:

The plan must include a more detailed drawing showing the design of benches on the northern face of the AB+ disposal area slopes. Specifically, the interface between the eastern edges of the benches and the sheet flow area should be shown in detail.

Response:

Drawing No. 1 submitted with Volume IV of IV Replacement Amendment to the Final Closure Plan has been revised. See attachment I.

Comment No 6:

The drawing depicting the drainage plan should include drainage patterns for the entire landfill in accordance with the design described in the revised plan.

Response:

Figure No. 3-1 and Drawing No.1 submitted with Volume IV of IV Replacement Amendment to the Final Closure Plan have been revised. See attachment I.

LEA COMMENTS DATED SEPTEMBER 29, 1996, AND BUREAU RESPONSES

Comment No. 1:
Appendix H, Final Cover Performance Evaluation, Section 3.1, Page 17

Remove reference to the reason for alternative cover being "would significantly reduce the volume of waste that Disposal Area C can accommodate." This is not a valid reason for selection of the alternative cover design because the facility closed before the utilization of total capacity. In this specific case, the LEA did not consider the reduction of waste capacity as a factor in the evaluation of the alternative final cover design. Please delete other text referring to reduced waste disposal capacity (e.g., Page 19, second paragraph, etc.).

Response:

Comment acknowledged. However, the Final Cover Performance Evaluation Report was submitted in January 1994 for approval of an alternative final cover for Disposal area C, when reduction in landfill volume was still a legitimate concern, and an integral part of the justification process.

Comment No. 2:
Section 9.2.1

Please revise Section 9.2.1 and remove the description of the geotextile cushion and associated cost discussion, as you have elected not to use the geotextile cushion, Revise Appendix F, Updated Closure and Post-Closure Estimates-Revised Initial Cost Estimate Worksheet (Amends Appendix K of Volume II of IV of the FCP and Table 4-1 of Volume II of II of the FPCMP) Line Item 21 (a) (3) and related cost items.

Response:

Section 9.2.1 discusses use of the geotextile cushion and costs related to it for the deck and benches of Disposal Area C only. This should not be confused with the City's decission to delete the use of the geotextile cushion on the decks of Disposal Areas A, B, and AB+. The City has requested to be reimbursed for the total estimated amount shown on line 21(a)(3) of Appendix F of the final closure plan.

Comment No. 3:
Appendix H, Final Cover Performance Evaluation Report, Page 18

The LEA requests all literature or documentation in your files on the reduction of the static safety factor on a sloped surface. Particularly, when the vegetative soiled layer is saturated, in a final cover design, and the vegetative soil layer is in direct contact *of a geomembrance and/or geotextile/geomembrane.*

Response:

Appendix H, page 18, discusses justification of the use of an alternative final cover for the slopes of Disposal Area C, namely the various analyses that were undergone to determine what frictional angle and slope angle would yield the required static factor of safety of 1.5. These analyses showed that placement of geotextile and/or geomembrane on the slopes of Disposal Area C would be both impractical and burdensome. The alternative final cover maintains the required factor of safety of 1.5.

Comment:

Additionally, the LEA had another comment in a letter to the Bureau dated February 25, 1997, requiring the Lopez Canyon Landfill proposed energy recovery facility (ERF) to be included in the Final Closure Plan.

Response:

Section 7, "Revised Landfill Gas Control System," of the Final Closure Plan, has been revised to include Section 7.3 on the proposed ERF. Figure Nos. 7-2 through 7-4 have also been included into the Figures Section of the closure plan. Refer to Attachment J.

****

The Bureau acknowledges that the Board will issue a conditional approval letter on the closure plan pending compliance with the CEQA requirements, since formal approval of the plan cannot be granted prior to the finalization of the CEQA documents.

If you have any questions regarding the above issues or the attached closure plan revisions, please contact Reina Pereira at (213) 893-8206.

Drew B. Sones BAJ
DREW B. SONES
Assistant Director

c: Joe Maturino, LEA
Rod Nelson, RWQCB
Kelly Gharios
Reina Pereira

a:ciwmbcom/rp.wp

**SUMMARY TABLE OF REVISIONS TO
VOLUME IV OF IV REPLACEMENT AMENDMENT TO
FINAL CLOSURE PLAN**

The following revisions and additions to the final closure plan address the CIWMB and LEA's comments of October 1, 1996, and September 30, 1996, respectively. Please ensure that these revisions are incorporated into your closure plan, and all previous sections discarded.

Sections, Details, Drawings to be Amended	Description of Change	Comment
Table of Contents	Replace in Entirety	Updated to reflect revisions/additions
Section 2: "Revised Final Cover Design"	Replace in Entirety	Revised Sections 2.3.1 & 2.3.2 to include VFPE and GCL specifications.
Section 7: "Revised Landfill Gas Control System"	Replace in Entirety	Include Section 7.3 on proposed Energy Recovery Facility.
Section 9: "Revised Closure Cost Estimate"	Replace in Entirety	Revised Sections 9.2.1 and 9.3 to include corrected final cover costs.
Tables	Add Table 2-1 Add Table 2-2 Replace Table 9-1	VFPE Properties GCL Properties Revised Summary of Closure Cost Est.
Figures	Replace Fig. 2-1 Add Fig. 2-1(a) Add Fig. 2-1(b) Add Fig. 2-1(c) Add Fig. 2-1(d) Add Fig. 2-1(e) Replace Fig. 2-2 Add Fig. 2-2(a) Add Fig. 2-2(b) Add Fig. 2-2(c) Add Fig. 2-2(d) Add Fig. 2-4 Add Fig. 2-4(a) Replace Fig. 3-1 Add Fig. 7-2 Add Fig. 7-3 Add Fig. 7-4	Revised Figure GCL option - C Cny (Deck) VFPE limits - C Cny Vertical well - C Cny (Deck) Vert. well, GCL option - C Cny (Deck) Downdrain Placement - C Cny (Bench) Revised Figure GCL option - A, B, AB+ Cny (Decks) GCL limits on Deck Areas Vertical well - A, B, AB+ Cny (Decks) Vert. Well, GCL - A, B, AB+ (Decks) Final Cover at Haul Road Final Cover at Haul Road, GCL option Revised Figure Site Map for Energy Recovery Facility (ERF) Floor Plan for ERF Process & Instrumentation Drawing for ERF
Drawings	Replace Dwg. No. 1 Add Drawing No. 5	Revised to include drainage and grading changes. Limits of Refuse
Appendix H: "Final Cover Performance Evaluation Report"	Add Appendix III to back of Appendix H	"Analysis of VFPE geomembrane"
Appendix I: "Revised Construction Quality Assurance Plan"	Replace in Entirety	Includes CQA for GCL option, and Appendix on justification of Design Yield Point for VFPE.

a:summtble.wpd/rp

ATTACHMENT A

Cal/EPA

California
Environmental
Protection
Agency

Integrated
Waste
Management
Board

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(916) 255-2200

OCT 0 1 1996

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Ken
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Environmental
Protection

Mr. Delwin A. Biagi
City of Los Angeles Department
of Public Works
Bureau of Sanitation
Suite 1400, City Hall East
200 North Main Street
Los Angeles, California 90012

Subject: Adequacy of the Replacement Volume IV of the Final Closure Plan for Lopez Canyon Sanitary Landfill, City of Los Angeles, Facility No. 19-AA-0820

Dear Mr. Biagi:

The California Integrated Waste Management Board (Board) received revised closure documents for the Lopez Canyon Sanitary Landfill. The documents received included:

- Cover letter dated June 14, 1996; and

- Final Closure Plan, Lopez Canyon Sanitary Landfill, Lakeview Terrace, California, Volume IV of IV Replacement Amendment to Final Closure Plan, dated June 1996.

After review of the revised closure and postclosure maintenance plan (plan), Board Closure and Remediation Branch staff have the following comments which must be addressed prior to the Board approval. In addition, the City of Los Angeles Environmental Affairs Department (Local Enforcement Agency [LEA]) staff have submitted their comments in the September 19, 1996 letter (copy attached). The comments from both agencies must be addressed prior to the plan approval. Majority of Board comments have been already verbally communicated to the City of Los Angeles Bureau of Sanitation staff during a meeting which took place on August 6, 1996 at the Lopez Canyon Sanitary Landfill.

Please note that although the Los Angeles Regional Water Quality Control Board (regional water board) staff have approved the revised plan on July 30, 1996, they have reserved the right to comment, should significant changes occur in the Plan, as a result of this review.

1. There are several inconsistencies in the closure estimate submitted with this closure plan revision. Specifically, on page 9-2, the text mentions that decrease in final elevations of the Disposal Area "C" will result in material and construction savings totalling $1,535,386. However, on the bottom of the same page, the final cover construction costs are shown to be reduced from $10,687,998 to $10,278,252 (difference of only $409,746). Also, it is unclear if the cost of demolishing and reconstructing the access road and perimeter drainage channel are additional construction costs induced by the decrease in the final elevation of Area "C".

   In addition, Table 9-2 should include a statement explaining that the final construction costs for Area "C" are incorporated under "Other Activities" and that the total final cover cost is a sum of the "Final Cover" and a part of "Other Activities" items.

   These issues have been already discussed with Mr. Jeff Dobrowolski of your staff during several recent telephone conversations. Mr. Dobrowolski has agreed verbally to revise both the relevant portions of text and the closure cost estimate to address Board staff concerns.

2. The revised plan indicates that the top deck and benches of Area "C" will incorporate a 40-mil very flexible polyethylene (VFPE) synthetic membrane (smooth on the top deck and textured on the benches). However, Section 2.3, Revised Final Cover Configuration, does not include detailed design information or any design justification which is expected from a final closure plan. Specifically:

   a. No technical specifications are provided for the VFPE to be used on Area "C". The plan must include a set of minimum specifications for the synthetic membrane which are acceptable for the proposed design.

   b. The plan must include calculations supporting use of VFPE (shear stress, VFPE elongation vs. differential settlement, anchorage, etc.).

   c. The plan must include design drawings showing synthetic membrane system key points (anchors [if present], key points, pipe intercepts, etc.).

   d. The plan must provide supporting documentation used to establish the minimum design yield point and its interpretation as "the point on the stress-strain curve at which the tangent modulus first becomes 290 psi."

e. The plan must provide design drawings for portions of the access road which is to be constructed over disposal areas (over final cover). Please include culvert details and final cover protection features.

While the text of the closure plan refers to the synthetic membrane to be used in the final cover as very flexible polyethylene (VFPE), Appendix I (Revised Construction Quality Assurance Plan) addresses the synthetic membrane as Very Low Density Polyethylene (VLDPE). It is our understanding that the VLDPE material is either no longer available or very difficult to obtain in large quantities. Thus, we request that the synthetic membrane terminology remain consistent throughout the closure documents.

3. Section 2.3.2, Revised Final Cover Configuration, Disposal Area A, B, and AB- Deck Areas, states that the geotextile between the vegetative layer and low permeability layer had been deleted. Please provide an explanation why this change occurred.

Also, the same section of text states that a geosynthetic clay liner (GCL) may be used as a barrier layer in the event a low permeability source is not available. Since the current submittal is identified as the final closure plan, the issue of securing sufficient volumes of cover material should be already resolved. Since the current cost estimate accounts for a clay low permeability material at a specific cost, this statement raises concern about the accuracy of the cost estimate.

Board staff indicated the above concern to Bureau of Sanitation (BOS) staff and was informed that the low permeability material for remaining portions of the landfill will be handled under a separate bid and the choice of the material will depend not only on its availability but also on economical conditions within the BOS (utilizing existing BOS work force, agreement with labor union, etc.). Thus, we request that the plan include an explanation of this approach along with the anticipated time frames.

As agreed during the meeting, should the low permeability type and/or its source differ from the current one, appropriate steps will be taken to update the current plan. These will include:

a. A new test pad and permeability tests conducted prior to implementation of low permeability layer installation. This requirement shall be enforced in the event that an alternate source of clay is chosen.

b. An updated grading plan for the affected areas and supplemental QA/QC plan along with updated postclosure maintenance plan. This requirement shall be enforced in the event that a GCL is proposed instead of clay as a low permeability barrier.

For the purpose of the current plan revision, the text should include a section stating an intent to comply with the above conditions. The current closure plan should also acknowledge that all changes will be submitted as an amendment to the current plan and include updated cost estimates.

Finally, it must be acknowledged that in the event the costs of a changed design exceed the costs provided with the current plan, these additional costs will be absorbed by the BOS using additional funds.

4. The limits of the refuse must be clearly shown on all appropriate drawings.

5. The plan must include a more detailed drawing showing the design of benches on the northern face of the AB+ disposal area slopes. Specifically, the interface between the eastern edges of the benches and the sheet flow area should be shown in detail.

6. The drawing depicting the drainage plan should include drainage patterns for the entire landfill in accordance with the design described in the revised plan.

Please note that California Environmetal Quality Act (CEQA) requirements must be complied with prior to approval of the closure and postclosure maintenance plan by the Board. BOS has reported that the CEQA documents are expected to be completed by February 1997. Therefore, although the plan may be considered technically adequate by Board staff prior to February 1997, formal approval of the plan cannot be granted prior to the finalization of the CEQA documents.

Should you have any questions, please contact me at (916) 255-1195.

Sincerely,

Peter Janicki
Closure and Remediation Branch
Permitting and Enforcement Division

Attachment

cc: Mr. Rod Nelson, Los Angeles Regional Water Quality Control Board

Mr. Joe Maturino, City of Los Angeles Environmental Affairs Department

CITY OF LOS ANGELES
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RICHARD J. RIORDAN
MAYOR

September 19, 1996

SEP 30 1996

Mr. Peter Janicki
California Integrated Waste Management Board
Closure and Remediation Branch
8800 Cal Center Drive
Sacramento, CA 95826

Subject: Amendment to the Lopez Canyon Sanitary Landfill - Final Closure and Final Post Closure Maintenance Plan (19-AA-0820)

Dear Mr. Janicki:

The City of Los Angeles, Local Enforcement Agency (LEA) has completed the review of the Amendment to the Lopez Canyon Sanitary Landfill - Final Closure and Final Post Closure Maintenance Plan.

The LEA approves of the changes in the final cover design. The approved final cover design consists of the following:

A. Disposal Area C Deck/Bench Areas
1. Vegetative layer with minimum of 24 inches thick.
2. 40-mil thick VFPE geomembrane.
3. 12 inch thick low-permeability layer (hydraulic conductivity < 1×10^{-6} cm/s).
4. 24 inch thick foundation layer.

B. Disposal Area A, B, and AB+ Deck Areas
1. Vegetative layer with minimum of 24 inches thick.
2. 12 inch thick low-permeability layer (hydraulic conductivity < 1×10^{-6} cm/s).
3. (Low permeability layer may be replaced by a GCL).
4. 24 inch thick foundation layer.

C. Disposal Area C Slope Areas
1. Vegetative layer with minimum of 24 inches thick.
2. 12 inch thick low-permeability layer (hydraulic conductivity < 1×10^{-6} cm/s).
3. 24 inch thick foundation layer.

D. Disposal Areas A, B, and AB+ Slope Areas
1. Vegetative layer with minimum of 24 inches thick.

2. 12 inch thick low-permeability layer (hydraulic conductivity < 1 x 10^{-6} cm/s).
3. 24 inch thick foundation layer.

The LEA approves of the Amendment to the Lopez Canyon Sanitary Landfill - Final Closure and Final Post Closure Maintenance Plan when the following information requests are provided and changes are made to the document.

1. Appendix H, Final Cover Performance Evaluation, Section 3.1, Page 17

Remove reference to the reason for alternative cover being *"would significantly reduce the volume of waste that Disposal Area C can accommodate."* This is not a valid reason for selection of the alternative cover design because the facility closed before the utilization of total capacity. In this specific case, the LEA did not consider the reduction of waste capacity as a factor in the evaluation of the alternative final cover design. Please delete other text referring to reduced waste disposal capacity (e.g., Page 19, second paragraph, etc.).

2. Section 9.2.1

Please revise Section 9.2.1 and remove the description of the geotextile cushion and associated cost discussion, as you have elected not to use the geotextile cushion. Revise Appendix F, Updated Closure and Post-Closure Estimates- Revised Initial Cost Estimate Worksheet (Amends Appendix K of Volume II of IV of the FCP and Table 4-1 of Volume II of II of the FPCMP) Line Item 21(a)(3) and related cost items.

3. Appendix H, Final Cover Performance Evaluation Report, Page 18

The LEA requests all literature or documentation in your files on the reduction of the static safety factor on a sloped surface. Particularly, when the vegetative soil layer is saturated, in a final cover design, and the vegetative soil layer is in direct contact of a geomembrane and/or geotextile/geomembrane.

If you have any questions, please call me at (213) 580-1070 or call David Thompson at (213) 580-1075.

Sincerely,

Joe Maturino

Joe Maturino
LEA Program Manager

c: Rod Nelson, LARWQCB
Del Biagi, LA City, BOS

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RICHARD J. RIORDAN
MAYOR

February 25, 1997

Mr. Lateef Sholebo
City Planning Department
221 North Figueroa, Suite 200
Los Angeles, CA 90012

Subject: Lopez Canyon Landfill (19-AA-0820) Proposed Energy Recove

Dear Mr. Sholebo:

The City of Los Angeles Local Enforcement Agency (LEA) has received and reviewed a Project Description, dated January 16, 1997, for the proposed Lopez Canyon Landfill Energy Recovery Facility. The project consists of installing two Caterpillar engines connected to two electrical generating units which will produce three megawatts each of electricity through the burning of landfill gas. We have the following comments on the submittal:

- The Governmental Approvals Required section, on pages 6-7 needs to be revised. The energy recovery facility is required to be included in the Lopez Canyon Final Closure Plan. Therefore, the plan must be amended to include this proposed facility. Amendment of the plan requires approval by the LEA, Los Angeles Regional Water Quality Control Board, and the California Integrated Waste Management Board.

- Since the main controlling document for the closure and postclosure maintenance of the site is the final closure plan, the LEA will consider both the Bureau of Sanitation and the project applicant as being responsible for compliance with the closure plan and applicable landfill closure statues and regulations.

- When the system shuts down, section 3.2.1.3 says that the system will be diverted to the existing BOS flares after twelve hours. The LEA believes that gas should be diverted to BOS flares within a much shorter time to prevent odors from landfill gas affecting the surrounding community. The applicant should provide this office with an alternative plan.

- Indicate how and where the landfill gas conveyance line will cross the road between the energy facility and the landfill.

- Submit a set of construction blueprints for the energy facility.

Should you have any questions, please call me at (213) 580-1068 or Mr. Joe Maturino, Program Manager at (213) 580-1070.

Sincerely,

Wayne Tsuda

Wayne Tsuda
LEA Program Director

c: Steve Fortune, City Bureau of Sanitation
Peter Janicki, CIWMB

WT.JM:clg/L97-084

ATTACHMENT B

9. REVISED CLOSURE COST ESTIMATE

9.1 General

This section presents the February 1995 revised cost estimate for closure of the Lopez Canyon Sanitary Landfill. This estimate supersedes the estimate presented in Section 11 of the PCP and supersedes the estimate presented in Section 8 of the amendment to the PCP (FCP) submitted in February 1994. The modifications to the closure cost estimate are related to the modifications in the final cover design and final grading, landfill gas control system, irrigation system, and surface-water drainage system. In addition, the City of Los Angeles maintains a fully funded trust fund for the entire value of the closure cost estimate.

9.2 Cost Categories

9.2.1 Final Cover

The Lopez Canyon Sanitary Landfill Disposal Areas A, B, AB+, and C are comprised of about 84 acres (34 hectares) of deck surface area and about 77 acres (31 hectares) of slope surface area. A minimum 24-in. (600-mm) thick layer of interim cover will exist over the entire landfill area once filling is complete. This cover is placed during the normal landfill operations at the site. The planned final cover for the deck area of Disposal Areas A, B, and AB+ and the slope area of Disposal Areas A, B, AB+, and C consists of a compacted low-permeability soil barrier layer approximately 12-in. (300-mm) thick, and a 24-in. (600-mm) thick protective soil vegetation layer.

The final cover design for the deck and bench areas of Disposal Area C consists of an 12-in. (150-mm) thick compacted low-permeability soil barrier layer, a 40-mil (1-mm) thick VFPE geomembrane, a 12 oz/yd^2 (410 g/m^2) nonwoven geotextile cushion, and a 24-in. (600-mm) thick protective soil vegetative layer. The final cover for the slope areas of Disposal Areas AB+ and C differs from the deck and bench areas of Disposal Area C in that no geotextile cushion or geomembrane is used. The deck/bench surface area of Disposal Area C is about 24.1 acres (9.8 hectares) while the slope surface area is about 10.9 acres (4.4 hectares). The deck surface area of Disposal Area AB+ is about 31.6 acres (12.8 hectares). The Disposal Area AB+ deck includes about 4.8 acres (2.0 hectares) and about 2,000 linear feet of the existing paved haul road and concrete trapezoidal perimeter channel to the north of the proposed access road. The slope surface area of Disposal Area AB+ is about 17.5 acres (7.1 hectares).

The revised cost estimate for final cover construction reflects the supply and installation of the geotextile cushion and VFPE geomembrane on the deck and bench areas of Disposal Area C, the revised quantity of earthen material used in the final cover for Disposal Areas AB+ and C, the changes in surface areas resulting from the final grading design modifications, and the need to reconstruct the existing haul road and perimeter channel.

Installation of the geotextile cushion and VFPE geomembrane is estimated to cost about $785,740 based on a unit cost of $0.75 per square foot ($8.07 per square meter) which includes construction quality assurance. The revised final grading design for Disposal Areas AB+ and C resulted in a decrease in earthwork quantities (i.e., low-permeability clay and vegetative cover). This resulted in a decrease of $1,722,585 in earthwork costs. The cost of demolishing and reconstructing those portions of the existing haul road and perimeter channel that overly waste has been estimated at $305,640. This resulted in a decrease of $1,466,586 in total closure costs. As a result of the above changes, the total cost of final cover construction has decreased from $10,687,998 to $9,221,412 in 1995 dollars. Note that this includes an increase of $359 for construction management costs and a reduction of $50,000 for closure plan costs that were considered when figuring the total cost reduction of closure construction.

9.2.2 Revegetation and Irrigation

Revegetation and irrigation costs cover the cost of soil preparation and planting of the vegetative cover, and temporary and permanent irrigation systems on the deck and slope areas, respectively. The revised revegetation and irrigation plan and figures are presented in Section 8 of this document. The revised cost estimate for revegetation reflects the decrease of about 5 acres (4 hectares) in the total surface area of the landfill to be revegetated. At a unit cost of about $3,225 per acre ($8,000 per hectare) for soil preparation, planting, fertilizing, and mulching, the revised surface area results in a revegetation cost savings of $16,125. The elimination of the temporary irrigation system on the deck areas resulted in an additional cost savings of $232,000. The permanent slope irrigation system has a unit cost of about $19,000 per acre ($47,000 per hectare). The revised final grading plan resulted in a decrease of slope surface area of about 16.5 acres (hectares). The revised surface area results in a decrease in irrigation costs of about $313,500. The total cost for revegetation and irrigation decreased from $2,382,350 to $1,821,823 in 1995 dollars.

9.2.3 Landfill Gas Control System

The cost estimate for the landfill gas control system is essentially unchanged from that presented in the FCP since the proposed vertical and horizontal landfill gas wells in Disposal Area C will already be in place when closure is implemented.

9.2.4 Surface-Water Drainage System

Costs for the surface-water drainage system include construction of the on-site drainage facilities. The revised cost for the surface-water drainage system reflects the decrease of about 5 acres (2 hectares) in the total landfill surface area and the corresponding changes to the surface-water drainage system presented in the FCP and which are described in Section 5 of this amendment. These changes result in: (i) a reduction of about 780 ft (240 m) in the total length of downchutes; (ii) a reduction of 6 inlet structures and bench crossings; (iii) the addition of about 1,000 ft (305 m) of diversion channel; and (iv) the addition of two splash walls.

In addition, several surface-water drainage elements included in the closure cost estimate presented in the FCP have either been: (i) built since the FCP was issued; or (ii) eliminated as a result of design modifications. These elements include: (i) three detention basins ($980,000); (ii) one debris basin ($180,000); (iii) 6,100 ft (1,860 m) of concrete trapezoidal channel ($176,530); (iv) 2,070 ft (630 m) of reinforced concrete pipe; (v) 6,000 square feet (560 square meters) of grouted riprap ($48,000); and (vi) 143,250 square feet (13,310 square meters) of 4-in. (100-mm) thick asphaltic concrete paving for access roads ($14,800). As a result of all the above changes, the total cost for the surface-water drainage system has decreased from $2,394,989 to $829,870 in 1995 dollars.

9.2.5 Security Installation

This category includes installation of the signs and perimeter fence and the cost is unchanged from that presented in the FCP.

9.2.6 Contingency

A 20 percent contingency factor has been added to the closure construction cost estimate presented in Section 9.3. This percentage is unchanged from the FCP.

9.3 Cost Estimate

Table 9-1 presents a summary of costs for the closure features previously described by category. The revised total cost for closure implementation has decreased from $21,849,558 to $17,538,990 in 1995 dollars. Any cost overruns that result from this cost estimate will be paid by the City. Appendix K of the FCP Volume II of IV has been revised to include the updated closure cost estimate. Appendix K is provided as Appendix F of this document.

ATTACHMENT C

TABLE 9-1

REVISED SUMMARY OF CLOSURE COST ESTIMATE
PARTIAL CLOSURE PLAN AMENDMENT
LOPEZ CANYON SANITARY LANDFILL

CLOSURE FEATURE		ESTIMATED COST (1995 Dollars)
Final Cover Construction*		$ 5,407,249
Revegetation/Irrigation*		$1,821,823
Surface-Water Drainage System Installation*		$829,870
Site Security Installation		$33,000
Other[1] (includes clay - C Deck, geotextile - C Deck & Benches, clay - all slopes, rebuilding portions of the haul road and drainage channel, landfill gas system modifications, ground-water monitoring modifications, vadose zone monitoring modifications, and construction management)		$6,523,883
I.	Subtotal	$14,615,825
II.	Contingency Costs (20 percent)	$2,923,165
III.	Total Closure Costs	$17,538,990

[1] Total final cover cost is the sum of "Final Cover Construction" costs and a portion of "Other" costs.

Note: * Cost estimate features changed from the PCP.

ATTACHMENT D

MONITORING SYSTEMS REPORT
LOPEZ CANYON SANITARY LANDFILL
LAKE VIEW TERRACE, CALIFORNIA

Prepared for:

Bureau of Sanitation
Department of Public Works
City of Los Angeles
419 South Spring Street, Suite 800
Los Angeles, California 90013
(213) 893-8211

Prepared by:

GeoSyntec Consultants
16541 Gothard Street, Suite 211
Huntington Beach, California 92647
(714) 843-6866

GeoSyntec Consultants Project No. CE4100-09

5 August 1994

2.6.3 Vadose Zone Monitoring

The BOS has been collecting samples of vadose zone liquids from lysimeters LYS-1 and LYS-2 -- the locations of which are shown on Figure 2-6. The BOS received approval from the RWQCB to abandon these lysimeters in conjunction with the final closure of Disposal Area A [RWQCB, 1994]. The lysimeters were abandoned in June, 1994.

Pending approval of the RWQCB, the BOS proposes to use landfill gas migration monitoring probes (GMMP) for vadose zone monitoring at the landfill. The BOS monitors gases on a monthly basis in 41 GMMP installed around the landfill. The GMMP are checked for organic compounds using a hand held organic vapor analyzer (OVA). Gas samples are collected from the two GMMP with the highest concentrations of organic compounds (as detected by the OVA) for analysis for VOC using the methodology of USEPA Method TO-14. The two gas samples collected for laboratory testing are also analyzed for oxygen, nitrogen, carbon dioxide, methane, and non-methane organic compounds (NMOC).

2.6.4 Surface Water Monitoring

The BOS currently monitors surface water run-off from the landfill semi-annually at the four locations shown on Figure 2-6. The existing four locations where surface water run-off samples are collected are (i) SWS-1, at the Canyon A outlet, (ii) SWS-2, at the Canyon B retention basin outlet, (iii) SWS-3, at the Canyon C retention basin outlet, and (iv) SWS-4, at the Haul Road subdrain pipe outlet. In addition to these surface water monitoring locations, the BOS proposes to collect and analyze samples of surface water from the following locations:

- the Area C subdrain line;
- the fill subdrain outlet down gradient of Area C near the pumping station; and

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NOV 04 1994

Dr. Robert P. Ghirelli
Executive Officer
California Regional Water Quality
Control Board (CRWQCB) - Los Angeles Region
101 Centre Plaza, Ca 91765-2156

Attention: Don Peterson, Senior Manager

GROUNDWATER MONITORING AT LOPEZ CANYON LANDFILL

Unless we hear otherwise from the CRWQCB, the Bureau of Sanitation plans to implement the procedures outlined in the Monitoring Systems Report submitted to the CRWQCB in August 1994. Monitoring will be conducted in December as per Monitoring and Reporting Program No. 5636. This will be the last sampling event conducted under this program.

If you have any questions, please call Ken Redd of my staff at (818) 834-5111.

DELWIN A. BIAGI,
Director

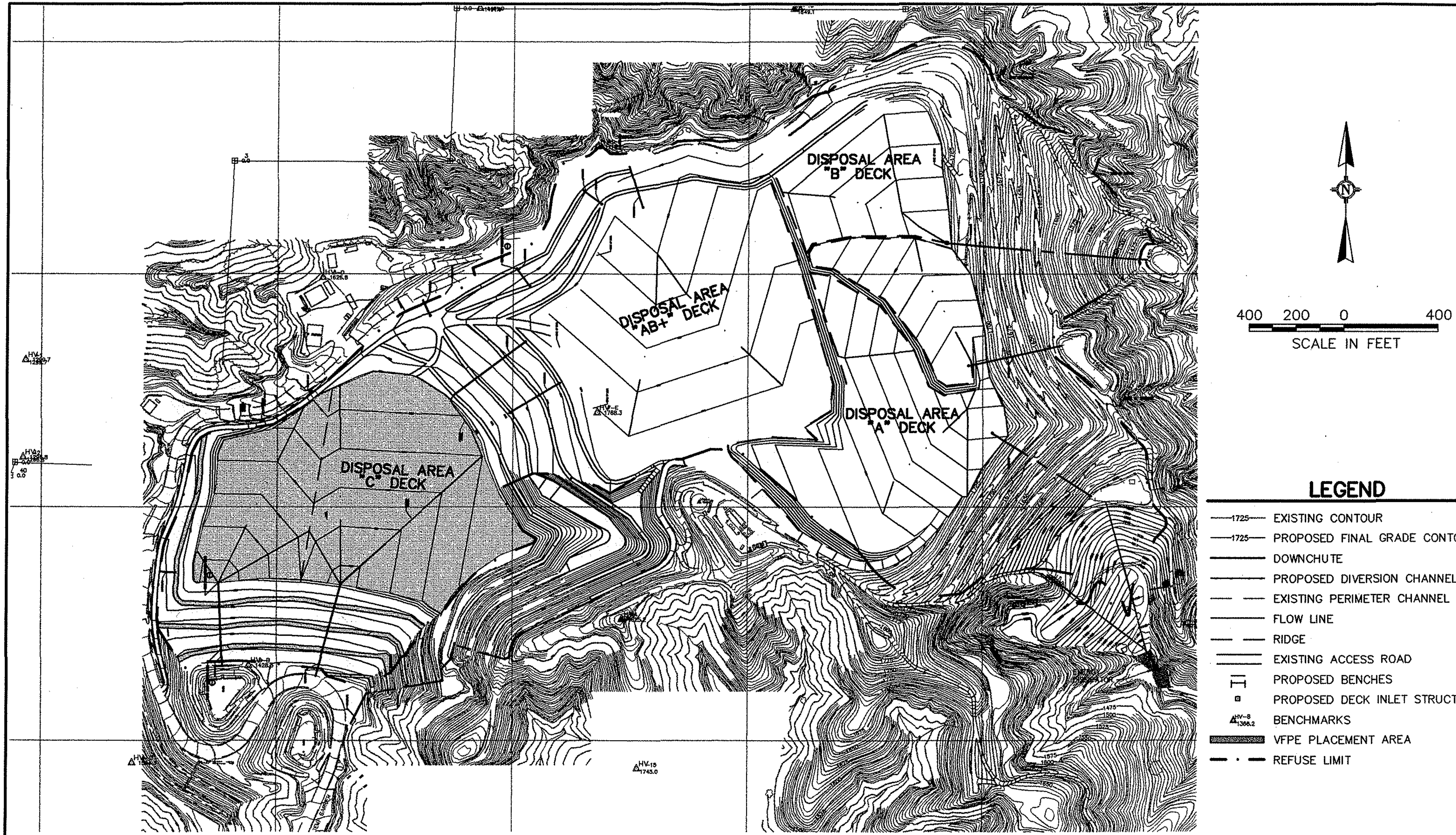
by:
Stephen A Fortune KRR
Stephen A. Fortune,
Division Manager
Solid Waste Management Division

KRR/RS:mep

c: S. Fortune
K. Redd
R. Strohm

(RWQCB_05)

ATTACHMENT E



LEGEND

- 1725 EXISTING CONTOUR
- 1725 PROPOSED FINAL GRADE CONTOURS
- DOWNCHUTE
- PROPOSED DIVERSION CHANNEL
- EXISTING PERIMETER CHANNEL
- FLOW LINE
- RIDGE
- EXISTING ACCESS ROAD
- PROPOSED BENCHES
- PROPOSED DECK INLET STRUCTURES
- HV-8 1386.2 BENCHMARKS
- VFPE PLACEMENT AREA
- REFUSE LIMIT

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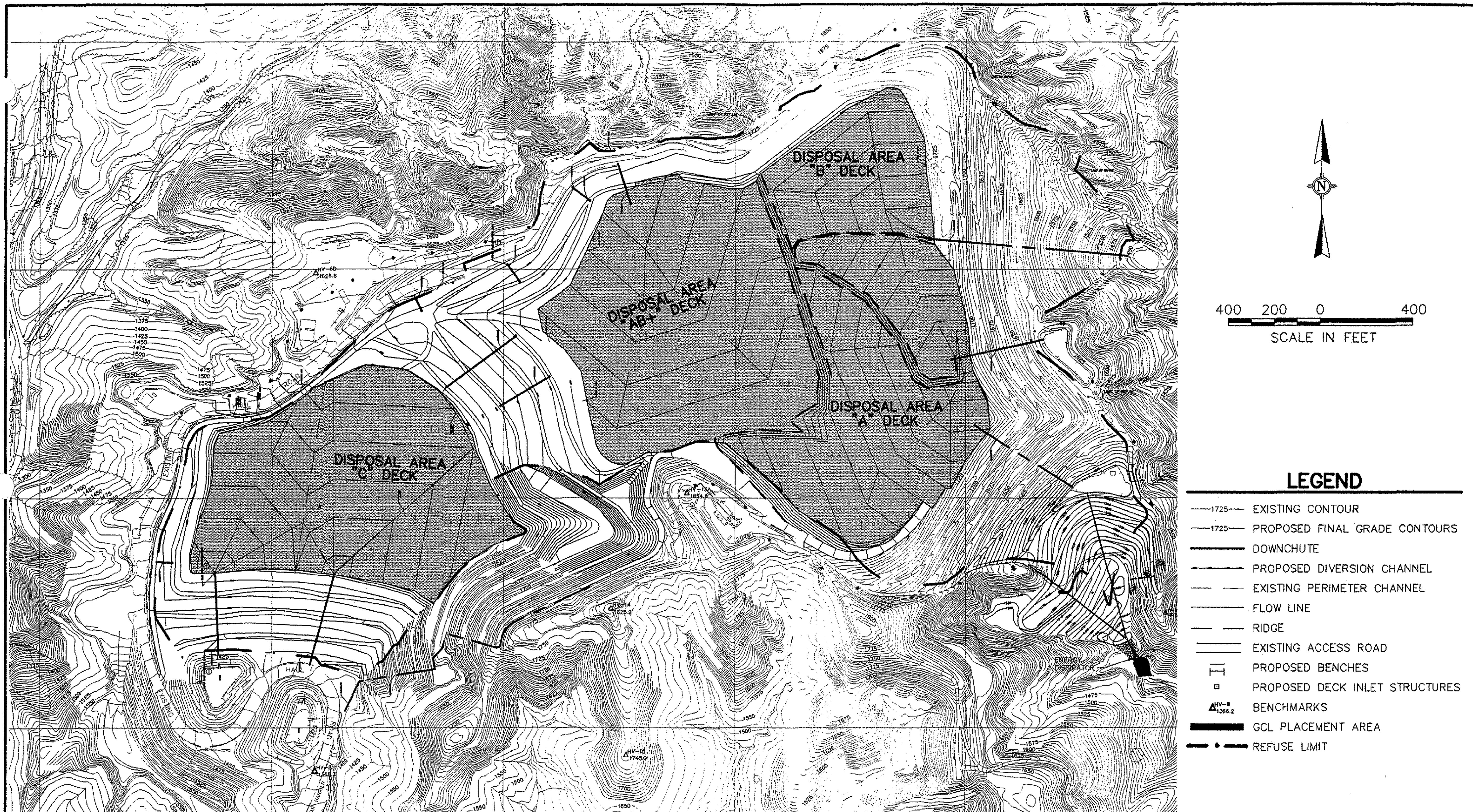
VERY FLEXIBLE POLYETHYLENE SYNTHETIC MEMBRANE (VFPE) PLAN DISPOSAL AREA A, B, AB+, AND C LOPEZ CANYON SANITARY LANDFILL LOS ANGELES, CALIFORNIA	FIGURE NO. 2-1 (b) PROJECT NO. CE4100-04 DATE: NOV-13-96

DWG: 4100FFML 199701160957 AC

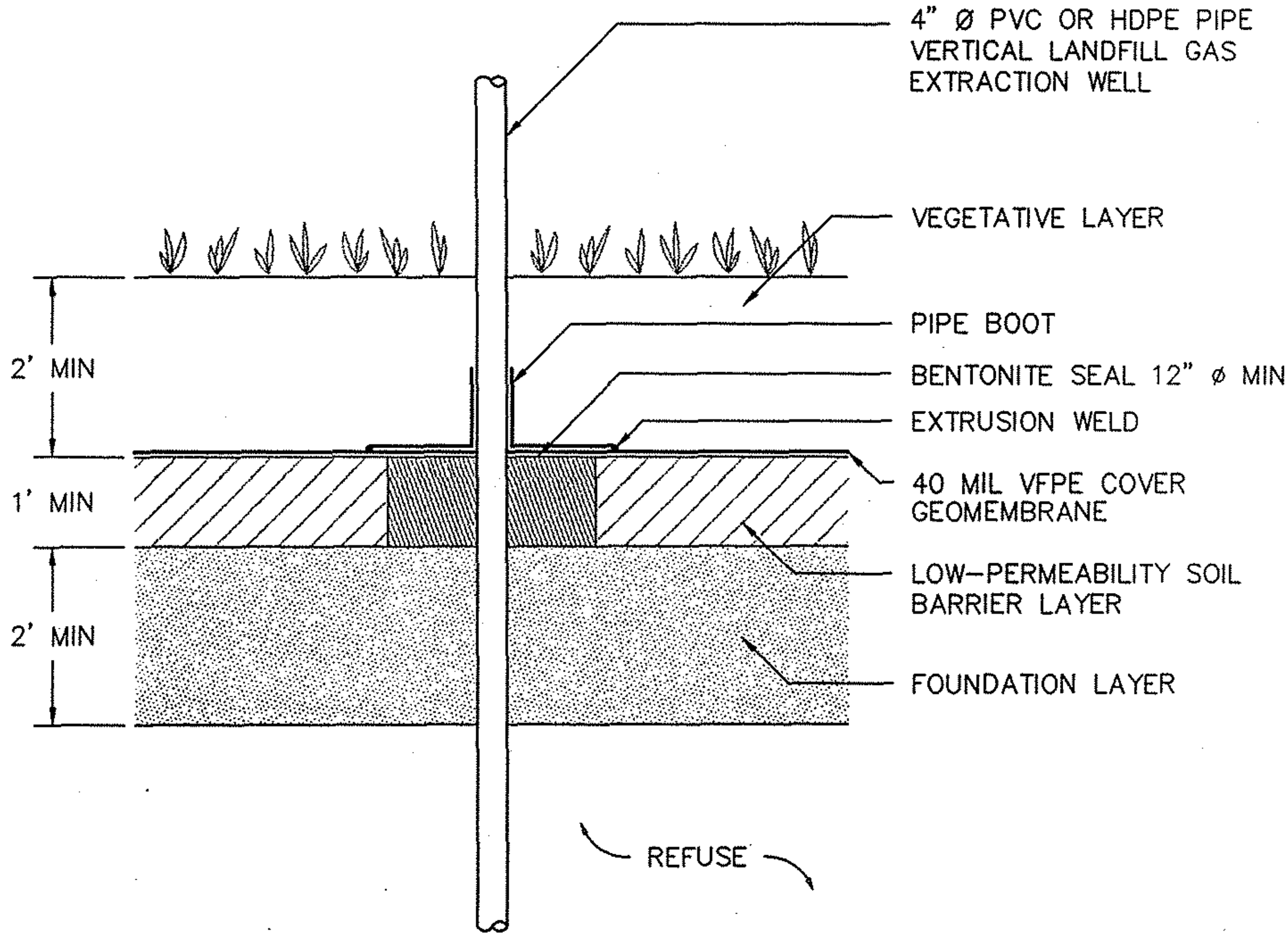


GeoSyntec Consultants

PROPOSED GEOSYNTHETIC CLAY LINER (GCL)
PLAN DISPOSAL AREA A, B, AB+, AND C
LOPEZ CANYON SANITARY LANDFILL
LOS ANGELES, CALIFORNIA

FIGURE NO.	2-2(b)
PROJECT NO:	CE4100-04
DATE:	NOV-13-96

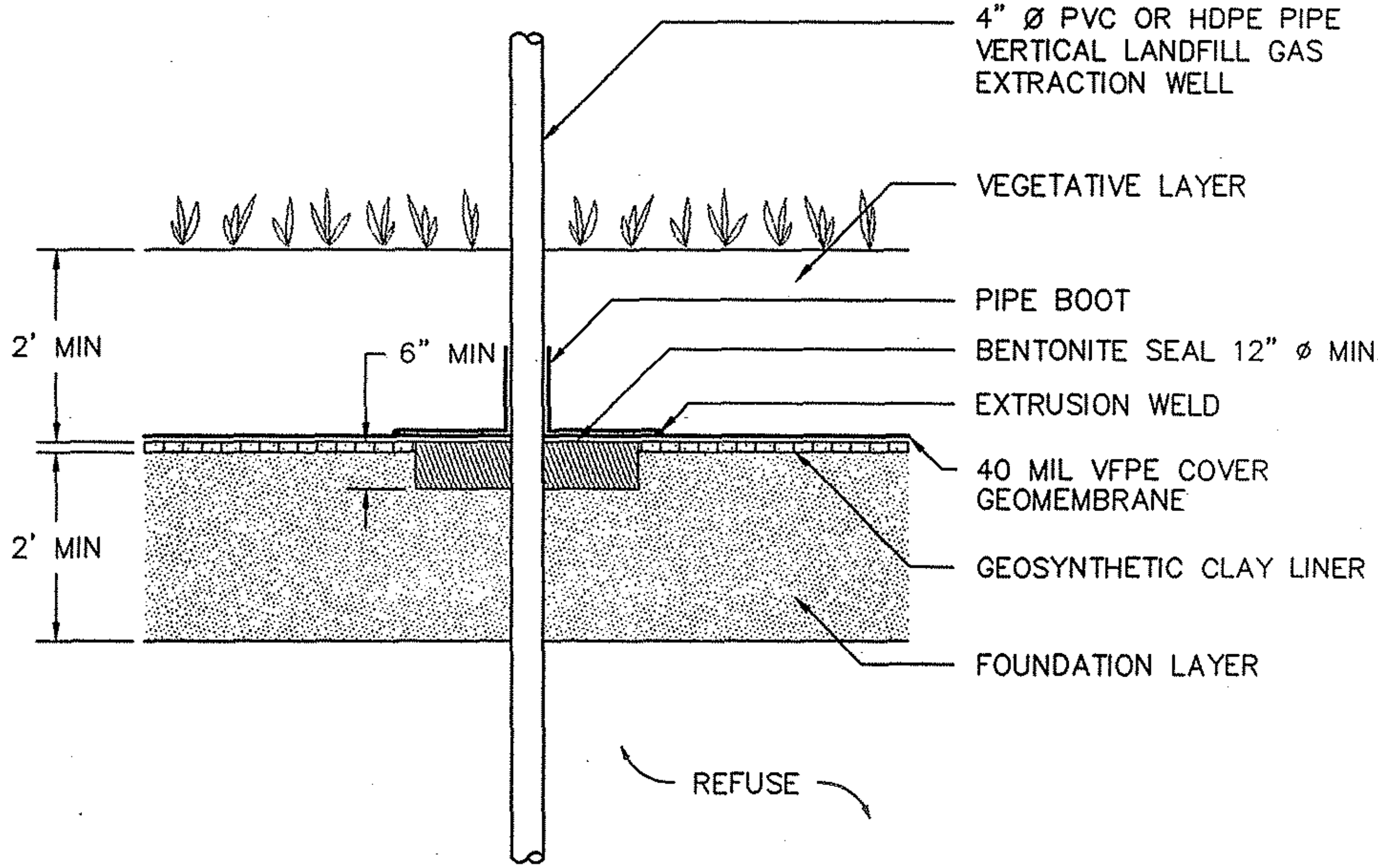
DWG: 4100FGCL 19961113



GEOSYNTEC CONSULTANTS

VERTICAL LANDFILL GAS EXTRACTION WELL PENETRATING FINAL COVER SYSTEM ON DECK WITH LOW-PERMEABILITY SOIL BARRIER LAYER AND GEOMEMBRANE LOPEZ CANYON SANITARY LANDFILL	FIGURE NO. 2-1(c)
	PROJECT NO. CE4100-06
	DATE: SEPT-04-96

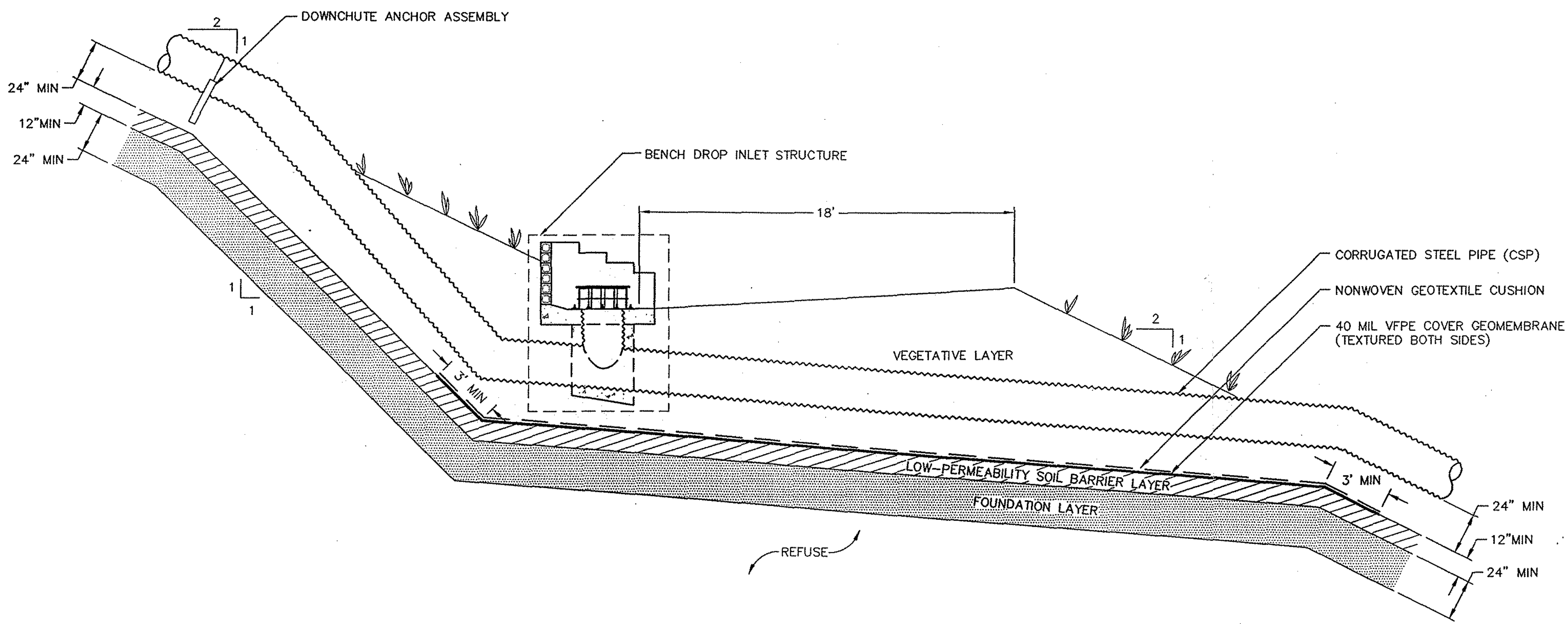
DWG: 4100F601 199609051153 LP



GEOSYNTEC CONSULTANTS

VERTICAL LANDFILL GAS EXTRACTION WELL PENETRATING FINAL COVER SYSTEM ON DECK WITH A GEOSYNTHETIC CLAY LINER AND GEOMEMBRANE LOPEZ CANYON SANITARY LANDFILL	FIGURE NO.	2-1(d)
	PROJECT NO.	CE4100-06
	DATE:	SEPT-04-96

DWG: 4100F601 199609051154 LP



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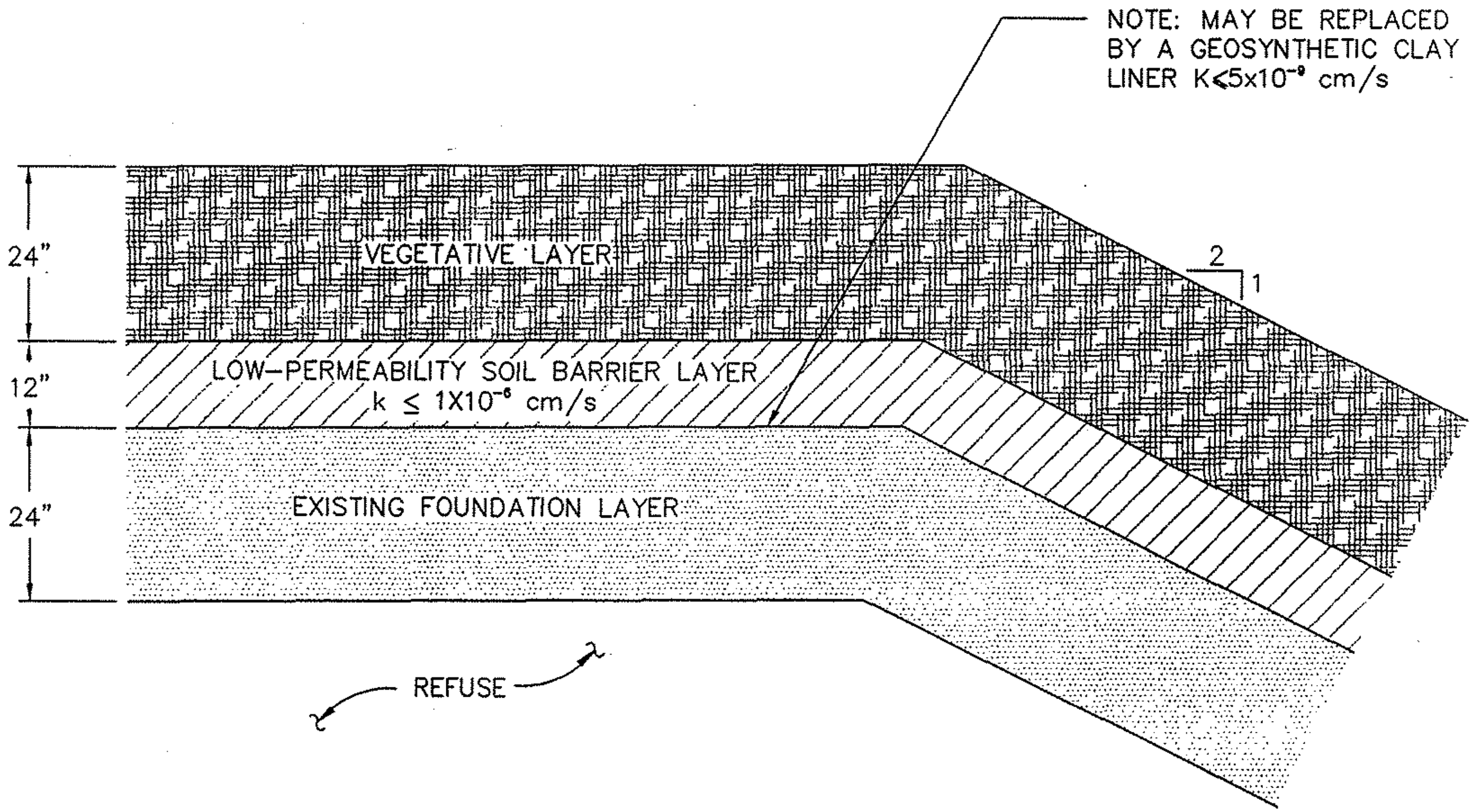
DOWNSHUTE CROSSING WITH CLAY, DISPOSAL AREA AT FINAL COVER BENCH LOPEZ CANYON SANITARY LANDFILL LOS ANGELES, CALIFORNIA	FIGURE NO. 2-1(e) PROJECT NO. CE4100-06 DATE: DEC-19-96

DWG: 4100F605 199612191600 LP

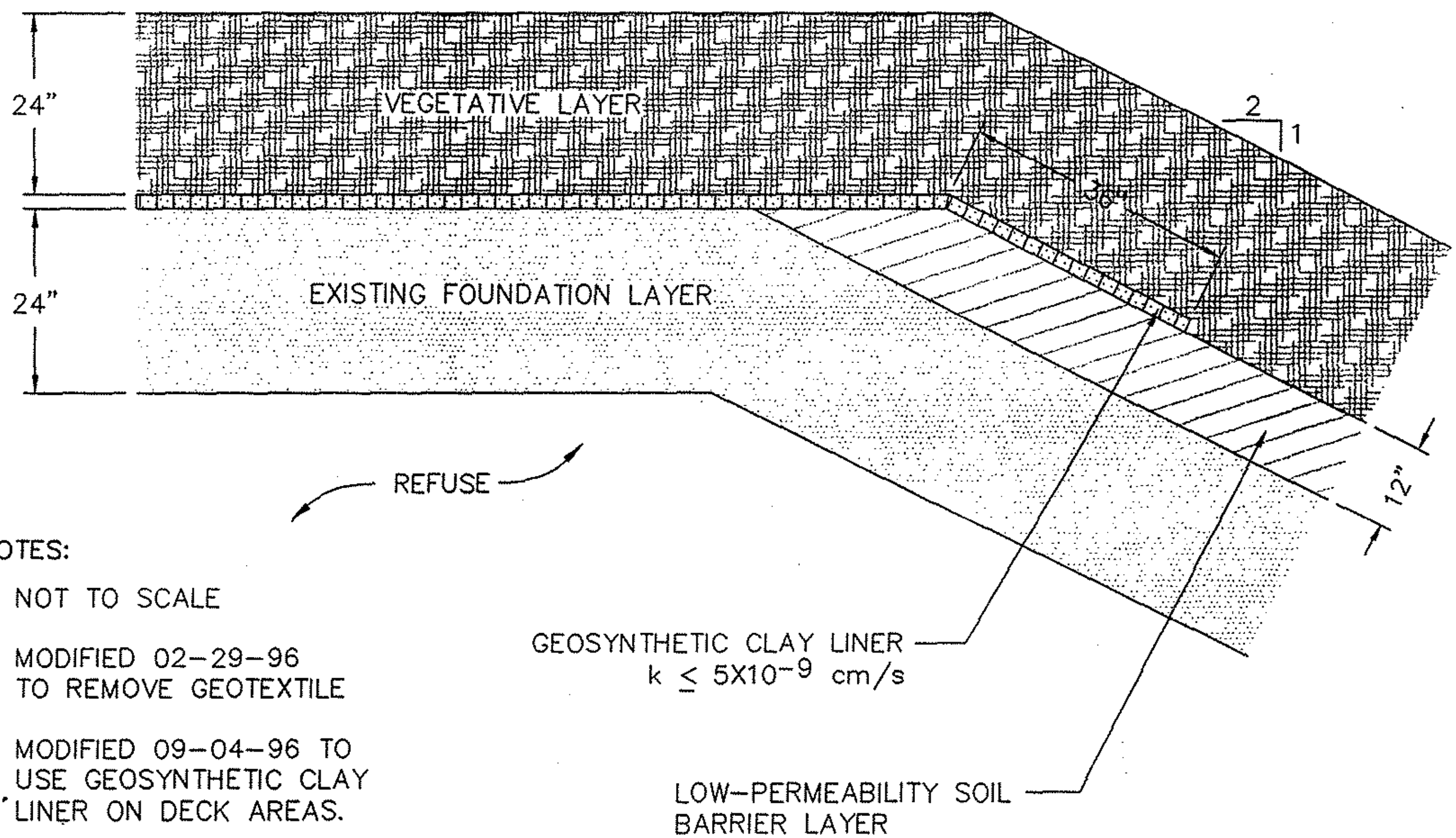


MODIFIED 02-29-96
TO REMOVE GEOTEXTILE

GEOSYNTEC CONSULTANTS

FINAL COVER ON DECK AREAS DISPOSAL AREA A, B, AND AB+ LOPEZ CANYON SANITARY LANDFILL	FIGURE NO.	2-2
	PROJECT NO.	CE4100-04
	DATE:	MAR-21-96

DWG: 4100F031 199611201752 AC



NOTES:

1. NOT TO SCALE
2. MODIFIED 02-29-96 TO REMOVE GEOTEXTILE
3. MODIFIED 09-04-96 TO USE GEOSYNTHETIC CLAY LINER ON DECK AREAS.

GEOSYNTEC CONSULTANTS

FINAL COVER ON DECK AREAS
DISPOSAL AREAS A, B, AND AB+
LOPEZ CANYON SANITARY LANDFILL

FIGURE NO.	2-2(a)
PROJECT NO.	CE4100-06
DATE:	SEP-04-96

DWG: 4100F604 199612161738 AC

2. REVISED FINAL COVER DESIGN

2.1 General

The final cover for Disposal Area C has been revised from the design presented in the PCP to conform to the requirements of Subtitle D, Chapter 15, and RWQCB Order No. 93-062 for final covers over bottom liners which include a geomembrane. This revised final cover design was submitted to the CIWMB in February 1994 and was approved on 10 October 1995. A copy of the approval is presented in Appendix G. The final cover presented in the PCP employed an infiltration barrier layer composed of compacted soil only. The revised design for Disposal Area C incorporates a geomembrane in the infiltration barrier layer in the deck and bench areas. The geomembrane was included in the deck and bench areas in accordance with the prescribed minimum construction standards of Subtitle D and Chapter 15. On the slopes of the waste face, an engineered alternative final cover is employed. The alternative slope final cover was designed in accordance with state and federal regulatory standards for a performance-based design of an engineered alternative final cover.

A performance evaluation of the Disposal Area C alternative slope final cover was conducted to demonstrate compliance with applicable state and federal regulations. The performance evaluation included an infiltration analysis and a slope stability assessment for the alternative slope final cover design. The performance evaluation also included a demonstration that the construction of the prescriptive final cover provided in state and federal regulations on the side slopes was burdensome and impractical and would not promote attainment of the performance goals for final covers, as required by the state regulations. A detailed presentation of the performance evaluation is contained in the Final Cover Performance Evaluation report presented as Appendix H of this addendum. A summary of the performance evaluation is presented herein.

2.2 Regulatory Framework

State of California regulations concerning design and construction of final covers for closure of municipal solid waste landfills are found in Title 14, Chapter 15, and RWQCB Order No. 93-062. Federal regulations for final covers are provided in Subtitle D. State and federal regulations both provide a minimum prescriptive construction standard for the final cover of Municipal Solid Waste Landfills (MSWLFs) that includes a protective vegetative erosion control layer and a low-permeability soil infiltration barrier layer. State regulations are somewhat more restrictive than federal regulations with respect to these layers, requiring a thicker erosion control layer and an order of magnitude lower hydraulic conductivity for the barrier layer. The state and federal regulations both require that the final cover have a "permeability" less than or equal to that of any bottom liner or underlying material. This requirement is generally interpreted as an implied prescriptive requirement that a geomembrane be included in the final cover barrier layer above areas which incorporate a geomembrane in the bottom liner. This "permeability" requirement is also interpreted as a performance standard requiring less infiltration of surface water through the final cover than liquid flux through the base of the landfill.

Based upon the state and federal regulations and considering that Disposal Area C does have a geomembrane bottom liner, the prescriptive final cover for Disposal Area C is inferred to consist of (from top to bottom):

- a vegetative layer at least 12-in. (300-mm) thick and of greater thickness than the rooting depth of any vegetation planted on the final cover;

- a geomembrane infiltration barrier;

- a compacted soil barrier layer not less than 12-in. (300-mm) thick with a maximum hydraulic conductivity of 1×10^{-6} cm/sec;

- a foundation layer at least 24-in. (600-mm) thick; and

- a design which provides for the minimum maintenance possible.

Both federal and state regulations provide for design of an alternative to the prescriptive final cover. Federal regulations allow the director of an approved state to approve an alternative design shown to be equivalent or superior to the performance of the prescriptive design with respect to infiltration and wind and water erosion. California is an approved state.

Section 17773. of Title 14 provides for the approval of alternative final covers when the owner demonstrates that:

- the prescriptive standard described in Chapter 15 is not feasible; and

- the engineered alternative is consistent with the performance goal of the prescriptive standard and provides equivalent protection to the ground water;

To establish that the prescriptive standard of Chapter 15 is not feasible, the owner must further demonstrate that the prescriptive final cover:

- is reasonably and unnecessarily burdensome and will cost substantially more; and

- is impractical and will not promote attainment of the performance goals.

The state and federal requirement that the final cover have a "permeability" less than or equal to the bottom liner or underlying material is generally interpreted as an implied final cover infiltration performance standard that the flux through the cover should be less than the flux through the base liner. United States Environmental Protection Agency (USEPA) has confirmed this interpretation of the implied prescriptive requirement and performance standard of the Subtitle D closure requirement in the "Final rule; corrections" for Subtitle D published in the Federal Register of 26 June 1992 (Vol. 57, No. 124, pp. 28626-28628). USEPA's comments on the prescriptive and performance standards for final cover design are discussed in detail in the Final Cover Performance Evaluation report presented in Appendix H.

The Final Cover Performance Evaluation report presented in Appendix H of this addendum contains the demonstration required by state regulations that construction of the prescriptive final cover on the slopes of the waste face of Disposal Area C is both burdensome and impractical and will not promote attainment of the performance goals for final covers. On the basis of this demonstration, an engineered alternative final cover for the Disposal Area C waste slopes was developed.

2.3 Revised Final Cover Configuration

2.3.1 Disposal Area C Deck/Bench Areas

The final cover on deck and bench areas of Disposal Area C satisfies the prescriptive standard in the California regulations. The deck and bench area final cover, shown in Figures 2-1 through 2-1(f), consists of the following components (from top to bottom):

- vegetative layer at least 24-in. (600-mm) thick;

- 12 oz/yd^2 (410 g/m^2) non-woven geotextile cushion;

- 40-mil (1-mm) thick very-flexible polyethylene (VFPE) geomembrane (smooth on the deck areas and textured on the bench areas). Technical specifications are shown in Table 2-1. Note that VFPE geomembranes include very low density polyethylene (VLDPE) and linear low density polyethylene (LLDPE), as noted in Appendices H and I;

- 12-in. (300-mm) thick barrier layer of compacted low-permeability soil, with a hydraulic conductivity no greater than 1×10^{-6} cm/s. A geosynthetic clay liner (GCL) with a hydraulic conductivity no greater than 5×10^{-9} cm/s may be used as a barrier layer for the deck area instead of the low-permeability soil. Technical specifications for GCL are shown in Table 2-2; and

- 24-in. (600-mm) thick foundation layer.

2.3.2 Disposal Area A, B, and AB+ Deck Areas

The final cover on the deck of Disposal Areas A, B, and AB+ has been modified from that presented in the PCP to delete the geotextile between the vegetative layer and the low-permeability soil barrier layer. In addition, a geosynthetic clay liner (GCL) with a hydraulic conductivity no greater than 5×10^{-9} cm/s may be used as a barrier layer. The use of a GCL will depend on the availability of low-permeability soil, ease of application, and economical feasibility. The modified final cover is presented in Figures 2-2 through 2-2(d).

2.3.3 Disposal Area C Slope Areas

An engineered alternative final cover was developed for the slope areas of the Disposal Area C waste face. The engineered alternative was developed on the basis of the demonstration included in Appendix H of this amendment, the Final Cover Performance Evaluation report, that inclusion of a geomembrane in the slope areas of the Disposal Area C final cover would be burdensome and impractical and would not promote attainment of the performance goals of a final cover. Use of a geomembrane in the final cover on the waste slopes was deemed burdensome and impractical due to constructability, stability, and cost considerations. Furthermore, the maintenance requirements for a slope final cover incorporating a geomembrane were deemed contrary to the performance goal of minimizing final cover maintenance.

The engineered alternative final cover design for the slope areas of the Disposal Area C waste face is shown in Figure 2-3. The final cover for the slope area consists of the following components (from top to bottom):

- vegetative layer at least 24-in. (600-mm) thick;
- 12-in. (300-mm) thick barrier layer of compacted low-permeability soil with a hydraulic conductivity no greater than 1 x 10^{-6} cm/s; and
- 24-in. (600-mm) thick foundation layer.

2.3.4 Disposal Areas A, B, and AB+ Slope Areas

The change in the final elevation of Disposal Area C has produced a split-deck final grading plan, with the deck of Disposal Area C at elevation 1,600 ft msl and the deck of Disposal Area AB+ at elevation 1770 ft msl. This split deck has created a need for construction of a final cover on the waste slopes of Disposal Area AB+ between the decks of Disposal Areas AB+ and C. The same final cover used on the Disposal Area C slopes will be used on the slopes of Disposal Areas A, B, and AB+. This final cover for the A and B slopes is different than that which was originally submitted in the PCP. The monolithic cover was replaced with the final cover as described in the above section. This modification was submitted to the CIWMB on 31 May 1994 and approved on 10 October 1995. A copy of the approval letter is presented in Appendix G. This final cover is shown in Figure 2-3 and described in the preceding section. As Disposal Areas

A, B, and AB+ are not underlain by a geomembrane liner, the final cover for the decks and benches in these areas do not require a geomembrane. The final cover conforms to the prescriptive design standard. Additionally, a portion of the haul road and perimeter channel in Disposal Area AB+ will be reconstructed to include a final cover, since refuse underlies this area. This final cover detail is shown in Figures 2-4 and 2-4(a).

2.4 Infiltration Analyses

Use of an engineered alternative final cover on the waste slopes of Disposal Area C requires a demonstration that the alternative design provides equivalent protection to ground water and resistance to infiltration compared to the prescriptive design. The potential for infiltration of surface water through the alternative final cover on the slopes of the waste face was evaluated using two USEPA-developed water balance models: (i) HELP Model Version 2 [USEPA; 1984 a,b]; and (ii) the SW-168 Model developed by Fenn et al. [1975]. The infiltration calculations are included in Appendix H of this addendum, the Final Cover Performance Evaluation report.

Neither the HELP nor the SW-168 Model predicted infiltration through the cover. One factor influencing the lack of infiltration is the high percentage of run-off from the 2H:1V Disposal Area C slopes. In addition, the annual precipitation is significantly less than the annual pan evaporation rate. As a result, the soil moisture storage capacity was not exceeded in either short term or long term conditions, resulting in no infiltration through the final cover barrier layer. Because there was no infiltration through the barrier layer, the engineered alternative final cover design for the Disposal Area C slopes meets the infiltration performance standard of less infiltration through the final cover than through the bottom liner.

2.5 Final Cover Slope Stability

Both one-dimensional (infinite slope) and two-dimensional slope stability analyses of the Disposal Area C final cover were performed. Slope stability calculations are included in Appendix H of this report, the Final Cover Performance Evaluation report. The one-dimensional slope stability analyses were performed using the methodology suggested by Matasović [1991]. Two-dimensional slope stability analyses were performed using the computer program PC STABL 5M [Achilleos, 1988].

One-dimensional stability analyses yielded a minimum (static) factor of safety of 2.0 for a failure surface passing through the waste immediately below the existing foundation layer. The corresponding pseudo-static factor of safety for a seismic coefficient of 0.2 was 1.41. GeoSyntec considers this pseudo-static factor of safety acceptable based upon the conclusions of Seed [1979]. Based upon observations of the performance of slopes and embankments in earthquakes around the world, Seed [1979] concluded that slopes designed with a pseudo-static factor of safety of 1.15 for a seismic coefficient of 0.15 experienced "acceptable" deformations (less than 1 ft (0.3 m)) in earthquakes of all magnitudes and intensities. However, to substantiate this conclusion, maximum permanent seismic displacements were estimated using charts developed by Hynes and Franklin [1984] using Newmark analyses. Predicted displacements for the critical final cover failure surface were on the order of 2 in. (50 mm) for the design peak ground acceleration of 0.69 g. Two-dimensional slope stability analyses yielded a minimum (static) factor of safety of 2.86 and a pseudo-static factor of safety of 2.0.

The infiltration analyses indicated the potential for development of down slope seepage parallel to the face of the slope within the vegetative cover layer was negligible, even for the 100-year, 24-hour storm. However, stability analyses were conducted for the limiting case of seepage parallel to the slope. Stability analyses for the condition of seepage parallel to the slope yielded a minimum (static) factor of safety of 2.5 for this condition.

The final cover on the slopes of the Disposal Area AB+ waste face will have the same cross section as the final cover on the Disposal Area C waste face. However, the inclination of the slopes on the Disposal Area AB+ waste face is 2.5H:1V, flatter than the 2H:1V inclination of the slopes on the Disposal Area C waste face. As the final cover on the Disposal Area C waste face was demonstrated to be stable, separate stability calculations for the flatter Disposal Area AB+ final cover were not considered necessary.

The stability calculations are included in Appendix H of this addendum, the Final Cover Performance Evaluation report.

TABLE 2-1

REQUIRED PHYSICAL PROPERTIES OF 40-MIL VFPE COVER GEOMEMBRANE FINAL CLOSURE PLAN LOPEZ CANYON SANITARY LANDFILL

PROPERTY	TEST METHOD	REQUIREMENTS
Thickness, mil.	ASTM D 751 (modified with Conical Tip)	36 mils (minimum) 40 mils (average)
Specific Gravity (g/cm^3)	ASTM D 792, ASTM D 1505	0.92 (minimum) 0.94 (maximum)
Min. Tensile Properties	ASTM D 638 (NSF 54) (3)	
• Tension at Yield (lb/in)	(20 in. per min.)	50
• Tension at Break (lb/in)		145
• Strain at Yield (%)		20
• Strain at Break (%)		625
Tear Resistance, lbs.	ASTM D 1004, Die C	24
Puncture Resistance, lbs.	FTMS 101 Method 2065	56
Low Temp. Impact, °F (max.)	ASTM D 746	-120
Dimensional Stability, % (max.)	ASTM D 1204 (NSF 54)	±1.0
Carbon Black Content, Allowable Range in percent	ASTM D 1603	2-3
Carbon Black Dispersion	ASTM D 5596	(2)

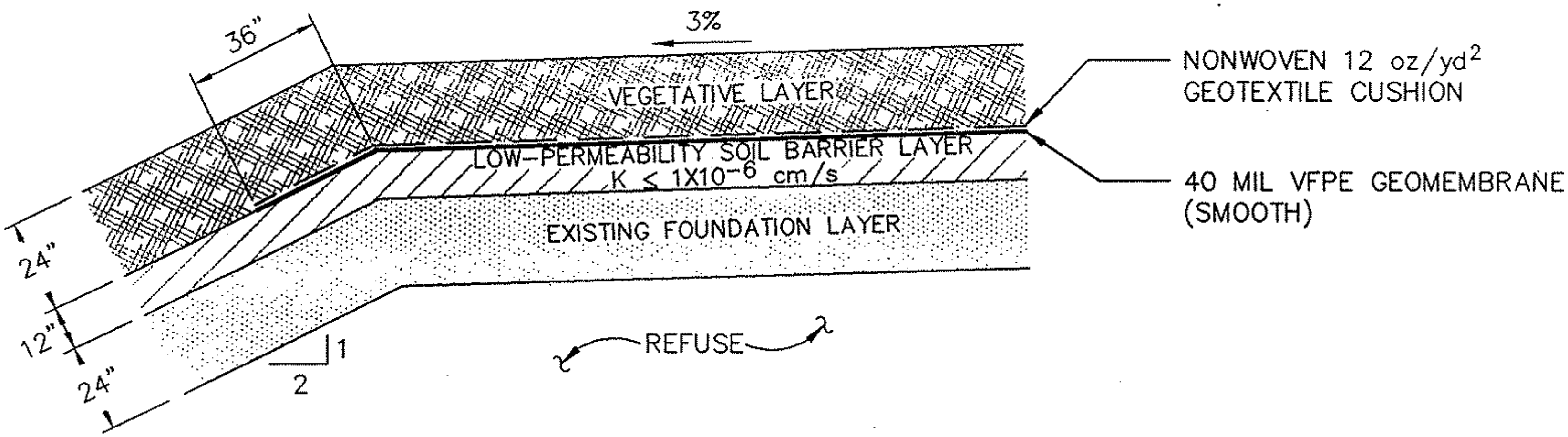
Notes: (1) Elongation at break shall be calculated using a gage length of 2.5 in.
(2) Minimum of 5 Category 1, Minimum of 8 Categories 1 and 2, and Minimum of 10 Categories 1,2,3.
(3) The yield stress and strain will be defined as the point on the stress-strain curve where the tangent modulus first reaches 290 psi.

TABLE 2-2

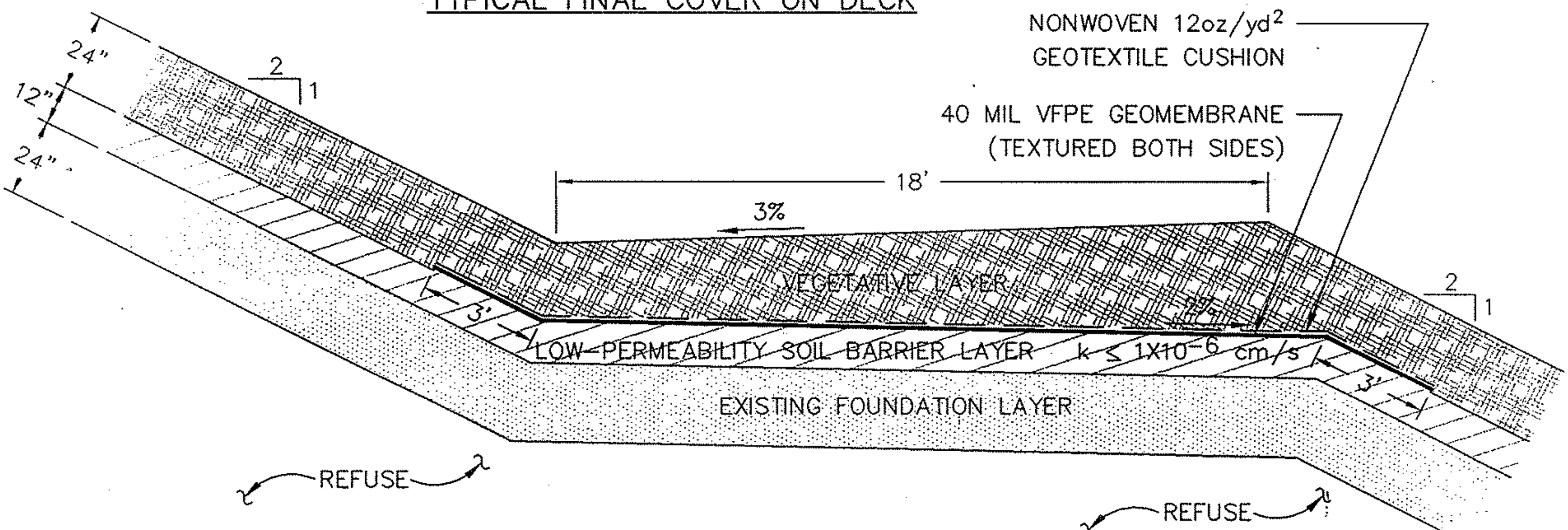
REQUIRED PHYSICAL PROPERTIES FOR GEOSYNTHETIC CLAY LINER FINAL CLOSURE PLAN LOPEZ CANYON SANITARY LANDFILL

PROPERTY	TEST METHOD	REQUIREMENTS
Dry Mass of Bentonite per Unit Area	ASTM D 3776	0.8 lb/ft^2
Puncture Strength, Unhydrated GCL	ASTM D 4833	100 lb
Bentonite Free Swell	USP NF XVII	25 ml
Hydraulic Conductivity[(1)]	ASTM D 5084	5×10^{-9} cm/s

Notes: [(1)]Performed under a confining pressure of 5 psi.



TYPICAL FINAL COVER ON DECK



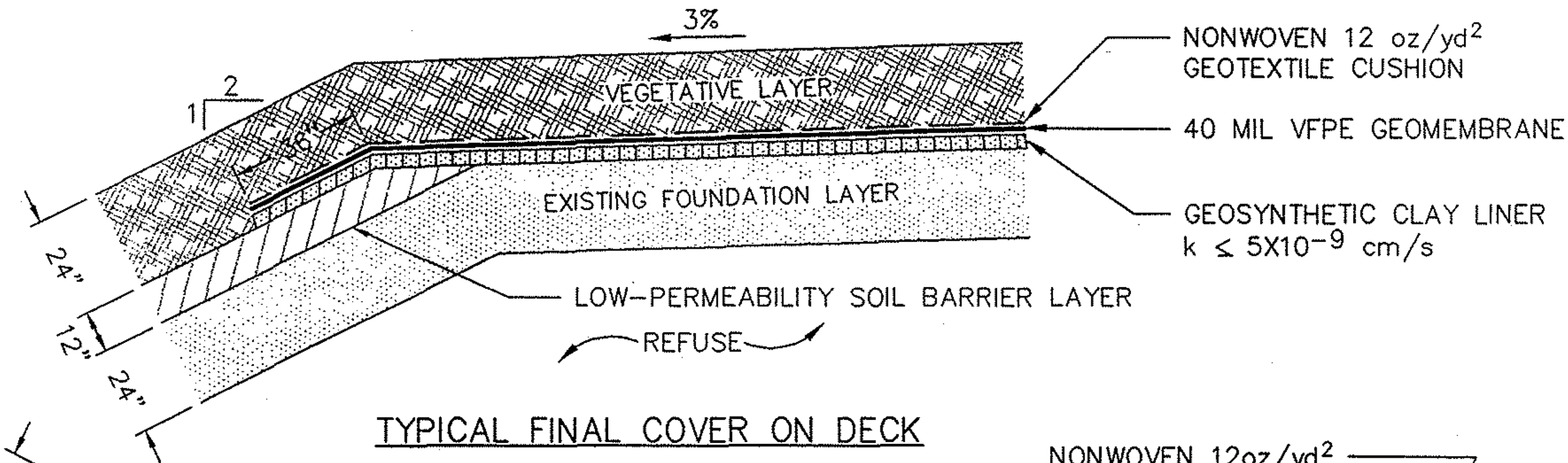
TYPICAL FINAL COVER ON BENCH

APPROXIMATE SCALE: 1" = 2'

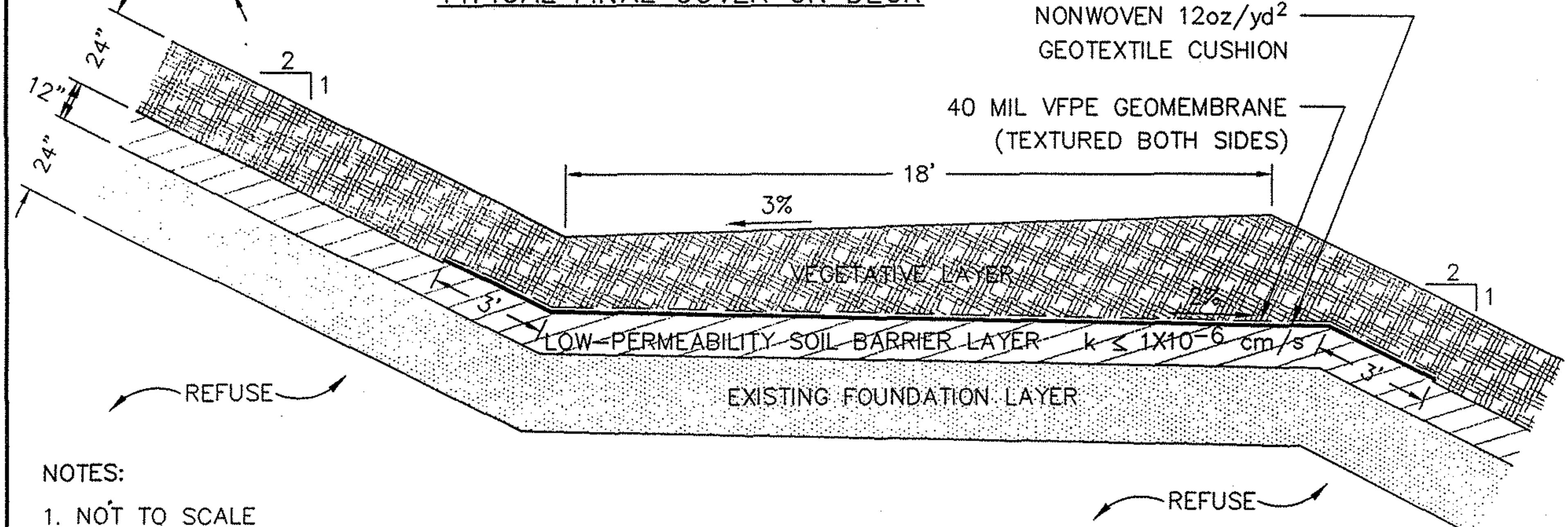
GEOSYNTEC CONSULTANTS

FINAL COVER ON DECK/BENCH AREAS DISPOSAL AREA C LOPEZ CANYON SANITARY LANDFILL	FIGURE NO.	2-1
	PROJECT NO.	CE4100-04
	DATE:	DEC-07-93

DWG: 4100F031 199611201815 AC



TYPICAL FINAL COVER ON DECK



NOTES:

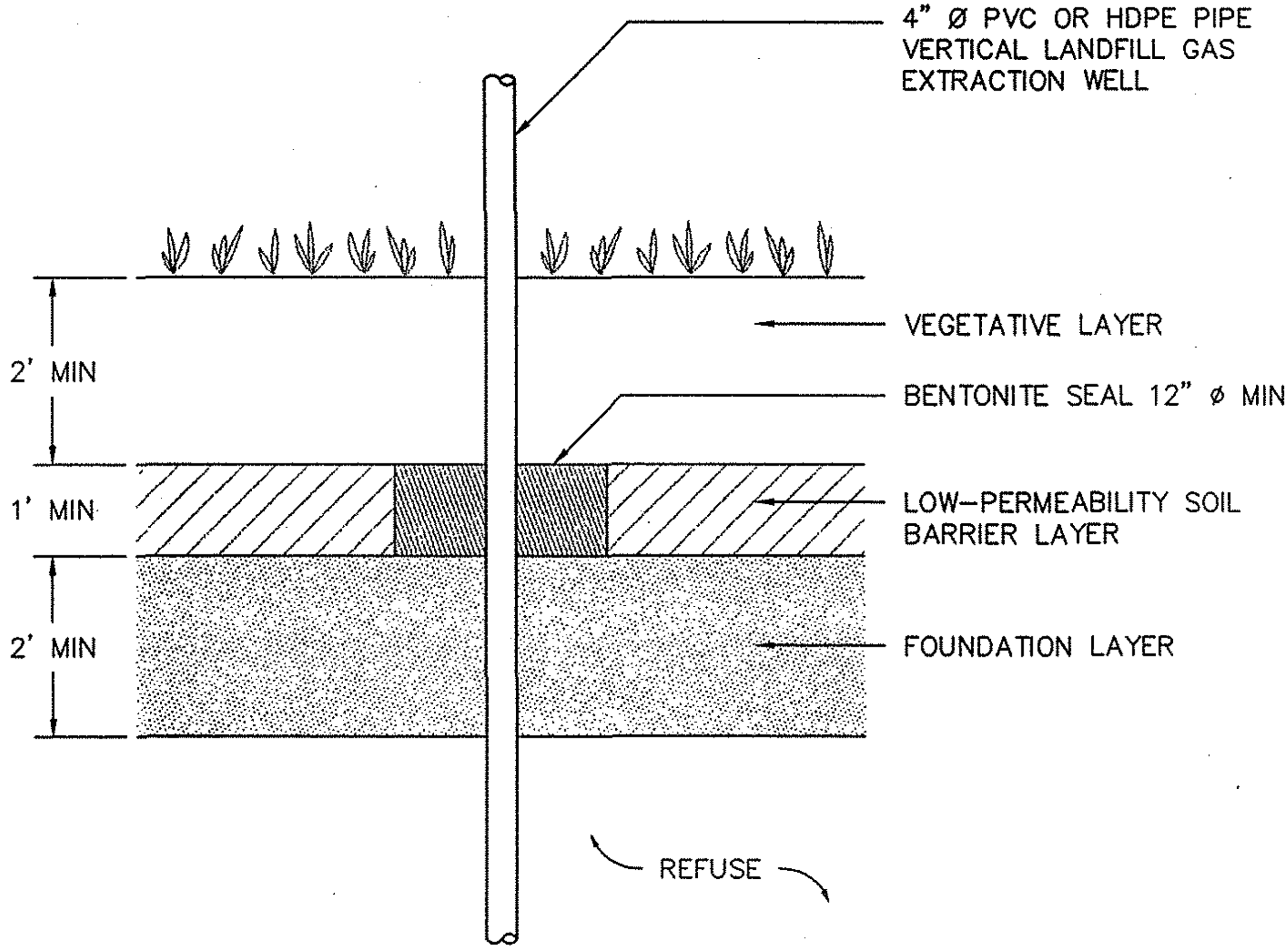
1. NOT TO SCALE
2. MODIFIED 09-04-96 TO USE GEOSYNTHETIC CLAY LINER ON DECK AREAS.

TYPICAL FINAL COVER ON BENCH

GEOSYNTEC CONSULTANTS

FINAL COVER ON DECK/BENCH AREAS DISPOSAL AREA C LOPEZ CANYON SANITARY LANDFILL	FIGURE NO.	2-1(a)
	PROJECT NO.	CE4100-06
	DATE:	SEP-04-96

DWG: 4100F604 199612161738 AC

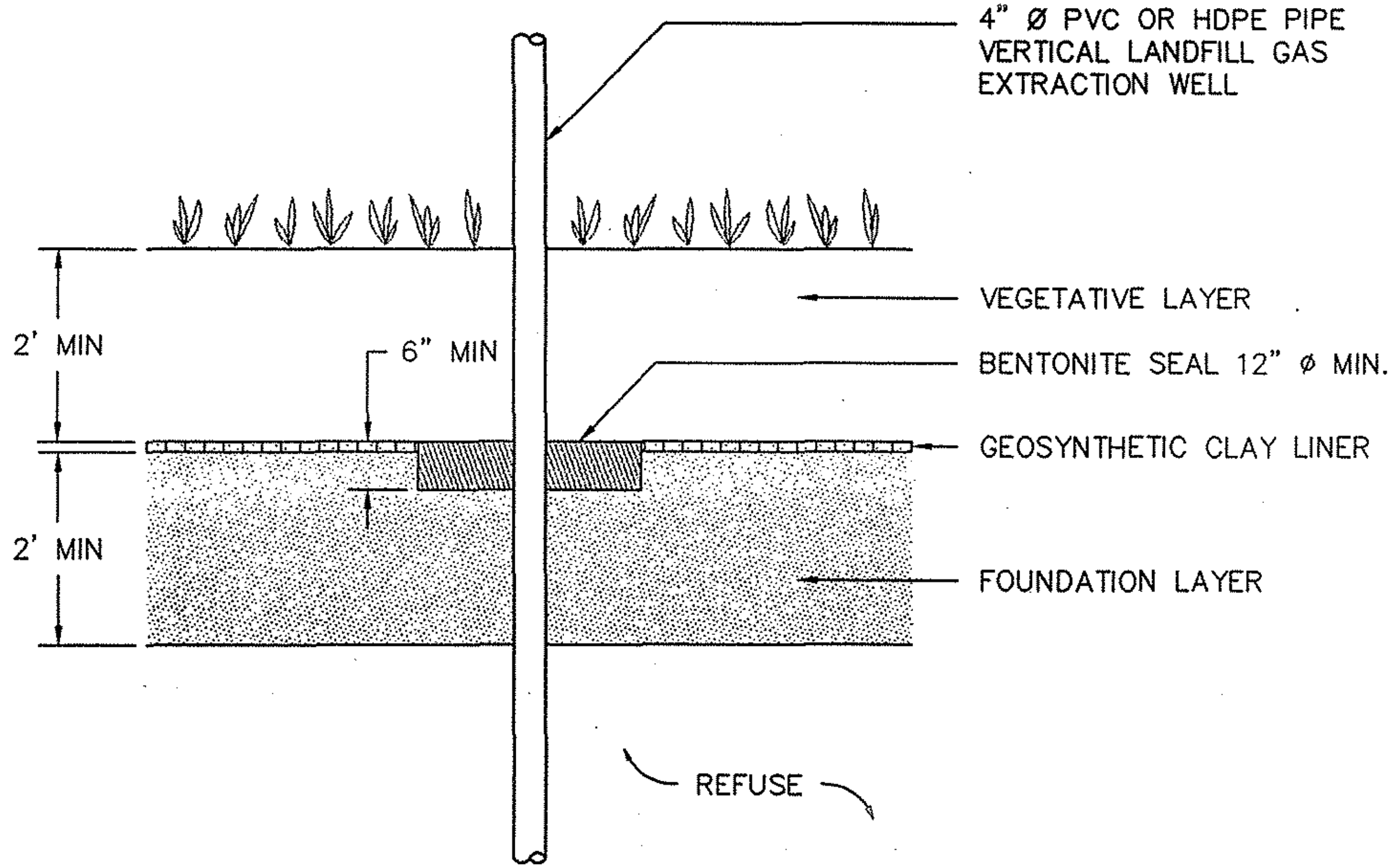


GEOSYNTEC CONSULTANTS

VERTICAL LANDFILL GAS EXTRACTION WELL PENETRATING
FINAL COVER SYSTEM ON DECK WITH LOW-PERMEABILITY
SOIL BARRIER LAYER
LOPEZ CANYON SANITARY LANDFILL

FIGURE NO.	2-2(c)
PROJECT NO.	CE4100-06
DATE:	SEPT-04-96

DWG: 4100F601 199609051152 LP

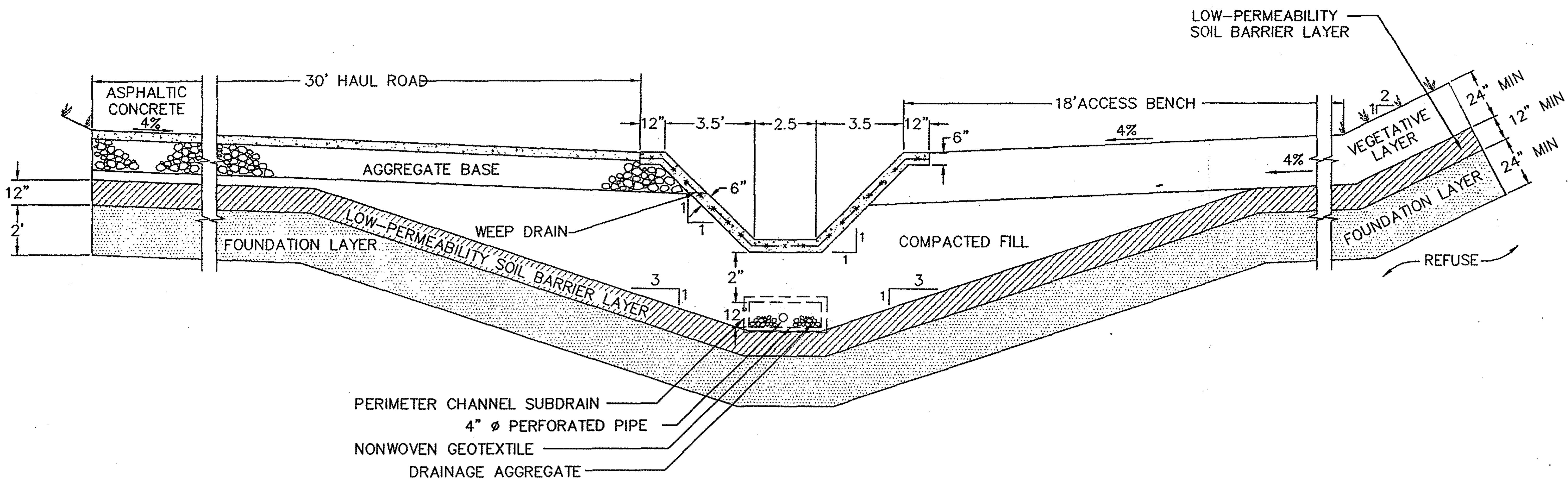


GEOSYNTEC CONSULTANTS

VERTICAL LANDFILL GAS EXTRACTION WELL PENETRATING
FINAL COVER SYSTEM ON DECK
WITH A GEOSYNTHETIC CLAY LINER
LOPEZ CANYON SANITARY LANDFILL

FIGURE NO.	2-2(d)
PROJECT NO.	CE4100-06
DATE:	SEPT-04-96

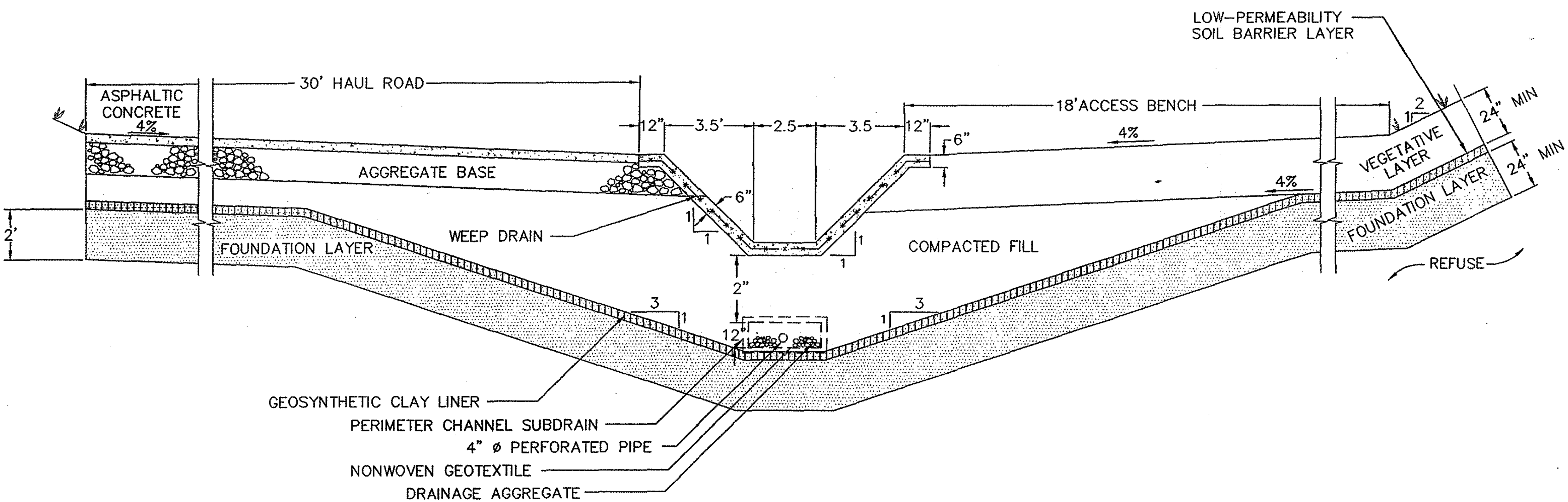
DWG: 4100F601 199609051154 LP



GEOSYNTEC CONSULTANTS

FINAL COVER TERMINATION AT HAUL ROAD IN DISPOSAL AREA AB+ LOPEZ CANYON SANITARY LANDFILL	FIGURE NO. 2-4
	PROJECT NO. CE4100-06
	DATE: SEPT-25-96

DWG: 4100F603 199609260922 SOS



GEOSYNTEC CONSULTANTS

FINAL COVER TERMINATION AT
HAUL ROAD IN DISPOSAL AREA AB+
LOPEZ CANYON SANITARY LANDFILL

FIGURE NO.	2-4(a)
PROJECT NO.	CE4100-06
DATE:	SEPT-25-96

DWG: 4100F603 199609260921 SOS

ATTACHMENT F

REVISION TO

APPENDIX H
OF
VOLUME IV OF IV REPLACEMENT
AMENDMENT TO FINAL CLOSURE PLAN

DISPOSAL AREA C
FINAL COVER PERFORMANCE EVALUATION
LOPEZ CANYON SANITARY LANDFILL

TABLE OF CONTENTS (continued)

TABLES

FIGURES

APPENDICES

the slope minimizes the potential for ponding and infiltration, the geomembrane is omitted from the infiltration barrier layer of the final cover. Infiltration analyses show that, due to the high percentage of surface water run-off from the final cover slopes and the arid climate at Lopez Canyon, this alternative final cover on the slopes of the Disposal Area C waste face will satisfy final cover performance standards, including the performance standard for surface water infiltration.

A final cover satisfying the prescriptive minimum standard will be used on deck and bench areas of Disposal Area C. The final cover cross-section proposed for the deck and bench areas is shown in Figure 3-1. This deck and bench area final cover consists of the following components, from top to bottom:

- 24-in. (600-mm) thick, minimum, vegetative layer (thickness varies from about 26 in. (650 mm) to 35 in. (875 mm) on bench areas);

- 12 oz/yd^2 (410 g/m^2) nonwoven geotextile cushion;

- 40-mil (1-mm) thick VLDPE geomembrane (both sides textured on bench areas); Appendix III provides an analysis of this geomembrane barrier;

- 12-in. (300-mm) thick compacted low-permeability soil barrier layer having a saturated hydraulic conductivity no greater than 1 x 10^{-6} cm/s; and

- 24-in. (600-mm) thick foundation layer (existing at the time of closure).

The alternative final cover cross-section proposed for the slopes of the Disposal Area C waste face is shown in Figure 3-2. It consists of the following components,

APPENDIX III

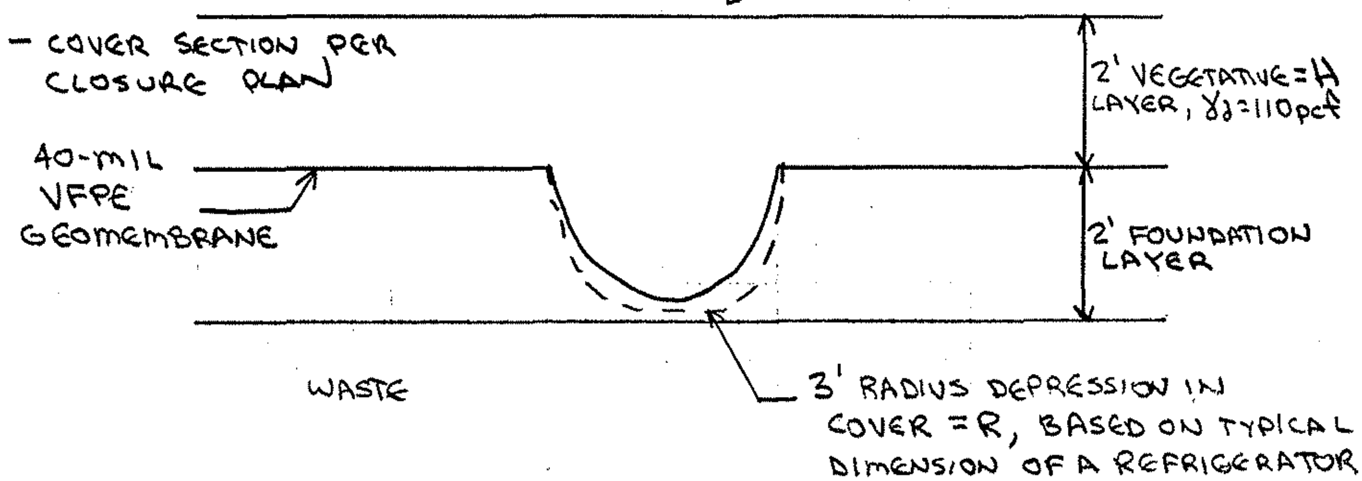
ANALYSIS OF VFPE GEOMEMBRANE

Written by: M. SNOW Date: 96/11/6 (YY MM DD) Reviewed by: SAMcKerway Date: 96/11/7 (YY MM DD)

Client: CITY OF L.A. Project: LOPEZ CANYON Project/Proposal No.: CE4100 Task No.: 04

ANALYSIS OF GEOMEMBRANE CAP
DISPOSAL AREA "C"
LOPEZ CANYON LANDFILL

SURFACE LOAD = q = 0



— PROPERTIES SPECIFIED FOR 40-MIL VFPE

1) GRAB TEST YIELD STRAIN = 20% (MIN.) (ASTM D 638)
   ↳ PER SPECIFICATION

BASED ON DATA PRESENTED TABLE 5.3 OF "DESIGNING WITH GEOSYNTHETICS" BY ROBERT M. KOERNER (SEE ATTACHED), THE ACTUAL MULTIAXIAL STRAIN OF A POLYETHYLENE GEOMEMBRANE IS 4.3 TIMES GREATER. CONSERVATIVELY ASSUME:

MULTIAXIAL YIELD STRAIN = 2.5 × 20% = 50%

2) GRAB TEST YIELD STRESS = 50 lb./in. (MIN) (ASTM D 638)
   ↳ PER SPECIFICATION

BASED ON TABLE 5.3, THE ACTUAL MULTIAXIAL STRESS OF A PE GEOMEMBRANE IS 1.4 TIMES LESS. CONSERVATIVELY ASSUME

MULTIAXIAL YIELD STRESS = $\frac{50 \text{ lb/in}}{1.5}$ = 33.3 lb/in.

Written by: M. SNOW Date: 96 / 11 / 6 Reviewed by: ______ Date: __/__/__

Client: CITY OF LA Project: LOPEZ CANYON Project/Proposal No.: CE 4100 Task No.: 04

PROBLEM ANALYSIS BASED ON "DESIGN OF SOIL LAYER-GEOSYNTHETIC SYSTEMS OVERLYING VOIDS" BY GIROUD ET AL (SEE ATTACHED)

$$1+\varepsilon = 2\Omega \sin^{-1}\left[\frac{1}{2\Omega}\right]$$

(eq. #11 GIROUD PAPER) for $y/b \le 0.5$

ε = strain = 50%

$$\Omega = 0.5$$

Ω = shape factor (see GIROUD PAPER, TABLE 2)

$$\alpha = \Omega\left(2\gamma R^2\left(1-e^{-0.5H/R}\right)\right)$$

(eq. #16 GIROUD PAPER) $q = 0$

H = cover soil thickness R = void radius
γ = density of cover soil
α = tensile strain in geomembrane

$$\alpha = 0.5\left(2\times 130\text{ pcf}\times(3\text{ ft})^2\left(1-e^{-0.5\frac{(2\text{ ft})}{(3\text{ ft})}}\right)\right)$$

$$\alpha = 0.5\left(2{,}340\ \frac{\text{lb}}{\text{ft}}\ (1-0.7165)\right)$$

$$\alpha = 332\ \text{lb/ft} = 27.7\ \text{lb/in.}$$

FACTOR OF SAFETY $= \frac{33.3}{27.7} = 1.2$ OK

This factor of safety is acceptable given:

- the low likelihood of having a 6 ft diameter void at a modern MSW site
- the ability to repair the cover geomembrane if there is damage

TABLE 5.3 TENSILE BEHAVIOR PROPERTIES OF 30-MIL PVC, 36-MIL CSPE, AND 30-MIL HDPE

Property		Dumbbell shape (Fig. 5.2)			Narrow-width (1.0-in. [25 mm]) shape (Fig. 5.3)			Wide-width (8.0-in. [100-mm]) shape (Fig. 5.3)			Three-dimensional shape (Fig. 5.5)		
		PVC	CSPE-R	HDPE	PVC	CSPE-R	HDPE	PVC	CSPE-R	HDPE	PVC	CSPE-R	HDPE
maximum stress*	(lb./in.^{2})	3400	5700	3200	2900	5100	3000	2800	4300	2800	1200	3300	2300
	(megapascals)	23	39	22	20	35	21	19	30	19	8.3	23	16
maximum strain*	(%)	300	17	11	300	35	13	300	30	15	120+	100	47
modulus	(lb./in.^{2})	9000	33,000	94,000	9000	15,000	40,000	9000	14,000	33,000	4000	5000	25,000
	(megapascals)	62	227	648	62	103	275	62	96	227	28	34	172
ultimate stress	(lb./in.^{2})	3400	1300	≅4000	2700	1200	≅3500	2800	1100	≅3000	d.n.f.	3300	2300
	(megapascals)	23	9.0	28	19	8.3	24	19	7.6	21		23	16
ultimate strain	(%)	300	100	≅700	300	58	≅600	300	51	≅500	d.n.f.	100	47

*Notes:
PVC values are at ultimate
CSPE-R values are at scrim break
HDPE values are at yield
d.n.f. = did not fail

$\dfrac{\text{DUMBBELL (i.e., GRAB)}}{\text{3-D (i.e., MULTIAXIAL)}}$ RATIO.

FOR HDPE STRESS YIELD, RATIO = $\dfrac{3200}{2,300}$ = 1.4 times less

FOR HDPE STRAIN YIELD, RATIO = $\dfrac{11}{47}$ = 0.23, OR 4.3 times GREATER

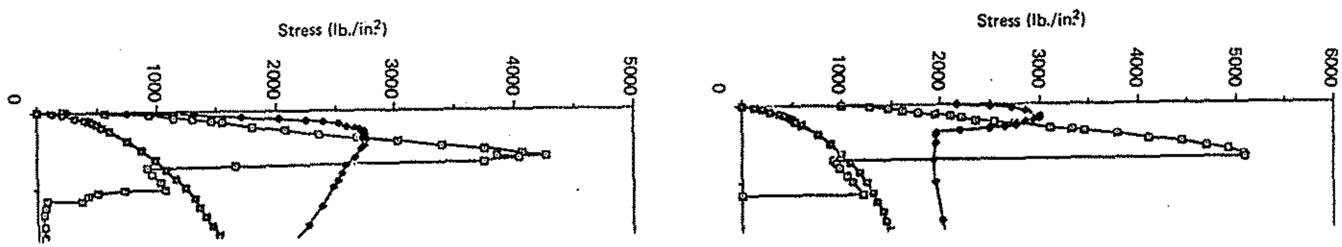


Figure 5.3 Tensile
8.0-in. (200-mm) wi

Geotextiles and Geomembranes 9 (1990) 11–50

Design of Soil Layer-Geosynthetic Systems Overlying Voids

J. P. Giroud, R. Bonaparte, J. F. Beech & B. A. Gross

GeoServices Inc. Consulting Engineers,
1200 South Federal Highway, Suite 204
Boynton Beach, Florida 33435, USA

ABSTRACT

This paper presents equations, tables, and charts to design soil layer-geosynthetic systems to span voids such as tension cracks, sinkholes, dissolution cavities, and depressions in foundation soils due to differential settlements or localized subsidence. These equations, tables, and charts were developed by combining tensioned membrane theory (for the geosynthetic) with arching theory (for the soil layer), thereby providing a more complete design approach than one that considers tensioned membrane theory only.

Design examples are presented to illustrate the solution of typical problems such as: selection of the required geosynthetic properties, determination of the maximum void size that can be bridged by a given system, and evaluation of the load-bearing capacity of a given system.

NOTATION

b	Width of the infinitely long void (m)
c	Cohesion of the soil (N/m^2)
D	Depth of the void (m)
H	Thickness of the soil layer (m)
K	Coefficient of lateral earth pressure (dimensionless)
K_a	Coefficient of active earth pressure (dimensionless)
p	Pressure on the geosynthetic (i.e. vertical stress at the bottom of the soil layer) over the void area (N/m^2)

Geotextiles and Geomembranes 0266-1144/90/$03.50

p_{lim} Limit value for the pressure on the geosynthetic, over the void area (N/m^2)
p_b Pressure transmitted to the bottom of the void (N/m^2)
p_0 Pressure on the geosynthetic over the void area neglecting soil arching (N/m^2)
q Uniformly distributed normal stress applied on top of the soil layer (N/m^2)
r Radius of the circular void (m)
r_{max} Maximum radius of a circular void which can be bridged by a given geosynthetic (m)
s Soil shear strength (N/m^2)
y Geosynthetic deflection (m)
z Depth measured from the top of the soil layer (m)

α Geosynthetic tension (force per unit width) corresponding to the geosynthetic strain ε (N/m)
α_{lim} Limit value for the required geosynthetic tension (N/m)
ε Geosynthetic strain (dimensionless)
γ Unit weight of soil (N/m^3)
Ω Factor related to y and ε (dimensionless)
ϕ Friction angle of the soil (degrees and dimensionless)
σ_H Horizontal stress at depth z (N/m^2)
σ_V Vertical stress at depth z (N/m^2)

INTRODUCTION

Description of the Problem

In many practical situations, a load is applied on a soil layer-geosynthetic system that will eventually overlie a void. (In this paper, the term 'void' is used generically for cracks, cavities, depressions, etc.) Two typical examples are a road embankment or a lining system for a reservoir constructed on a foundation where localized subsidence may develop.

The design engineer has to verify that, should subsidence develop, the geosynthetic layer can support the loads applied by the overlying soil and any other source (such as traffic on the road or the liquid in the reservoir) without failing or undergoing excessive deflection. The soil-geosynthetic system deflects over the void, and, from a design standpoint, three possibilities must be considered:

- The geosynthetic fails (Fig. 1(a)).
- The geosynthetic undergoes limited deflection and bridges the void (Fig. 1(b)).
- The geosynthetic deflects until it comes in contact with the bottom of the void (Fig. 1(c)).

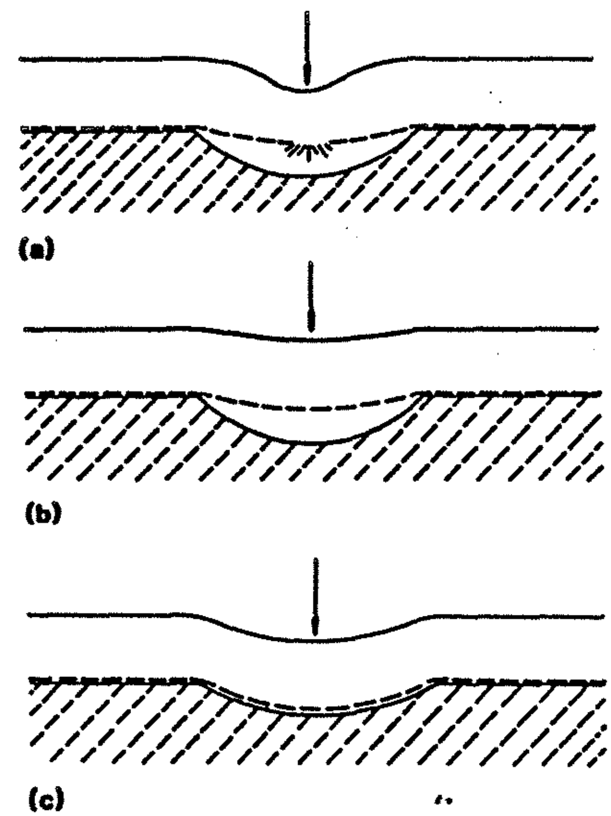


Fig. 1. Three design situations: (a) the geosynthetic fails; (b) the geosynthetic undergoes limited deflection and bridges the void; and (c) the geosynthetic deflects until it comes in contact with the bottom of the void.

The Nature of Voids

Examples of voids that can develop under a geosynthetic are discussed below:

Tension Cracks
Such cracks can occur in non-saturated cohesive soils subjected to tensile stresses and/or differential movements caused by settlement or other

Fig. 2. Large tension crack formed under a geomembrane liner.

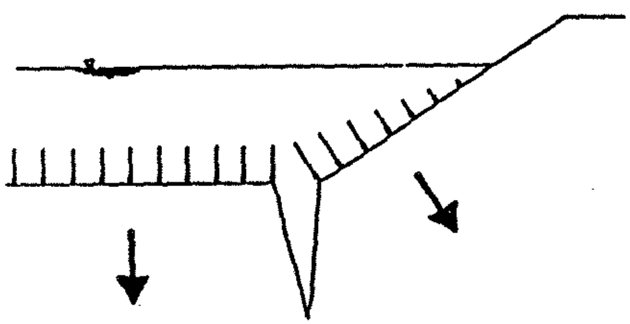


Fig. 3. Mechanism of tension crack formation at the toe of the side slope of a reservoir (not to scale). (After Loudière and Perrin.[1])

mechanisms. A case has been reported[1] where very large cracks (0·1–0·3 m wide) developed in the cohesive soil located under the geomembrane liner of a reservoir (Fig. 2). The cracks occurred near the toe of the side slopes of the reservoir. In this area, tensile stresses and differential movements resulted from the different water pressure orientations on the bottom and on the slopes, as shown in Fig. 3.

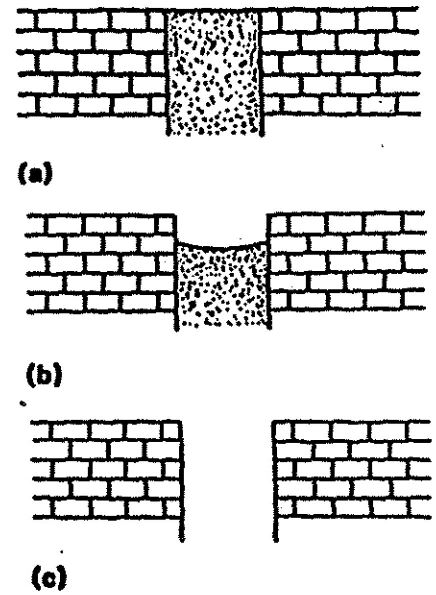


Fig. 4. Sinkhole in a karstic limestone mass: (a) before collapse; (b) after partial collapse and (c) after complete collapse.

Fissures and Cracks in Bedrock

Soil layers or masses are sometimes constructed on a bedrock with fissures or cracks. A rare but important case is the construction of the clay core of a dam on a bedrock where cracks may develop. Some dam failures have resulted from this situation.

Sinkholes due to Karstic Collapse

Karstic limestone masses contain pockets or chimneys filled with soil. Water or other liquids seeping through a karstic limestone mass may remove soil from these pockets or chimneys, thereby creating a void which can be on the order of one to several meters in diameter (Fig. 4). These voids are usually referred to as sinkholes. The bursting of a geomembrane liner installed on a mass of karstic limestone which subsequently collapsed has been described by Giroud and Goldstein[2] and Giroud.[3] Karstic collapses can occur under other types of structures, such as road embankments, as discussed by Bonaparte and Berg.[4]

Soil Dissolution

Dissolution cavities can be caused by water in soils containing gypsum or by acid in soils containing calcium carbonate. The senior author has

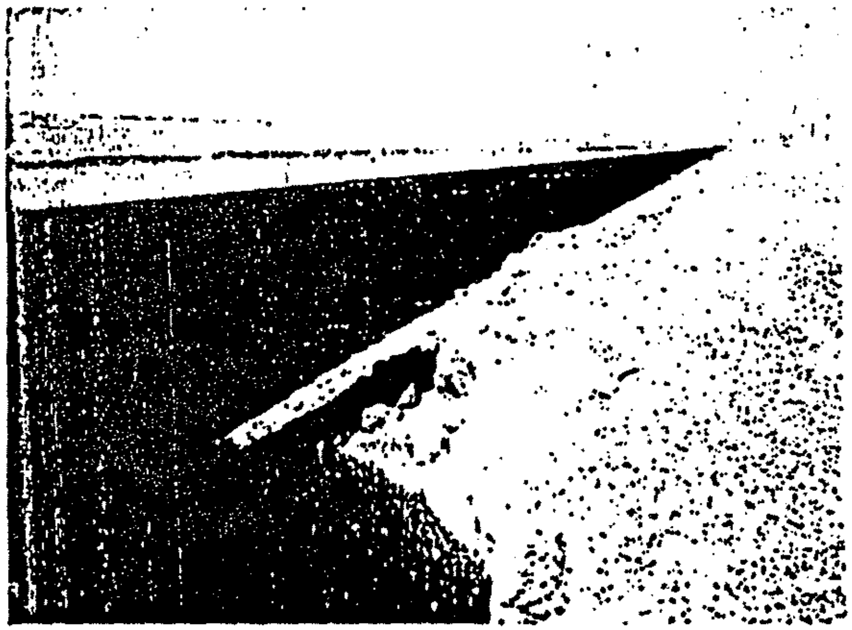


Fig. 5. Dissolution Cavity. This cavity in high gypsum content soil was caused by water leaking through a concrete canal liner.

observed cavities about one meter deep and one meter wide caused by: (i) water leaking through the concrete liner of canals constructed in soils with a high gypsum content (Fig. 5); and (ii) phosphoric acid leaking through a faulty seam of the geomembrane liner of a reservoir constructed on a high calcium-carbonate content soil (Fig. 6).[5]

Differential Settlement

Depressions in the ground surface may be formed when a localized area settles more than the rest ('differential settlement'). There are many situations where depressions result from differential settlement. These include depressions resulting from: (i) differential settlement of municipal solid waste (resulting from the heterogeneity of the waste) affecting a geosynthetic-soil cover system placed on the waste; (ii) settlement of a

Fig. 6. Dissolution Cavity. This cavity in high calcium-carbonate content soil was caused by phosphoric acid leaking through a geomembrane liner.

localized lens of compressible soil; (iii) thawing of subsurface ice lenses; and (iv) settlement of a poorly compacted trench backfill. Tisserand[6] has reported a case of geomembrane failure over the depression resulting from trench backfill settlement. Differential settlement due to lenses of compressible soils frequently occur under road embankments.

Localized Subsidence

The surface of the ground may be locally depressed as a result of the collapse of underground cavities such as: natural caves, tunnels, mine workings, pipes, and tanks. Localized subsidence may also occur at the surface of municipal solid waste as a result of the collapse of deteriorating structures such as refrigerators.

Classification of Voids

Two shapes of voids are considered in the study presented in this paper: infinitely long voids with a width b and circular voids with a diameter $2r$. The voids presented above can therefore be put into two categories:

- Cracks and depressions resulting from trench backfill settlement may be modeled approximately as an infinitely long void.
- Karstic sinkholes, dissolution cavities, municipal solid waste settlement, lens settlement, soil surface depressions and ground subsidence may be modeled approximately as a circular void.

In the case of cracks and complete karstic collapse (Fig. 4(c)), the geosynthetic deflects without reaching the bottom of the void. With the other types of voids, the geosynthetic may or may not reach the bottom of the void, depending on the geometry of the void, the modulus of the geosynthetic and the applied loads.

Load-Carrying Mechanism

The soil layer and underlying geosynthetic are assumed initially to be resting on a firm foundation. At some point in time, a void of a certain size opens below the geosynthetic. Under the weight of the soil layer and any applied loads, the geosynthetic deflects. The deflection has two effects; *bending* of the soil layer and *stretching* of the geosynthetic.

The *bending* of the soil layer generates arching inside the soil, which transfers part of the applied load away from the void area, as shown in Fig. 7. As a result, the vertical stress, σ_V, over the void area is smaller than the average vertical stress, $\gamma H + q$, due to the weight of a soil layer of thickness H and an applied uniform normal stress of magnitude q.

The *stretching* of the geosynthetic mobilizes a portion of the geosynthetic's strength. Consequently, the geosynthetic acts as a 'tensioned membrane' and can carry a load applied normally to its surface. As a result of geosynthetic stretching, two cases can be considered:

- In the first case, the stretched geosynthetic comes in contact with the bottom of the void. The mobilized portion of the geosynthetic strength carries a portion of the load applied normal to the surface of the geosynthetic. The rest of the load is transmitted to the bottom of the void.
- In the second case, the geosynthetic does not deflect enough to come in contact with the bottom of the void. In this case, either the geosynthetic is strong enough to support the entire load applied normal to its surface or it fails.

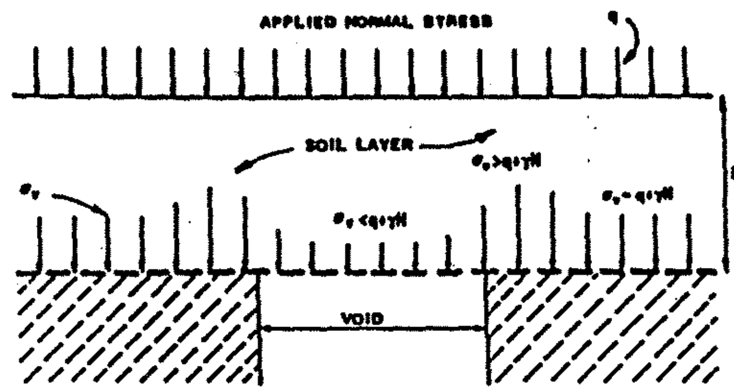


Fig. 7. Effect of soil arching on load distribution.

In summary, the soil-geosynthetic system deflects and the geosynthetic stretches until it fails (Fig. 1(a)) or until an equilibrium condition is reached (Fig. 1(b) or 1(c)).

Scope of this Paper

This paper presents the development and use of equations, tables, and charts for the case of a soil layer subjected to a uniformly distributed normal load and resting on a geosynthetic overlying a rigid foundation containing a single infinitely long void (plane-strain problem) or circular void (axisymmetric problem). The parameters considered in this paper are:

- Geometric Parameters: These include the thickness of the soil layer and the geometry of the void (width of an infinitely long void or diameter of a circular void, and depth of void) (Fig. 8).
- Mechanical Parameters: These include the soil mechanical properties and the geosynthetic tensile behavior (expressed by its tension–strain curve).
- Loading Conditions: These include the unit weight of the soil layer and the load exerted on the top of the soil layer, which is assumed to be normal and uniformly distributed.

The equations, tables, and charts make it possible to solve design problems such as:

- select the required geosynthetic mechanical properties when the geometric parameters and the loading conditions are known;

- determine the required thickness of the soil layer associated with a given geosynthetic over a given void and subjected to given loading conditions;
- determine the void size that a given geosynthetic may bridge when it is associated with a given soil layer subjected to given loading conditions; and
- determine the maximum load which can be carried by a given soil-geosynthetic system over a given void.

The solution of any of the above design problems depends on the allowable geosynthetic strain.

Originality of this Paper

The use of tensioned membrane theory to evaluate the load-carrying capacity of a geosynthetic bridging a void was presented by Giroud.[7] Subsequently, Giroud[8] developed a design chart based on tensioned membrane theory. This chart has often been used to evaluate the load-carrying capacity of a soil layer associated with a geosynthetic. By doing so, the internal shear strength of the soil layer is neglected, and this can be very conservative. Therefore, Bonaparte and Berg[4] combined arching theory (for the soil layer) with tensioned membrane theory (for the geosynthetic) to formulate a more complete design approach.

This paper significantly extends the earlier work of Giroud[7,8] and Bonaparte and Berg[4] and provides an extensive analysis of soil-geosynthetic system bridging a void.

ANALYSIS

Assumptions

The void can be either circular (diameter $2r$) or infinitely long (width b). Regarding the bottom of the void, two cases can be considered: (i) a bottomless void (Fig. 8(a)); and (ii) a bottom with a maximum depth D and a spherical shape (for the circular void) or a cylindrical shape with a circular cross section (for the infinite void) (Fig. 8(b)). From a design standpoint, both cases are identical if the deflection y of the geosynthetic is less than the depth of D of the void.

The soil layer is assumed to be horizontal and to have a uniform thickness H. The stress q applied on the soil layer is assumed to be normal and uniformly distributed.

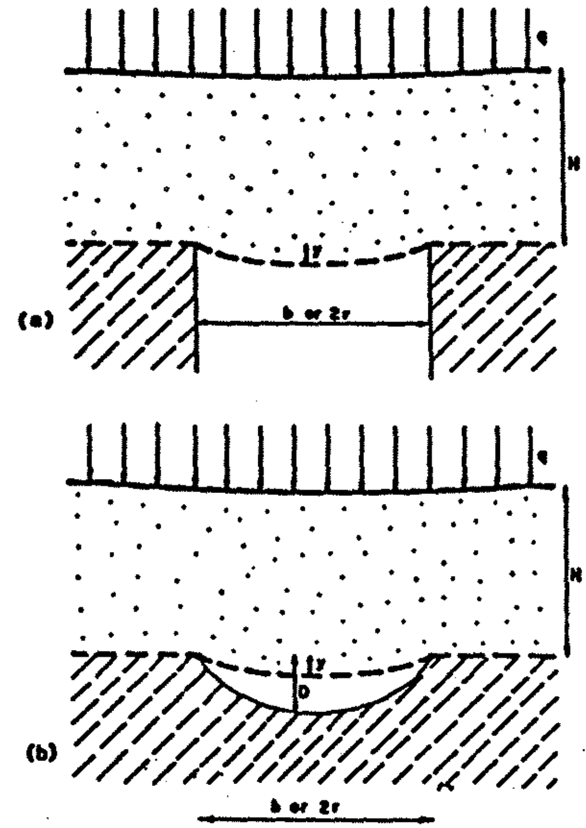


Fig. 8. Schematic cross section for theoretical analysis. Two cases can be considered: (a) the void is bottomless; and (b) the bottom of the void is assumed to have a circular cross section and the depth of the void is D. The void located under the geosynthetic is either infinitely long (with a width b), or circular (with a diameter $2r$); y is the geosynthetic deflection.

Relevant geosynthetic properties are the tension–strain curve or, at least, the tension α corresponding to the design strain ε.

Relevant soil properties are the friction angle ϕ and the cohesion c. For the analysis presented in this paper, the cohesion is neglected. In other words, the charts are established for $c = 0$ and can be conservatively used for $c > 0$. Also, it will be shown that the friction angle ϕ does not have a significant influence on the analysis results if it is equal to or greater than 20°.

Approach

The problem under consideration involves a complex soil-geosynthetic interaction. The problem can be greatly simplified, however, if the soil response (arching) is uncoupled from the geosynthetic response (tensioned membrane). Therefore, a two-step approach is used. First, the behavior of the soil layer is analyzed using classical *arching theory*. This step gives the pressure at the base of the soil layer on the portion of the geosynthetic located above the void. Second, *tensioned membrane theory* is used to establish a relationship between the pressure on the geosynthetic, the tension and strain in the geosynthetic, and the deflection of the geosynthetic. Accordingly, the following sections deal with arching theory, tensioned membrane theory, and the combination of both.

An inherent assumption in this uncoupled two-step approach is that the soil deformation required to generate the soil arch is compatible with the tensile strain required to mobilize the geosynthetic tension. This assumption has not been verified.

Arching Theory (see Fig. 9)

When the geosynthetic deflects, arching develops in the soil layer. As a result, a portion of the applied stress is transmitted laterally and, consequently, the normal stress transmitted to the portion of the geosynthetic located above the void is smaller than the average vertical stress due to the weight of the soil layer and the uniformly distributed normal stress applied on top of the soil layer (Fig. 7). The procedures for calculating the reduced

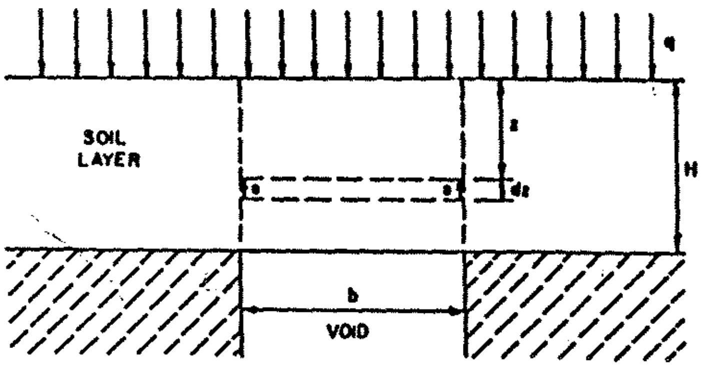


Fig. 9. Derivation of arching equation.

stress transmitted to the portion of the geosynthetic located above the void are presented below for an infinitely long void and a circular void.

Infinitely Long Void

Terzaghi[9] has established equations for soil arching over an infinitely long void assuming that the lateral load transfer is achieved through shear stresses along vertical planes located at the edges of the void (Fig. 9). As a result of this assumption, the incremental change in vertical stress, $d\sigma_V$, due to an incremental change in depth, dz, is given by

$$d\sigma_V = [\gamma - 2(s/b)]dz \tag{1}$$

where: b = width of the infinitely long void; σ_V = vertical stress at depth z; γ = unit weight of soil; z = depth measured from the top of the soil layer; and s = soil shear strength. Basic SI units are: b(m), σ_V (N/m^2), γ (N/m^3), z (m), and s (N/m^2).

The soil shear strength along a vertical plane is expressed by

$$s = c + \sigma_H \tan\phi \tag{2}$$

where: c = cohesion of the soil; σ_H = horizontal stress at depth z; and ϕ = friction angle of the soil. Basic SI units are: s (N/m^2), c (N/m^2), σ_H (N/m^2), and ϕ (degrees); ϕ is dimensionless.

The relationship between the horizontal stress and the vertical stress is given by the following classical relationship

$$\sigma_H = K\sigma_V \tag{3}$$

where: K = coefficient of lateral earth pressure (dimensionless).

It should be noted that many of the relationships presented in this paper are valid for both effective and total stress conditions; however, eqn (3) is valid only for effective stress conditions.

Combining eqns (1), (2) and (3) and solving the differential equation for the boundary condition $\sigma_V = q$ for $z = 0$ gives

$$\sigma_V = \frac{b(\gamma - 2c/b)}{2K\tan\phi}\left[1 - e^{-K\tan\phi(2z/b)}\right] + q\,e^{-K\tan\phi(2z/b)} \tag{4}$$

where: q = uniformly distributed normal stress applied on the top of the soil layer (basic SI unit: N/m^2); all other notations as defined above and in the Notations section.

The pressure on top of the geosynthetic, over the void area, p, is the value of σ_v for $z = H$ in eqn (4). If the soil cohesion, c, is assumed to equal zero, the value of p is

$$p = \frac{\gamma b}{2K \tan\phi}\left[1 - e^{-2K \tan\phi\, H/b}\right] + q\, e^{-2K \tan\phi\, H/b} \tag{5}$$

where: p = pressure on top of the geosynthetic (i.e. vertical stress at the bottom of the soil layer), over the void area (basic SI unit: N/m^2); and other notations as defined above and in the Notations section.

Circular Void

Using the same approach, Kezdi[10] has established that eqn (5) can be used for a circular void if b is replaced by r (and not by $2r$), which shows that arching is twice as significant for a circular void compared to an infinitely long void.

Practical Approximate Equations

Selection of the value of the coefficient of lateral earth pressure is not easy since the state of stress of the soil in the zone where arching develops is not fully understood. Handy[11] has made a thorough analysis of soil arching and proposed the following value

$$K = 1{\cdot}06(\cos^2\theta + K_a \sin^2\theta) \tag{6}$$

with

$$\theta = 45° + \phi/2 \tag{7}$$

and

$$K_a = \tan^2(45° - \phi/2) \tag{8}$$

where: K_a = coefficient of active earth pressure (dimensionless); and other notations as defined above and in the Notations section.

Another approach would consist of using the coefficient of earth pressure at rest, expressed as follows, according to Jaky[12]

$$K = 1 - \sin\phi \tag{9}$$

In eqn (5), K is multiplied by $\tan\phi$. Values of $K\tan\phi$, calculated using eqns (6) and (9), are given in Table 1. It appears that $K\tan\phi$ does not vary significantly with ϕ, if ϕ is equal to or greater than 20°, which is the case for virtually all granular soils and for many fine-grained soils under drained conditions. Therefore, a constant value of 0·25 can be used for $K\tan\phi$ when ϕ is equal to or greater than 20°. As a result, eqn (5) becomes

$$p = 2\gamma b(1 - e^{-0{\cdot}5H/b}) + q\, e^{-0{\cdot}5H/b} \tag{10}$$

TABLE 1
Values of $K\tan\phi$

Soil friction angle (ϕ, degrees)	*Values of $K\tan\phi$* — *Using K from Handy (eqn (6))*	*Values of $K\tan\phi$* — *Using K from Jaky (eqn (9))*
0	0	0
5	0·08	0·08
10	0·15	0·15
15	0·21	0·20
20	0·25	0·24
25	0·29	0·27
30	0·31	0·29
35	0·32	0·30
40	0·32	0·30
45	0·31	0·29
50	0·30	0·28
55	0·27	0·26

Two values of K, the coefficient of lateral earth pressure, are considered: the value proposed by Handy[11] for arching and the value proposed by Jaky[12] for the 'at rest' state of stress.

Like eqn (5), eqn (10) is also valid for the circular void if b is replaced by r.

Equation 10 was used to establish Tables 3 and 4, and the charts given in Figs 11 and 14.

Comment on the Validity of Arching Theory

The analysis presented above is the classical analysis by Terzaghi.[9] This analysis does not consider soil dilatancy, which can increase the horizontal stress in the soil, thereby increasing the ability of the soil to arch. Therefore, the analysis presented in this paper can be considered conservative from this viewpoint. On the other hand, the analysis may not be conservative for loose soils that tend to contract when sheared.

Tensioned Membrane Theory

The tensioned membrane theory has been used by Giroud[3,7] to deal with the case of a geosynthetic overlying a void and subjected to a uniformly distributed stress normal to its surface.

The equations given below have been established with the following assumptions: (i) the strain in the portion of the [illegible] void (i.e. the deflected portion of the geosynthetic) is uniformly distributed; and (ii) the strain in the portion of the geosynthetic outside the void area is zero and, therefore, that portion of the geosynthetic does not move (i.e. the geosynthetic does not slide toward the void). These two assumptions greatly simplify the analysis, but no attempt has been made to evaluate their range of validity.

Infinitely Long Void

In the case of an infinitely long void, the deflected shape of the geosynthetic across the width of the void is cylindrical with a circular cross section, the strain is uniform, and the following relationships exist

$$1 + \varepsilon = 2\Omega \sin^{-1}[1/(2\Omega)] \qquad (\text{valid if } y/b \leq 0{\cdot}5) \qquad (11)$$

$$1 + \varepsilon = 2\Omega\{\pi - \sin^{-1}[1/(2\Omega)]\} \qquad (\text{valid if } y/b \geq 0{\cdot}5) \qquad (12)$$

where: ε = geosynthetic strain; y = geosynthetic deflection; b = width of the infinitely long void; and Ω = dimensionless factor. Basic SI units are: y(m) and b(m); ε and Ω are dimensionless.

The dimensionless factor Ω is defined by

$$\Omega = (1/4)[2y/b + b/(2y)] \qquad (13)$$

As a result of eqns (11), (12) and (13), there is a unique relationship between y/b, ε and Ω, which is given in Table 2 and shown in Fig. 10.

It is interesting to note that as ε tends towards zero eqn (11) tends toward

$$\Omega = 1/\sqrt{24\varepsilon} \qquad (14)$$

This equation gives a good approximation of Ω when ε is less than 1% (see Fig. 10).

Giroud[3,7] has also shown that the tension in the geosynthetic, in the case of an infinitely long void, is given by

$$\alpha = pb\Omega \qquad (15)$$

where: α = geosynthetic tension; p = pressure on the geosynthetic over the void area (i.e. vertical stress at the bottom of the soil layer over the void area); b = width of the infintely long void; Ω = dimensionless factor

TABLE 2

Values of Ω as a Function of Deflection or Strain

y/b or $y/(2r)$	ε(%)	Ω	y/b or $y/(2r)$	ε(%)	Ω
0·000	0·000	∞	0·242	15·00	0·64
0·010	0·027	12·51	0·250	15·91	0·62
0·020	0·107	6·26	0·260	17·15	0·61
0·030	0·240	4·18	0·270	18·43	0·60
0·040	0·425	3·15	0·280	19·75	0·59
0·050	0·663	2·53	0·282	20·00	0·58
0·060	0·960	2·11	0·290	21·10	0·58
0·061	1·000	2·07	0·300	22·50	0·57
0·070	1·30	1·82	0·310	23·93	0·56
0·080	1·70	1·60	0·317	25·00	0·55
0·087	2·00	1·47	0·320	25·39	0·55
0·090	2·15	1·43	0·330	26·89	0·54
0·100	2·65	1·30	0·340	28·43	0·54
0·107	3·00	1·23	0·350	30·00	0·53
0·110	3·20	1·19	0·360	31·60	0·53
0·120	3·80	1·10	0·370	33·23	0·52
0·123	4·00	1·08	0·380	34·90	0·52
0·130	4·45	1·03	0·381	35·00	0·52
0·138	5·00	0·97	0·390	36·60	0·52
0·140	5·15	0·96	0·400	38·32	0·51
0·150	5·90	0·91	0·410	40·00	0·52
0·151	6·00	0·90	0·420	41·86	0·51
0·160	6·69	0·86	0·430	43·67	0·51
0·164	7·00	0·84	0·437	45·00	0·50
0·170	7·54	0·82	0·440	45·51	0·50
0·175	8·00	0·80	0·450	47·38	0·50
0·180	8·43	0·78	0·460	49·27	0·50
0·186	9·00	0·76	0·464	50·00	0·50
0·190	9·36	0·75	0·470	51·18	0·50
0·197	10·00	0·73	0·480	53·13	0·50
0·200	10·35	0·72	0·490	55·00	0·50
0·210	11·37	0·70	0·500	57·08	0·50
0·216	12·00	0·69	0·562	70·00	0·50
0·220	12·44	0·68	0·631	85·00	0·51
0·230	13·56	0·66	0·696	100·00	0·53
0·240	14·71	0·64	0·819	130·00	0·56

This table also gives values of the strain as a function of the deflection, and vice versa. (See also Fig. 10.) Notations: Ω = dimensionless factor used for the calculation of the tension in the geosynthetic; y = geosynthetic deflection; b = width of the infinitely long void; $2r$ = diameter of the circular void; and ε = geosynthetic strain. (Note: in the case of a circular void, the values of ε and Ω given in this table are approximate.)

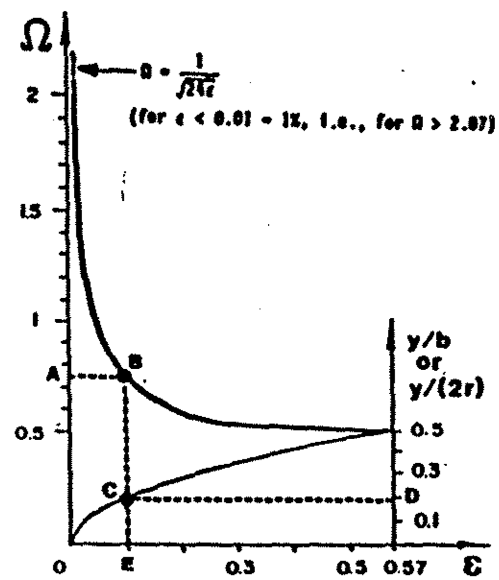


Fig. 10. Dimensionless factor Ω. (See also Table 2.) Notations: b = width of the infinitely long void; $2r$ = diameter of the circular void; y = geosynthetic deflection; ε = geosynthetic strain; and Ω = dimensionless factor. (Note that, in the case of a circular void, y is divided by $2r$, not by r.) This chart can be used as follows: (i) entering a known value of the geosynthetic strain, ε, in E and following EBA gives the value of Ω in A; (ii) entering a known value of the relative deflection, y/b or $y/(2r)$, in D and following DCBA gives the value of Ω in A; (iii) entering a known value of the relative deflection, y/b or $y/(2r)$, in D and following DCE gives the value of ε in E; and (iv) vice versa. (For example, ε = 0·1 (10%), Ω = 0·73, and y/b = 0·197 are related.)

given in Table 2 and Fig. 10 as a function of ε or y/b; ε = geosynthetic strain; and y = geosynthetic deflection. Basic SI units are: α (N/m), p (N/m^2), b (m), and y (m); Ω and ε are dimensionless.

Circular Void

As described by Giroud,[7] the deflected shape of the geosynthetic is not a sphere in the case of a circular void. As a consequence, incorporating $2r$ (diameter) instead of b (width) into eqns (11), (12) and (13), gives only an *approximate* value of the *average* geosynthetic strain, ε.

Since the strain is not uniform, the tension, α, in the case of a circular void is not uniformly distributed in the geosynthetic and its *average* value is given *approximately* by eqn (15) with r substituted for b.[3,7] It should be noted that, for a circular void, r is substituted for b in eqn (15) whereas $2r$ is used to determine Ω, as indicated in Table 2 and Fig. 10.

Equation (15) can be used for a circular void only if the geosynthetic has isotropic tensile characteristics, i.e. the same tensile characteristics in all directions. If this is not the case, recommendations given in the section 'Discussion of Special Problems' should be followed.

Applications of Tensioned Membrane Theory

Tensioned membrane theory can be used alone (i.e. not combined with arching theory) to solve design problems relating to the case of a geosynthetic acting alone and subjected to a uniformly distributed pressure. This typically occurs in the case of geomembranes directly overlying a void and subjected to pressure from a liquid. Typical design problems are as follows:

- Determine the maximum pressure that a geomembrane can withstand over a void of a given size.
- Select the required geomembrane properties for a geomembrane to bridge a given void when it is subjected to a given pressure.
- Determine the void size that a given geomembrane may bridge when it is subjected to a given pressure.
- Determine the deflection of a geomembrane subjected to a given pressure on a given void, and determine if the deflected geomembrane will come in contact with the bottom of the void.

A chart has been published[2] to help solve these problems. It is also possible to use Table 3 with H =0.

Combination of Arching and Tensioned Membrane Theories

The problem of a bottomless void is entirely solved by using eqns (10) and (15). The case when the geosynthetic comes in contact with the bottom of the void is more complex and will be discussed later in this paper, in the section 'Discussion of Special Problems'.

Equation (10) gives a relationship between the applied stress, the soil layer thickness, the void size, and the pressure on the geosynthetic. This equation was established using *arching theory*.

Equation (15) gives a relationship between the pressure on the geosynthetic, the void size, and the geosynthetic tensile characteristics (tension and strain). This equation was established using *tensioned membrane theory*.

The solution of typical design problems using the equations mentioned above is discussed in the next section.

SOLUTION OF TYPICAL DESIGN PROBLEMS

Overview of the Methods Used

In the presentation of the scope of this paper, a list of typical design problems was given. Solutions to these problems are presented below for the case when the geosynthetic does not come in contact with the bottom of the void. Solutions for the case where the geosynthetic comes in contact with the bottom of the void are presented in the section 'Discussion of Speecial Problems'.

Allowable Strain and Deflection
In all of the design cases considered below, the solution depends on the value of Ω, which depends either on the allowable geosynthetic strain, ε, or the allowable geosynthetic deflection, y. The *allowable geosynthetic strain* is the lesser of the maximum design strain for the considered geosynthetic and the strain beyond which the soil layer would be unacceptably deformed or cracked. The *allowable geosynthetic deflection* is considered when excessive deflection of the soil surface impairs the serviceability of the system. No method is proposed in this paper to evaluate the deflection of the soil surface; however, in the case of relatively thin soil layers, the soil surface deflection can be assumed to be on the same order as the geosynthetic deflection. In some instances, both the allowable geosynthetic strain and the allowable geosynthetic deflection may need to be considered.

Equations and Notations
All equations presented below were obtained by combining eqns (10) and (15). Notations for all subsequent equations are: b = width of the infinitely long void; r = radius of the circular void; Ω = dimensionless factor given in Table 2 as a function of ε or y; H = soil layer thickness; p = normal stress applied on the portion of the geosynthetic located over the void ('pressure on the geosynthetic'); q = uniformly distributed normal stress applied on the top of the soil layer; y = geosynthetic deflection; α = geosynthetic tension; γ = unit weight of soil; and ε = geosynthetic strain. Basis SI units are: b (m), r (m), H (m), p (N/m^2), q (N/m^2), y (m), α (N/m), and γ (N/m^3); Ω and ε are dimensionless.

Factor of Safety
In the following sections, each design problem is illustrated by an example. For the sake of simplicity, no factor of safety is used in the design examples. Engineers using the equations, tables, and charts presented in

this paper should use appropriate factors of safety. The factor of safety can be applied to the geosynthetic tension or the applied loads, with application to the geosynthetic tension being more common. The factor of safety should not be applied to the soil shear strength (as is commonly the case in geotechnical problems) due to the insensitivity of the arching theory results (eqn (5)) to the soil shear strength.

Determination of Required Geosynthetic Properties

The relevant equation for an infinitely long void is

$$\alpha/\Omega = pb = 2\gamma b^2(1 - e^{-0.5H/b}) + qb\, e^{-0.5H/b} \qquad (16)$$

Equation (16) can be rewritten in a dimensionless form as follows:

$$\frac{\alpha}{\gamma b^2 \Omega} = \frac{p}{\gamma b} = 2(1 - e^{-0.5H/b}) + \frac{q}{\gamma b} e^{-0.5H/b} \qquad (17)$$

Equations (16) and (17) can be used for a circular void if b is replaced by r. Equation (17) was used to establish the chart in Fig. 11.

The above equations can be used to solve problems that consist of determining the required geosynthetic tension, α, for a given strain, ε, when all other parameters are given (b or r, q, H, and γ). Alternatively, the chart given in Fig. 11 and the corresponding Table 3 can be used.

Example 1. The bedding soil supporting a geomembrane liner is placed on a geosynthetic reinforcement resting on a soil where karstic sinkholes may develop (Fig. 12). The function of the geosynthetic reinforcement is to support the bedding soil and the geomembrane liner should a sinkhole develop. The thickness of the bedding soil layer is 0·45 m and the depth of water on the geomembrane when the reservoir is full is 9 m. The unit weight of the bedding soil is 19,600 N/m^3. A deep sinkhole with a radius of 0·75 m is assumed for design purposes. Since the function of the geosynthetic reinforcement is only to act as a 'safety net', a rather large geosynthetic reinforcement strain is acceptable: ε = 10%. What is the required geosynthetic reinforcement tensile strength?
First, the applied stress, q, is calculated

$$q = 1000 \times 9{\cdot}81 \times 9 = 88\,290 \text{ N/m}^2$$

Then, eqn (16) is used as follows, with $H/r = 0{\cdot}45/0{\cdot}75 = 0{\cdot}6$

$$\alpha/\Omega = 2 \times 19\,600 \times (0{\cdot}75)^2(1 - e^{-0{\cdot}3}) + 88\,290 \times 0{\cdot}75\, e^{-0{\cdot}3}$$

$$\alpha/\Omega = 54\,395 \text{ N/m}$$

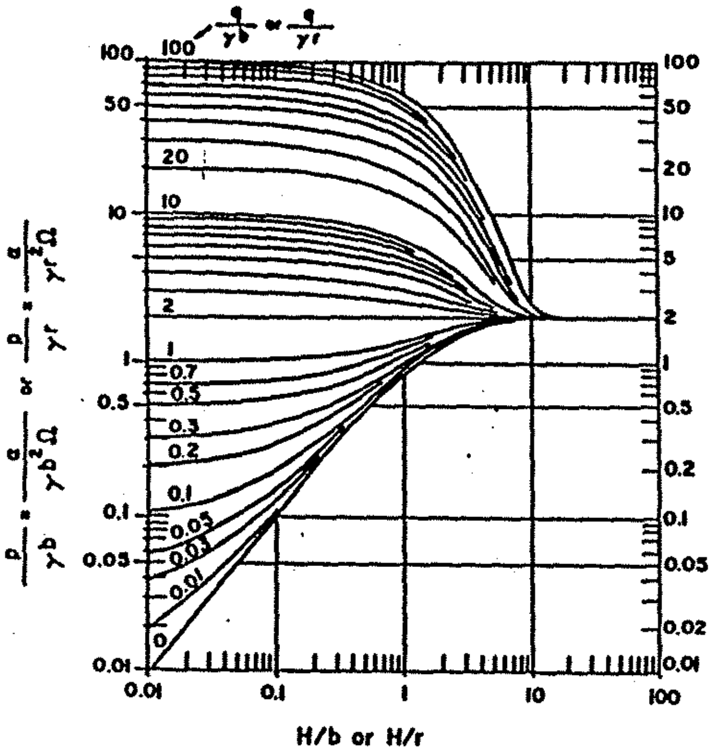


Fig. 11. Pressure on and tension in the geosynthetic. An example of use of this chart is given in Fig. 13. Notations: p = pressure on the geosynthetic over the void area; q = uniformly distributed normal stress applied on the top of the soil layer; H = thickness of the soil layer; γ = unit weight of soil; b = width of the infinitely long void; r = radius of the circular void; α = geosynthetic tension; and Ω = dimensionless factor given in Table 2 and Fig. 10. (Values of $p/(\gamma b)$ or $p/(\gamma r)$ used to draw the curves in this figure can be found in Table 3.)

Finally, according to Table 2 or Fig. 10, $\Omega = 0{\cdot}73$ for $\varepsilon = 10\%$. Therefore, the required value of the geosynthetic tension at a 10% strain is:

$\alpha = 0{\cdot}73 \times 54\,395 = 39\,708$ N/m $= 40$ kN/m

The same problem can be solved using the tables and charts with

$H/r = 0{\cdot}45/0{\cdot}75 = 0{\cdot}6$ and

$q/(\gamma r) = 88\,290/(19\,600 \times 0{\cdot}75) = 6{\cdot}0$

Table 3 of the chart given in Fig. 11 (see also Fig. 13) gives:

$\alpha/(\gamma r^2\Omega) = 4{\cdot}963$ hence

$\alpha = (4{\cdot}963) \times 19\,600 \times (0{\cdot}75)^2 \times 0{\cdot}73 = 39\,943$ N/m $= 40$ kN/m

It is interesting to compare the required geosynthetic reinforcement tension calculated above to that required if the bedding soil is a layer of compacted clay associated with the geomembrane to form a composite liner. In this case, it is important that the integrity of the clay layer be maintained. Therefore, the geosynthetic reinforcement strain must be small enough to prevent the development of tension cracks in the clay layer. Calculations similar to the above, with $\varepsilon = 1\%$ instead of 10%, give a required geosynthetic reinforcement tension of 113 kN/m, which is about three times greater than 40 kN/m. Therefore, the geosynthetic reinforcement required in the case of a 1% allowable strain has a tension about three times greater, and consequently a modulus about 30 times greater, than in the case of a 10% allowable strain. (Several layers of a very high-modulus geotextile would probably be needed.)

Determination of Required Soil Layer Thickness

The relevant equation for an infinitely long void is

$$H = 2b \ln \frac{[q/(\gamma b)] - 2}{[\alpha/(\gamma b^2\Omega)] - 2} \qquad (18)$$

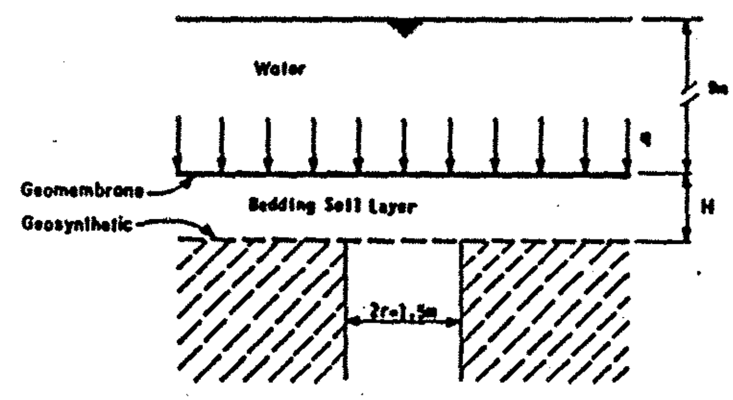


Fig. 12. Cross section for design examples.

TABLE 3
Pressure on the Geosynthetic

$q/(\gamma b)$ or $q/(\gamma r)$	*H/b or H/r*													
	0	*0·01*	*0·03*	*0·1*	*0·3*	*0·5*	*0·6*	*1·0*	*3·0*	*5·0*	*7·0*	*10·0*	*20·0*	∞
	(Values of $p/(\gamma b) = \alpha/(\gamma b^2 \Omega)$ or $p/(\gamma r) = \alpha/(\gamma r^2 \Omega)$)													
0·0	0	0·010	0·030	0·098	0·279	0·442	0·518	0·787	1·554	1·836	1·940	1·987	2·000	2·000
0·01	0·010	0·020	0·040	0·107	0·287	0·450	0·526	0·793	1·556	1·837	1·940	1·987	2·000	2·000
0·03	0·030	0·040	0·059	0·126	0·304	0·466	0·541	0·805	1·560	1·838	1·941	1·987	2·000	2·000
0·05	0·050	0·060	0·079	0·145	0·322	0·481	0·555	0·817	1·565	1·840	1·941	1·987	2·000	2·000
0·1	0·100	0·109	0·128	0·193	0·365	0·520	0·592	0·848	1·576	1·844	1·943	1·987	2·000	2·000
0·2	0·200	0·209	0·227	0·288	0·451	0·598	0·667	0·908	1·598	1·852	1·946	1·988	2·000	2·000
0·3	0·300	0·308	0·325	0·383	0·537	0·676	0·741	0·969	1·621	1·860	1·949	1·989	2·000	2·000
0·5	0·500	0·507	0·522	0·573	0·709	0·832	0·889	1·090	1·665	1·877	1·955	1·990	2·000	2·000
0·7	0·700	0·706	0·719	0·763	0·881	0·988	1·037	1·212	1·710	1·893	1·961	1·991	2·000	2·000
1·0	1·000	1·005	1·015	1·049	1·139	1·221	1·259	1·393	1·777	1·918	1·970	1·993	2·000	2·000
1·5	1·500	1·502	1·507	1·524	1·570	1·611	1·630	1·697	1·888	1·959	1·985	1·997	2·000	2·000
2·0	2·000	2·000	2·000	2·000	2·000	2·000	2·000	2·000	2·000	2·000	2·000	2·000	2·000	2·000
2·5	2·500	2·498	2·493	2·476	2·430	2·389	2·370	2·303	2·112	2·041	2·015	2·003	2·000	2·000
3·0	3·000	2·995	2·985	2·951	2·861	2·779	2·741	2·607	2·223	2·082	2·030	2·007	2·000	2·000
4·0	4·000	3·990	3·970	3·902	3·721	3·558	3·482	3·213	2·446	2·164	2·060	2·013	2·000	2·000
5·0	5·000	4·985	4·955	4·854	4·582	4·336	4·222	3·820	2·669	2·246	2·091	2·020	2·000	2·000
6·0	6·000	5·980	5·940	5·805	5·443	5·115	4·963	4·426	2·893	2·328	2·121	2·027	2·000	2·000
7·0	7·000	6·975	6·926	6·756	6·304	5·894	5·704	5·033	3·116	2·410	2·151	2·034	2·000	2·000
8·0	8·000	7·970	7·911	7·707	7·164	6·673	6·445	5·639	3·339	2·493	2·181	2·040	2·000	2·000
9·0	9·000	8·965	8·896	8·659	8·025	7·452	7·186	6·246	3·562	2·575	2·211	2·047	2·000	2·000
10	10·000	9·960	9·881	9·610	8·886	8·230	7·927	6·852	3·785	2·657	2·242	2·054	2·000	2·000
15	15·000	14·935	14·806	14·366	13·189	12·124	11·631	9·885	4·901	3·067	2·393	2·088	2·000	2·000
20	20·000	19·910	19·732	19·122	17·493	16·018	15·335	12·918	6·016	3·478	2·544	2·121	2·000	2·000
25	25·000	24·885	24·658	23·878	21·796	19·912	19·039	15·950	7·132	3·888	2·695	2·155	2·000	2·000
30	30·000	29·860	29·583	28·634	26·100	23·806	22·743	18·983	8·248	4·298	2·846	2·189	2·000	2·000
40	40·000	39·810	38·434	38·147	34·707	31·594	30·151	25·048	10·479	5·119	3·148	2·256	2·000	2·000
50	50·000	49·761	49·285	47·659	43·314	39·382	37·559	31·113	12·710	5·940	3·449	2·323	2·000	2·000
60	60·000	59·711	59·136	57·171	51·921	47·170	44·967	37·179	14·942	6·761	3·751	2·391	2·000	2·000
70	70·000	69·661	68·988	66·684	60·528	54·958	52·376	43·244	17·173	7·582	4·053	2·458	2·000	2·000
80	80·000	79·611	78·839	76·196	69·135	62·746	59·784	49·309	19·404	8·403	4·355	2·526	2·000	2·000
90	90·000	89·561	88·690	85·708	77·742	70·534	67·192	55·375	21·635	9·223	4·657	2·593	2·000	2·000
100	100·000	99·511	98·541	95·220	86·349	78·322	74·600	61·440	23·867	10·044	4·959	2·660	2·000	2·000

This table gives $p/(\gamma b)$ or $p/(\gamma r)$ and the geosynthetic tension as a function of the other parameters involved. Notation: p = pressure on the geosynthetic over the void area; q = uniformly distributed normal stress applied on the top of the soil layer; H = thickness of the soil layer; γ = unit weight of the soil in the soil layer; b = width of the infinitely long void; r = radius of the circular void; α = geosynthetic tension; and Ω = dimensionless factor given in Table 2 as a function of the geosynthetic strain, ε. Note that: values of $p/(\gamma b)$ or $p/(\gamma r)$ for H/b = 0 are identical to values of $q/(\gamma b)$ or $q/(\gamma r)$; and $p = 2\gamma b$ if H is greater than approximately $20b$ and $p = 2\gamma r$ if H is greater than approximately $20r$. (See the chart given in Fig. 11.)

The same equation can be used for a circular void by substituting r for b.

The above equations can be used to solve problems that consist of determining the required soil layer thickness, H, when all other parameters are given (b or r, q, γ, α, and ε). Alternatively, the charts given in Fig. 11, and the corresponding Table 3, can be used.

Example 2. This example is identical to Example 1, except that the soil layer thickness, H, is unknown, and the geosynthetic tension at a strain $\varepsilon = 10\%$ is known and is equal to 40 kN/m. What is the required soil layer thickness?

From Example 1, the relevant parameters are: $q = 88\,290$ N/m^2; $\gamma = 19\,600$ N/m^3; and $r = 0{\cdot}75$ m.

In order to use eqn (18), the following values must be calculated

$q/(\gamma r) = 6{\cdot}0$ (from Example 1)
$\alpha/(\gamma r^2 \Omega) = 40\,000/(19\,600 \times (0{\cdot}75)^2 \times 0{\cdot}73) = 4{\cdot}97$

Hence, using eqn 18

$$H = 2 \times 0{\cdot}75 \times \ln \frac{6{\cdot}0 - 2}{4{\cdot}97 - 2} = 0{\cdot}44 \text{ m}$$

It is also possible to solve this problem using Table 3 or Fig. 11 which gives $H/r = 0{\cdot}6$ for $q/(\gamma r) = 6{\cdot}0$ and $\alpha/(\gamma r^2 \Omega) = 4{\cdot}97$ (see Fig. 13). Hence, $H = 0{\cdot}6 \times 0{\cdot}75 = 0{\cdot}45$ m.

Determination of Maximum Void Size

There is no simple equation giving the void size (b or r) as a function of the other parameters. In order to determine the maximum void size that a given soil layer-geosynthetic system can bridge, it is necessary to solve eqn (16) by trial and error. To facilitate the process, a chart has been established (Fig. 14) by rewriting the two parts of eqn (17) in a dimensionless form as follows:

$$\frac{p}{\gamma H} = \frac{2(1 - e^{-0.5H/b})}{H/b} + \frac{q}{\gamma H} e^{-0.5H/b} \tag{19}$$

$$\frac{p}{\gamma H} = \frac{\alpha}{\gamma H^2 \Omega} \frac{H}{b} \tag{20}$$

In Fig. 14, eqn (19) is represented by a family of curves and eqn (20) is represented by a family of straight lines at 45°. For a given set of parameters, the abscissa of the intersection between the relevant curve

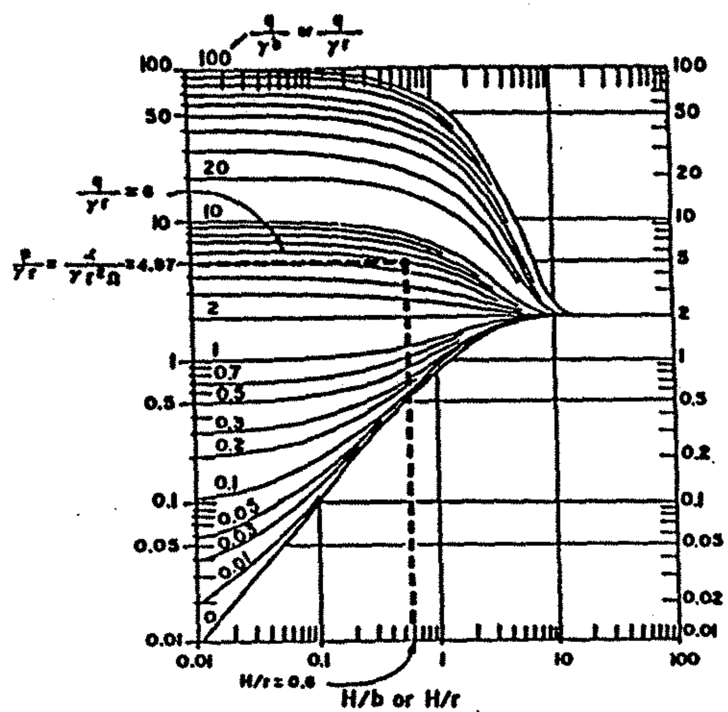


Fig. 13. Example of use of the chart given in Fig. 11.

and the relevant straight line gives the maximum value of the width, b, of an infinitely long void or the radius, r, of a circular void.

Example 3. This example is identical to Example 1, except that the radius of the void, r, is unknown, and the geosynthetic tension at a strain $\varepsilon = 10\%$ is known and is equal to 40 kN/m. What maximum void radius can be bridged by the considered soil-geosynthetic system?

From Example 1, the relevant parameters are: $q = 88\,290$ N/m^2; $\gamma = 19\,600$ N/m^3; and $H = 0{\cdot}45$ m.

In order to use the chart given in Fig. 14, the following must be calculated

$q/(\gamma H) = 88\,290/(19\,600 \times 0{\cdot}45) = 10{\cdot}0$

$\alpha/(\gamma H^2 \Omega) = 40\,000/(19\,600 \times (0{\cdot}45)^2 \times 0{\cdot}73) = 13{\cdot}8$

(Note: $\Omega = 0{\cdot}73$ is obtained from Table 2 with $\varepsilon = 10\%$)

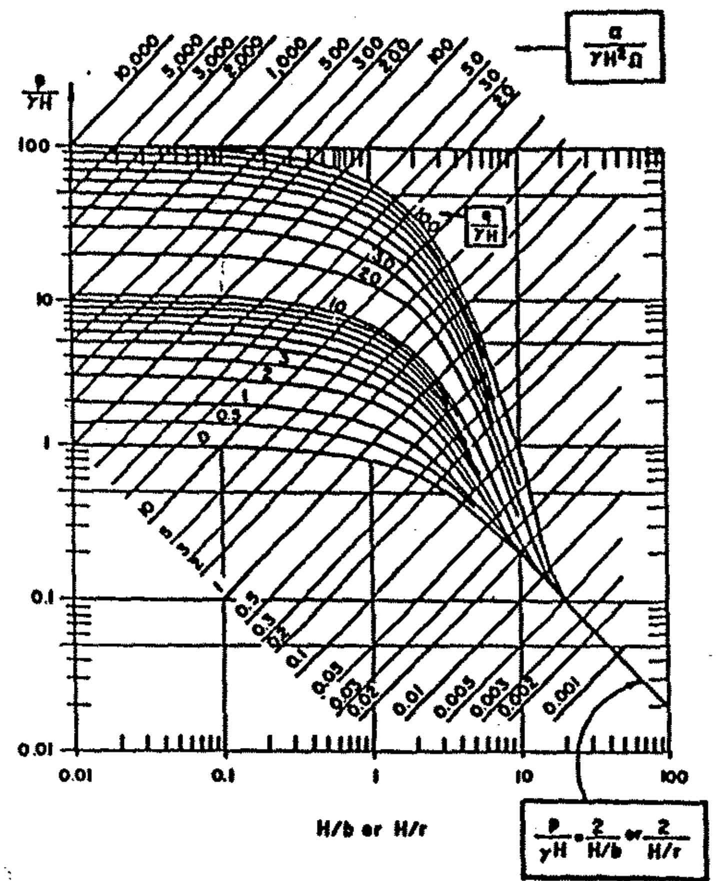


Fig. 14. Pressure on and tension in the geosynthetic. An example of use of this chart is given in Fig. 15. Notations: p = pressure on the geosynthetic over the void area; q = uniformly distributed normal stress applied on the top of the soil layer; H = thickness of the soil layer; γ = unit weight of soil; b = width of the infinitely long void; r = radius of the circular void; α = geosynthetic tension; and Ω = dimensionless factor given in Table 2 and Fig. 10. (Values of $p/(\gamma H)$ which were used to draw the curves in this figure can be found in Table 4.)

In Fig. 14, the curve related to $q/(\gamma H) = 10$ and the straight line at 45° related to $\alpha/(\gamma H^2\Omega) = 13{\cdot}8$ intersect at a point the abscissa of which is $H/r = 0{\cdot}6$ (see Fig. 15). Hence

$$r_{max} = 0{\cdot}45/0{\cdot}6 = 0{\cdot}75 \text{ m}$$

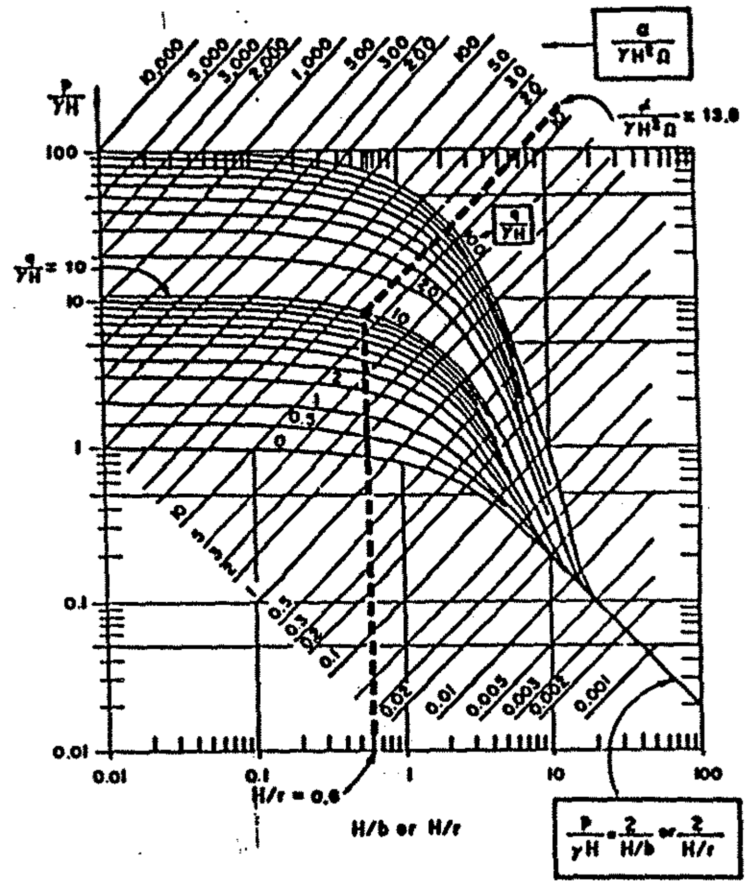


Fig. 15. Example of use of the chart given in Fig. 14.

Determination of the Maximum Load

The relevant equation for an infinitely long void is

$$q = 2\gamma b + \left\{ \frac{[\alpha/(\gamma b^2\Omega)] - 2}{e^{-0{\cdot}5H/b}} \right\} \gamma b \qquad (21)$$

The same equation can be used for a circular void by substituting r for b.

The above equation can be used to solve problems that consist of determining the maximum uniform normal stress, q, which can be applied on the top of the soil layer, when all other parameters are given (b or r, γ, H, α, and ε). Alternatively, the charts given in Fig. 11 or 14 can be used, as well as Table 3 or 4.

TABLE 4
Pressure on the Geosynthetic

$q/(\gamma H)$	H/b or H/r: 0	0·01	0·1	0·3	0·5	0·7	1·0	3·0	5·0	7·0	10·0	20·0	∞
	(Values of $p/(\gamma H)$)												
0·0	∞	0·998	0·975	0·929	0·885	0·844	0·787	0·518	0·367	0·277	0·199	0·100	0
0·5	∞	1·495	1·451	1·359	1·274	1·196	1·090	0·629	0·408	0·292	0·202	0·100	0
1·0	∞	1·993	1·927	1·789	1·664	1·548	1·393	0·741	0·449	0·307	0·205	0·100	0
2·0	∞	2·988	2·878	2·650	2·442	2·253	2·000	0·964	0·531	0·337	0·212	0·100	0
3·0	∞	3·983	3·829	3·511	3·221	2·958	2·607	1·187	0·613	0·368	0·219	0·100	0
4·0	∞	4·978	4·780	4·371	4·000	3·663	3·213	1·410	0·696	0·398	0·226	0·100	0
5·0	∞	5·973	5·732	5·232	4·779	4·367	3·820	1·634	0·778	0·428	0·232	0·100	0
6·0	∞	6·968	6·683	6·093	5·558	5·072	4·426	1·857	0·860	0·458	0·239	0·100	0
7·0	∞	7·963	7·634	6·954	6·336	5·777	5·033	2·080	0·942	0·488	0·246	0·100	0
8·0	∞	8·958	8·585	7·814	7·115	6·481	5·639	2·303	1·024	0·519	0·253	0·100	0
9·0	∞	9·953	9·536	8·675	7·894	7·186	6·246	2·526	1·106	0·549	0·259	0·100	0
10	∞	10·948	10·488	9·536	8·673	7·891	6·852	2·749	1·188	0·579	0·266	0·100	0
15	∞	15·923	15·244	13·839	12·567	11·414	9·885	3·865	1·598	0·730	0·300	0·101	0
20	∞	20·898	20·000	18·143	16·461	14·938	12·918	4·981	2·009	0·881	0·333	0·101	0
25	∞	25·873	24·756	22·446	20·355	18·461	15·950	6·096	2·419	1·032	0·367	0·101	0
30	∞	30·848	29·512	26·750	24·249	21·984	18·983	7·212	2·830	1·183	0·401	0·101	0
40	∞	40·798	39·025	35·357	32·037	29·031	15·048	9·443	3·651	1·485	0·468	0·102	0
50	∞	50·748	48·537	43·964	39·825	36·078	31·113	11·674	4·471	1·787	0·536	0·102	0
60	∞	60·698	58·049	52·571	47·613	43·125	37·179	13·906	5·292	2·089	0·603	0·103	0
70	∞	70·648	67·561	61·178	55·401	50·172	43·244	16·137	6·113	2·391	0·670	0·103	0
80	∞	80·599	77·074	69·786	63·189	57·219	49·309	18·368	6·934	2·693	0·738	0·104	0
90	∞	90·549	86·586	78·392	70·977	64·266	55·375	20·600	7·755	2·995	0·805	0·104	0
100	∞	100·499	96·098	86·999	78·765	71·313	61·440	22·831	8·576	3·297	0·872	0·105	0

This table gives $p/(\gamma H)$. Notation: p = pressure on the geosynthetic over the void area; q = uniformly distributed normal stress applied on the top of the soil layer; H = thickness of the soil layer; γ = unit weight of the soil in the soil layer; b = width of the infinitely long void; and r = radius of the circular void. Note that: values of $p/(\gamma H)$ are equal to: $1 + q/(\gamma H)$ if $H/b = 0$ or $H/r = 0$; and $2b/H$ or $2r/H$ if $H/b > 20$ or $H/r > 20$. (See the chart given in Fig. 14.)

Example 4. This example is identical to Example 1, except that the stress on top of the soil layer, q, is unknown, and the geosynthetic tension at strain ε = 10% is known and is equal to 40 kN/m. What maximum stress on top of the soil layer can be supported by the soil-geosynthetic system? From Example 1, the relevant parameters are: $H = 0{\cdot}45$ m; $r = 0{\cdot}75$ m; and $\gamma = 19\,600\ \mathrm{N/m^3}$.

In order to use eqn (21), the value of Ω must be obtained first from Table 2

$\Omega = 0{\cdot}73$ for $\varepsilon = 10\%$.

Then, eqn (21) is used as follows

$$q = 2\times 19\,600\times 0{\cdot}75 + \left\{\frac{[40\,000/(19\,600\times(0{\cdot}75)^2\times 0{\cdot}73)]-2}{e^{-0{\cdot}5\times 0{\cdot}45/0{\cdot}75}}\right\}19\,600\times 0{\cdot}75$$

$$= 88\,334\ \mathrm{N/m^2}$$

The problem can also be solved using charts and tables. To use Table 3 or the chart given in Fig. 11, the following must be calculated:

$H/r = 0{\cdot}45/0{\cdot}75 = 0{\cdot}6$

$\alpha/(\gamma r^2\Omega) = 40\,000/(19\,600\times(0{\cdot}75)^2\times 0{\cdot}73) = 4{\cdot}97$

With $H/r = 0{\cdot}6$ and $\alpha/(\gamma r^2\Omega) = 4{\cdot}97$, Table 3 or the chart given in Fig. 11 show that $q/(\gamma r) = 6$ (see Fig. 13). Therefore

$q = 6\times 19\,600\times 0{\cdot}75 = 88\,200\ \mathrm{N/m^2} = 88\ \mathrm{kN/m^2}$

To use the chart given in Fig. 14, the following must be calculated

$\alpha/(\gamma H^2\Omega) = 40\,000/(19\,600\times(0{\cdot}45)^2\times 0{\cdot}73) = 13{\cdot}8$

With $H/r = 0{\cdot}6$ and $\alpha/(\gamma H^2\Omega) = 13{\cdot}8$, the chart given in Fig. 14 shows that $q/(\gamma H) = 10$ (see Fig. 15). Therefore

$q = 10\times 19\,600\times 0{\cdot}45 = 88\,200\ \mathrm{N/m^2} = 88\ \mathrm{kN/m^2}$

DISCUSSION OF SPECIAL PROBLEMS

Anisotropic Geosynthetic

A geosynthetic is isotropic regarding a given characteristic when this characteristic has the same value in all directions. In this paper, a geosynthetic will be considered isotropic when it has the same tension–strain curve in all directions. This requirement is fulfilled by some nonwoven geotextiles. Woven geotextiles and biaxial geogrids are stronger in two directions ('principal directions') than in the others and, therefore, they are anisotropic. However, we assume that the design method presented in this paper can be used with woven geotextiles and biaxial geogrids that have the same tensile characteristics in the two principal directions (i.e. in the design, these materials are considered isotropic).

Special precautions must be taken when using the design method presented in this paper for geosynthetics that cannot be considered isotropic, as discussed below.

Infinitely Long Void

In the case of an infinitely long void, no geosynthetic tension is required in the direction of the length of the void (according to the plane-strain model which corresponds to an infinitely long void). Therefore, the value of α to be used in the equations, tables, and charts related to the infinitely long void is the geosynthetic tension in the direction of the width of the void for the considered design strain. However, some strength is required lengthwise in places where the actual situation departs from a pure plane-strain situation (for instance near the end of the void).

Circular Void

In the case of a circular void, the tensioned membrane equation (eqn (15)) is valid only if the geosynthetic has isotropic tensile characteristics. For practical purposes, eqn (15), and other equations as well as tables and charts related to circular voids, can be used for woven geotextiles and biaxial geogrids that have the same tension–strain curve *in the two principal directions* (instead of *in all directions* for a truly isotropic material). For woven geotextiles and biaxial geogrids that have different tensile characteristics in the two principal directions, two cases can be considered, depending on the ratio between the geosynthetic tensions at the design strain in the weak and the strong directions: (i) if the ratio is more than 0·5, α should be taken equal to the tension in the weak direction; and (ii) if the ratio is less than 0·5, α should be taken equal to half the tension in the strong direction.

The rationale for the above recommendation is as follows. There are two conservative approaches and the less conservative, which is closer to reality, should be selected.

The first conservative approach consists of designing with an isotropic geosynthetic weaker than the considered anisotropic geosynthetic. This is achieved by taking the geosynthetic strength in all directions equal to the

strength in the weak direction, α_{weak}. Equation (15) thus gives
pressure which can be carried by the geosynthetic

$$p_1 = \frac{\alpha_{weak}}{r\Omega}$$

The second conservative approach consists of designing with: (i) larger than the circular void by replacing the circular void by an in long void with a width, b, equal to the diameter, $2r$, of the circula and (ii) a geosynthetic weaker than the considered anisotropic geos tic by neglecting the tensile strength in the weak direction (α_{weak} Equation (15) thus gives for the pressure which can be carried geosynthetic

$$p_2 = \frac{\alpha_{strong}}{b\Omega} = \frac{\alpha_{strong}}{2r\Omega}$$

To compare p_1 and p_2, it is important to note that the values of Ω i (22) and (23) are identical because they are both determined for according to Table 2. Therefore, the comparison between p_1 and p down to a comparison between α_{weak} and 0·5 α_{strong}.

It appears that

$$p_1 > p_2 \quad \text{if} \quad \alpha_{weak} > 0{\cdot}5\alpha_{strong}$$

$$p_1 < p_2 \quad \text{if} \quad \alpha_{weak} < 0{\cdot}5\alpha_{strong}$$

hence the above recommendation.

There is another consideration when an anisotropic geosynthetic over a circular void. The complex pattern of strains in the geosy resulting from different tensions in different directions may have trimental effect on the behavior of the geosynthetic. Therefore recommended that for holes which can be modeled as circular, one following solutions be adopted: (i) an isotropic geosynthetic (only nonwoven geotextiles are isotropic but usually they do not have ad tensile characteristics for this application); or (ii) a 'practically iso geosynthetic (such as a woven geotextile or a biaxial geogrid having s tension–strain curves in the two principal directions); or (iii) two per cularly orientated layers of the same anisotropic geosynthetic.

Geosynthetic in Contact with Void Bottom

In some cases, the geosynthetic elongates to the point that it con contact with the bottom of the void (Fig. 1(c)); the geosynthetic defle is then equal to the void depth ($y = D$). In design, these cases correspond to a calculated geosynthetic deflection greater than or equal to the void depth ($y \geq D$). Usually, the design is complete when it is found that $y \geq D$. However, it may be of interest to *determine the pressure actually transmitted to the bottom of the void.* This pressure is obtained by subtracting the pressure inducing geosynthetic tension (which results from the tensioned membrane effect) from the pressure exerted by the soil layer on the geosynthetic.

In the case of an infinitely long void, the following equation can be obtained by subtracting the pressure given by eqn (15) from the pressure given by eqn (10)

$$p_b = 2\gamma b(1 - e^{-0.5H/b}) + q e^{-0.5H/b} - \frac{\alpha}{b\Omega} \qquad (24)$$

where: p_b = pressure transmitted to the bottom of the void; γ = unit weight of the soil (in the soil layer above the geosynthetic); b = width of the infinitely long void; H = soil layer thickness; q = uniformly distributed normal stress applied on the top of the soil layer; α = geosynthetic tension corresponding to the geosynthetic strain, ε, when the geosynthetic is in contact with the bottom of the void (i.e., ε corresponding to a deflection $y = D$ in Table 2); Ω = dimensionless factor given in Table 2 as a function of ε or y; and y = geosynthetic deflection, which, in this case, is equal to D; and D = depth of the void. Basic SI units are: p_b (N/m²), γ (N/m³), b (m), H (m), q (N/m²), α (N/m), y (m), and D (m); Ω is dimensionless. Note that eqn (24) assumes that the shape of the bottom of the void is approximately cylindrical with a circular cross section, so the geosynthetic will come in contact with all points on the surface of the void at the same time. If this were not the case, portions of the geosynthetic which come in contact with the bottom of the void last would elongate more than the others.

The same equation can be used for a circular void by substituting r for b, with r = radius of the circular void.

If a negative value were obtained for p_b when using the above equation, it would mean that the load on the geosynthetic is not large enough to force the geosynthetic to come in contact with the bottom of the void.

Example 5. This example is identical to Example 1 except that: (i) the void is not bottomless but has a depth D = 0·2 m; and (ii) the geosynthetic tension–strain curve is assumed to be a straight line between the origin and a tension α = 40 kN/m for a strain ε = 10%. What is the stress transmitted to the bottom of the hole?

From Example 1, the relevant parameters are: r = 0·75 m; H = 0·45 m; γ = 19 600 N/m³; and q = 88 290 N/m².

First, the approximate value of the average strain of the geosynthetic when it is in contact with the bottom of the void (assumed spherical) must be determined using Table 2 with y (geosynthetic deflection) = D (void depth)

$$y/2r = D/2r = 0{\cdot}2/(2 \times 0{\cdot}75) = 0{\cdot}133$$

Hence, interpolating in Table 2, ε = 4·65% and Ω = 1·01.

Then, the geosynthetic tension corresponding to a 4·65% geosynthetic strain can be calculated as follows:

$$\alpha = 40\,000 \times 4{\cdot}65/10 = 18\,600 \text{ N/m}$$

Finally, eqn (24) can be used with the values H/r = 0·6 and q = 88 290 N/m² determined in Example 1. This equation gives the stress transmitted to the bottom of the void as follows

$$p_b = 2 \times 19\,600 \times 0{\cdot}75(1 - e^{-0{\cdot}3}) + 88\,290\,e^{-0{\cdot}3} - \frac{18\,600}{0{\cdot}75 \times 1{\cdot}01}$$

$$= 73\,029 - 24\,554 = 48\,475 \text{ N/m}^2 = 48{\cdot}5 \text{ kN/m}^2$$

Therefore, this design example can be summarized as follows:

- A stress of 88·3 kN/m² is applied on top of the soil layer.
- As a result of soil arching, the soil layer transmits only a stress of 73 kN/m² to the top of the geosynthetic.
- As a result of the tensioned membrane effect, the geosynthetic supports 24·5 kN/m².
- The remainder, 48·5 kN/m², is transmitted to the bottom of the void.

It should be noted that, if the depth of the void had been D = 0·3 m, the strain of the geosynthetic would have been 10% and the last term of the above equation would have been

$$\frac{\alpha}{r\Omega} = \frac{40\,000}{0{\cdot}75 \times 0{\cdot}73} = 73\,059 \text{ N/m}^2$$

Hence, p_b = 0. In this case, Example 5 becomes identical to Example 1.

Influence of Soil Layer Thickness

The influence of the thickness of the soil layer is illustrated in Fig. 11. Three cases can be considered:

(1) *Large Applied Stress.* If the applied stress, q, is large (i.e. $q > 2\gamma b$ or $2\gamma r$), the pressure, p, on the geosynthetic and consequently the required geosynthetic tension, α, decrease towards a limit when the soil layer thickness increases. In this case, *it is beneficial to increase the thickness of the soil layer.* For each particular situation, the amount by which the thickness should be increased can be determined using the chart given in Fig. 11 or Table 3. The chart and table show that it would be useless to increase the soil layer thickness beyond a limiting value of $H = 20b$ or $20r$.

(2) *Small Applied Stress.* If the applied stress, q, is small (i.e. $q < 2\gamma b$ or $2\gamma r$), the pressure, p, on the geosynthetic and consequently the required geosynthetic tension, α, increase toward a limit when the soil thickness increases. In this case, from the perspective of the design of the geosynthetic, *it is detrimental to increase the thickness of the soil layer.* (This is because the added load due to soil weight is not fully compensated by the effect of soil arching.)

(3) *Limit Applied Stress.* If the applied stress, q, equals the limit (i.e. $q = 2\gamma b$ or $2\gamma r$), the pressure, p, on the geosynthetic remains constant and equal to q, regardless of the soil layer thickness.

The limit values for p and α are independent of the applied stress, q. The limit value for the pressure on the geosynthetic is

$$p_{\text{lim}} = 2\gamma b \quad \text{for an infinitely long void} \tag{25}$$

The limit value for the required geosynthetic tension is

$$\alpha_{\text{lim}} = 2\gamma b^2 \Omega \quad \text{for an infinitely long void} \tag{26}$$

Equations (25) and (26) can be used for a circular void by substituting r for b.

Comparison with Tensioned Membrane Theory

In the past, the tensioned membrane theory has been used alone to evaluate the required tensile characteristics of a geosynthetic located beneath a soil layer and bridging a void. This method neglects arching in

the soil layer and is, therefore, conservative. This conservativeness can be evaluated by comparing the pressure on the geosynthetic over the void area, p, calculated taking soil arching into account to the following value obtained by neglecting soil arching

$$p_0 = \gamma H + q \tag{27}$$

where: p_0 = pressure on the geosynthetic over the void area neglecting soil arching; γ = unit weight of the soil in the soil layer; H = thickness of the soil layer; and q = uniformly distributed normal stress applied on the top of the soil layer. Basic SI units are: p_0 (N/m^2), γ (N/m^3), H (m), and q (N/m^2).

The pressure, p, obtained taking soil arching into account is given by eqn (10).

Values of p/p_0 are given in Table 5 and Fig. 16. It appears that neglecting soil arching is conservative. However, when the soil thickness, H, is large

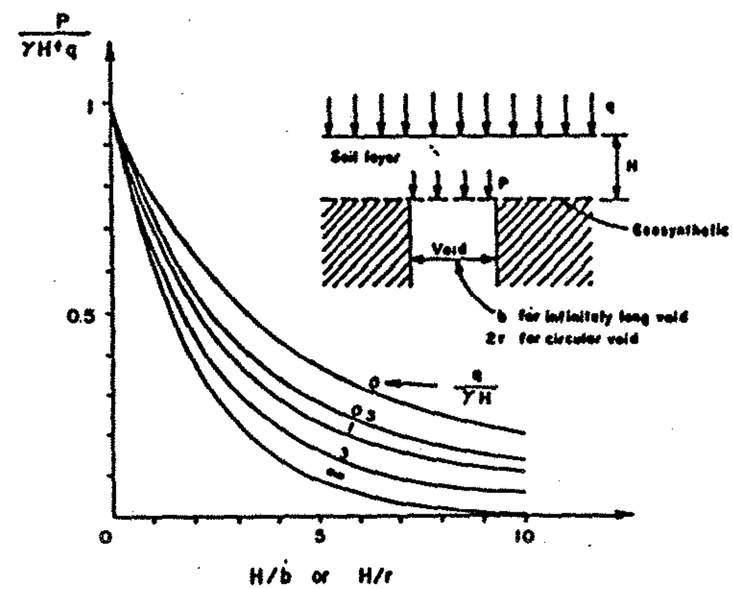


Fig. 16. Effectiveness of soil arching. The curves give the ratio between the pressure, p, on the geosynthetic over the void area, calculated taking soil arching into account, and the pressure $p_0 = \gamma H + q$ obtained by neglecting soil arching. The values of p/p_0 used to plot the curve can be found in Table 5.

TABLE 5
Effectiveness of Soil Arching

$q/(\gamma H)$	H/b or H/r: 0	0·3	0·6	1	1·5	2	2·3	3	4	5	10	20	∞
	(Values of p/p_0)												
0	1	0·929	0·864	0·787	0·704	0·632	0·571	0·518	0·432	0·367	0·199	0·100	0
0·5	1	0·906	0·823	0·727	0·626	0·544	0·476	0·420	0·333	0·272	0·135	0·067	0
1	1	0·895	0·812	0·697	0·588	0·500	0·429	0·371	0·284	0·225	0·103	0·050	0
2	1	0·883	0·782	0·667	0·549	0·456	0·381	0·321	0·234	0·177	0·071	0·033	0
3	1	0·878	0·772	0·652	0·530	0·434	0·358	0·297	0·210	0·153	0·055	0·025	0
5	1	0·872	0·761	0·637	0·511	0·412	0·334	0·272	0·185	0·130	0·039	0·017	0
10	1	0·867	0·752	0·623	0·493	0·392	0·312	0·250	0·162	0·108	0·024	0·009	0
20	1	0·864	0·747	0·615	0·481	0·380	0·300	0·237	0·149	0·096	0·016	0·005	0
∞	1	0·861	0·741	0·607	0·472	0·368	0·287	0·223	0·135	0·082	0·007	0·000	0

This table gives the ratio of the pressure on the geosynthetic over the void area calculated taking arching into account (p) or neglecting arching (p_0). The value of p is given by eqn (10). The value of p_0 is given by eqn (27). Notation: q = uniformly distributed stress applied on the top of the soil layer; γ = unit weight of the soil in the soil layer; H = soil layer thickness; b = width of an infinitely long void; and r = radius of circular void. (See also Fig. 16.)

compared to the width or radius of the void, neglecting soil arching is over-conservative.

CONCLUSION

This paper has presented an approach to the design of soil layer-geosynthetic systems overlying voids. The design approach superimposes arching theory for the soil layer with tensioned membrane theory for the geosynthetic. The analysis presented in this paper shows that neglecting soil arching would be over-conservative in many instances. The paper presents equations, tables, and charts that make it easy to perform design analyses for a range of possible field situations.

The analysis shows that the thickness of the soil layer associated with the geosynthetic plays a significant role. In contrast, the soil mechanical properties do not. It should not be inferred, however, that any soil will provide the same degree of arching. The equations used to prepare the tables and charts assume that the friction angle of the soil is at least 20°. Granular soils virtually always meet this condition. However, they should be well compacted to ensure arching because loose granular soils tend to contract when they are sheared or vibrated, which may destroy the arch.

Further refinements of the method presented herein can be considered. For instance, it is possible that the degree of soil arching (i.e. the amount of soil shear strength mobilized) depends on the geosynthetic strain, whereas the method presented in this paper does not consider the concept of degree of soil arching. Also, the method could be expanded to include cohesive soils, and could be refined to take into account elongation of the geosynthetic in the anchorage zone. Lastly, the method could be expanded to consider a system of regularly spaced voids.

In spite of its limitations, the method presented in this paper is believed to be a useful tool for engineers designing soil-geosynthetic systems resting on subgrades where voids may develop.

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ATTACHMENT G

REVISED
CONSTRUCTION QUALITY ASSURANCE PLAN
FINAL COVER CONSTRUCTION
LOPEZ CANYON SANITARY LANDFILL
LAKE VIEW TERRACE, CALIFORNIA

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APPENDIX

1. SITE AND PROJECT CONTROL

1.1 Project Coordination Meetings

To guarantee a high degree of quality during installation, clear, open channels of communication are essential. To this end, meetings of key project personnel are necessary.

1.1.1 Resolution Meeting

Following the completion of the design, plans, and specifications for the project, a Resolution Meeting will be held. This meeting will include the Geosynthetic CQA Managing Engineer, the Geosynthetic Site CQA Manager, the Soils CQA Managing Engineer, the Soils Site CQA Manager, the Engineer, and the Project Manager.

The purpose of this meeting is to begin planning for coordination of construction tasks, anticipate any installation problems which might cause difficulties and delays in construction, and, above all, present the CQA Plan to all of the parties involved. It is very important that the criteria regarding testing, repair, etc., be known and accepted by all parties prior to the installation of geosynthetic materials and construction of the soil components of the final cover system.

1.1.2 Preconstruction Meeting

A Preconstruction Meeting will be held at the site prior to installation of the geosynthetic materials and construction of soil components. As a minimum, the Preconstruction Meeting will be attended by the Geosynthetic Installer's Superintendent, the Geosynthetic CQA Managing Engineer, the Soils CQA Managing Engineer, the Geosynthetic Site CQA Manager, the Soils CQA Manager, the Earthwork Contractor, and the Project Manager.

1.1.3 Progress Meetings

A weekly progress meeting will be held between the Soils Site CQA Manager, the Geosynthetic Site CQA Manager, the Geosynthetic Installer's Superintendent, the Earthworks Contractor, the Project Manager, and any other concerned parties. The progress meetings will be used to discuss current progress, planned activities for the upcoming week, and any new business or revisions to the work. The Site CQA Managers will document any problems, decisions, or questions arising at this meeting in their daily reports. Any matter requiring action which is raised in this meeting will be reported to the appropriate parties. Minutes of the weekly progress meetings shall be documented by the Project Manager or his representative and distributed to all appropriate parties.

1.1.4 Problem or Work Deficiency Meeting

A special meeting will be held when and if a problem or deficiency is present or likely to occur. The meeting will be attended by the affected contractors, the Project Manager, the Site CQA Manager(s), and other parties as appropriate. If the problem requires a design modification, the Engineer should either be present at, consulted prior to, or notified immediately upon conclusion of this meeting. The purpose of the work deficiency meeting is to define and resolve the problem or work deficiency.

1.2 Project Control Visits

Periodically, the construction site will be visited by each CQA Managing Engineer and/or each CQA Project Manager (if different from the CQA Managing Engineer). If possible, each such visit should be coordinated with a similar visit by the Engineer. State of California regulatory officials may be informed of the dates of the visits.

2. DOCUMENTATION

2.1 General

An effective CQA plan depends largely on recognition of all construction activities that should be monitored, and on assigning responsibilities for the monitoring of each activity. This is most effectively accomplished and verified by the documentation of quality assurance activities. Each CQA Representative will document that all quality assurance requirements have been addressed and satisfied.

Each Site CQA Manager will provide the Project Manager with signed descriptive remarks, data sheets, and logs to verify that all monitoring activities have been carried out. Each Site CQA Manager will also maintain at the job site a complete file of plans and specifications, a CQA plan, checklists, test procedures, daily logs, and other pertinent documents.

2.2 Daily Recordkeeping

Standard reporting procedures will include preparation of daily CQA documentation which, at a minimum, will consist of: (i) field notes, including memoranda of meetings and/or discussions with the Earthwork Contractor, Installer, or Project Manager; (ii) CQA monitoring logs, and testing data sheets; and (iii) construction problem and solution summary sheets. This information will be regularly submitted to and reviewed by the Project Manager.

2.2.1 Monitoring Logs and Testing Data Sheets

Monitoring logs and testing data sheets will be prepared daily. At a minimum, these logs and data sheets will include the following information:

- an identifying sheet number for cross referencing and document control;

- date, project name, location, and other identification;
- data on weather conditions;
- a Site Plan showing work areas and test locations;
- descriptions and locations of ongoing construction;
- equipment and personnel in each work area, including subcontractors;
- descriptions and specific locations of areas, or units, of work being tested and/or observed and documented;
- locations where tests and samples were taken;
- a summary of test results;
- calibrations or recalibrations of test equipment, and actions taken as a result of recalibration;
- delivery schedule of off-site materials received, including quality control documentation;
- decisions made regarding acceptance of units of work, and/or corrective actions to be taken in instances of substandard testing results; and
- signature of the respective Site CQA Manager(s) and/or the Field Monitor(s).

In any case, all logs must be completely filled out with no items left blank.

2.2.2 Construction Problems

The Project Manager will be made aware of any significant recurring nonconformance with the construction plans, project specifications or CQA Plan. The cause of the nonconformance will be determined and appropriate changes in procedures or specifications will be recommended. These changes will be submitted to the Engineer for approval. When this type of evaluation is made, the results will be documented, and any revision to procedures or specifications will be approved by the City and Engineer.

A summary of all supporting data sheets, along with final testing results and the respective Site CQA Manager's approval of the work, will be required upon completion of construction.

2.3 Photographic Reporting

Photographs will serve as a pictorial record of work progress, problems, and mitigation activities. The primary project file will contain color prints; negatives will also be stored in a separate file. These records will be presented to the Project Manager upon completion of the project.

2.4 Design and/or Specifications Changes

Design and/or specifications changes may be required during construction. In such cases, the respective Site CQA Manager will notify the Project Manager.

Design and/or specifications changes will be made only with the written agreement of the Project Manager and the Engineer, and will take the form of an amendment to the specifications.

2.5 Final Report

At the completion of the work, the Soils and Geosynthetic CQA Representatives will submit to the Project Manager a signed and sealed final report. These reports will acknowledge: (i) that the work has been performed in compliance with the plans and specifications; (ii) physical sampling and testing has been conducted at the appropriate frequencies; and (iii) that the summary document provides the necessary supporting information.

At a minimum, this report will include:

- summaries of all construction activities;
- monitoring logs and testing data sheets including sample location plans;
- construction problems and solutions summary sheets;
- changes from design and material specifications;
- record drawings; and
- a summary statement indicating compliance with project plans and specifications which is signed and sealed by a Registered Civil Engineer or Certified Engineering Geologist in the State of California.

The record drawings will include scale drawings depicting the location of the construction and details pertaining to the extent of construction (e.g., depths, plan dimensions, elevations, soil component thicknesses, etc.). These documents will be prepared by the appropriate CQA Representative and included as part of the CQA plan documentation.

3. VERY FLEXIBLE POLYETHYLENE (VFPE) GEOMEMBRANE QUALITY ASSURANCE

3.1 Design

A copy of the VFPE geomembrane construction drawings and specifications prepared by the Engineer will be given to the Geosynthetics CQA Representative. The Geosynthetics CQA Representative will review these items for familiarity. This review should not be considered as the peer review of the design. Peer review should have been conducted at an earlier stage.

3.2 Manufacturing

The VFPE Geomembrane Manufacturer (Manufacturer) will provide the Project Manager with a list of guaranteed "minimum average roll value" properties for the type of geomembrane to be delivered. The Manufacturer will also provide the Project Manager with a written certification signed by a responsible representative of the Manufacturer that the materials actually delivered have "minimum average roll value" properties which meet or exceed all certified property values for that type of geomembrane.

The Manufacturer will also provide the Project Manager with the following information:

- the origin (Resin Supplier's name and resin production plant), identification (brand name, lot number), and production date of the resin; and
- a copy of the quality control certificates issued by the Resin Supplier.

The Geosynthetics CQA Representative will examine all of the Manufacturer and resin suppliers certificates to ensure that the property values listed on the certifications meet or exceed those specified. Any deviations will be reported to the Project Manager.

3.3 Shipment and Storage

During shipment and storage, the VFPE geomembrane will be protected from puncture, cutting, or any other damaging or deleterious conditions. The Geosynthetics CQA Representative will observe rolls upon delivery to the site and any deviations from the above requirements will be reported to the Project Manager. Any damaged rolls will be rejected and replaced at no cost to the City.

3.4 Conformance Testing

3.4.1 Testing Procedures

In order to ensure that the VFPE to be installed for this project meets the design requirements, a minimum Design Yield Point is specified. For the purpose of these specifications, the Design Yield Point is defined as the point on the stress-strain curve at which the tangent modulus first becomes 290 psi. The attached appendix provides supporting documentation used to establish this minimum design yield point. The stress-strain curve will be determined based on testing method ASTM D 638.

The following test procedures will also be conducted:

- thickness (ASTM D 751 with conical tip);
- specific gravity (ASTM D 792 Method A or ASTM D 1505);
- carbon black content (ASTM D 1603); and
- carbon black dispersion (ASTM D 5596).

Where optional procedures are noted in the test method, the requirements of the specifications shall prevail.

3.4.2 Sampling Procedures

Upon delivery of the geomembrane rolls, the Geosynthetics CQA Representative will ensure that samples are obtained from individual rolls at the frequency specified in this CQA plan. The samples will be forwarded to the

Geosynthetics CQA Laboratory for testing to ensure conformance to both the design specifications and the list of physical properties certified by the Manufacturer.

Samples will be taken across the entire width of the roll and will not include the first lineal 3 ft (1 m). Unless otherwise specified, samples will be 3 ft (1 m) long by the roll width. The Geosynthetics CQA Representative will indicate the machine direction on the samples by marking an arrow on each sample.

Unless otherwise specified, conformance samples of the VFPE geomembrane rolls will be taken at a frequency of one sample per lot or one per 100,000 ft^2 (10,000 m^2) of material delivered to the site, whichever requires the greater number of samples.

3.4.3 Test Results

The Geosynthetics CQA Representative will examine all results from laboratory conformance testing and compare results to the project specifications. The criteria used to determine acceptability are presented in the Specifications. The Geosynthetics CQA Representative will report any nonconformance to the Project Manager.

3.5 Handling and Placement

Transportation of the geomembrane is the responsibility of the Manufacturer, Installer, or other party as agreed upon. All handling on site is the responsibility of the Installer.

During the installation, the Geosynthetics CQA Representative will verify that:

- handling equipment used on the site is adequate to handle the geomembrane without causing damage to the geomembrane; and

- the Installer's personnel handle the geomembrane with care.

Upon delivery at the site, the Installer and the Geosynthetics CQA Representative will, to the best of his or her ability, conduct a surface observation of all rolls or factory panels for defects and damage. This examination will be conducted without unrolling each individual roll unless an above average frequency of defects or damage is observed or suspected. The Geosynthetics CQA Representative will report to the Project Manager:

- any rolls or portions thereof, which should be rejected and removed from the site because they have severe manufacturing defects or damage; and

- any rolls which exhibit an average occurrence of manufacturing defects or damage which are considered by the Geosynthetics CQA Representative as repairable flaws.

3.6 Storage

The Installer will be responsible for the storage of the geomembrane on site. The Project Manager will designate storage space in a location (or several locations) such that on-site transportation and handling are optimized if possible. Storage space should be protected from theft, vandalism, passage of vehicles, stormwater runon, etc. The storage space, if unpaved, should be graded and rolled smooth in order to protect the geomembrane materials from puncture.

The Geosynthetics CQA Representative will verify that storage of the geomembrane ensures adequate protection against dirt and sources of damage.

3.7 Geomembrane Installation

3.7.1 Surface Preparation

The Earthwork Contractor will be responsible for preparing the soil subbase which supports the geomembrane materials according to the Engineer's specifications.

The Geosynthetics CQA Representative will verify that:

- a qualified geotechnical engineer, normally the Soils CQA Representative, has verified that the supporting soil meets maximum dry density and moisture specifications (if applicable);

- the surface to be lined has been rolled and compacted so as to be free of irregularities, ruts, protrusions, loose soil, and abrupt changes in grade;

- the surface of the supporting soil does not contain angular to subangular stones, debris, or other objects which may damage the geomembrane; and

- there is no area of the supporting soils excessively softened by high moisture content.

The Installer will certify in writing that the surface on which the geomembrane will be installed is acceptable. The certificate of subgrade acceptance for the area under consideration will be given by the Installer to the Project Manager prior to commencement of geomembrane installation. The Geosynthetics CQA Representative will be furnished a copy of this certificate by the Project Manager.

After the supporting soil has been accepted by the Installer, it will be the Installer's responsibility to indicate to the Project Manager any change in the supporting soil condition that may require repair work. If the Geosynthetics CQA Representative and/or Soils CQA Representative concurs with the Installer assessment of the subgrade damage, then the Project Manager will ensure that the supporting soil is repaired.

3.7.2 Geomembrane Placement

3.7.2.1 Field Panel Identification

A field panel is the unit area of geomembrane which is to be seamed in the field (i.e., a field panel is a roll or a portion of roll cut in the field).

It will be the responsibility of the Geosynthetics CQA Representative to ensure that each field panel is given an "identification code" (number or letter-number) which may or may not be consistent with the Installer's proposed layout plan. This identification code will be agreed upon by the Project Manager, Installer, and Geosynthetics CQA Representative. This field panel identification code should be as simple and logical as possible. (Note: roll numbers established in the manufacturing plant are usually cumbersome and are not related to location in the field.) It will be the responsibility of the Installer to ensure that each field panel placed is marked with the original roll number. The roll number will be marked at a location agreed upon by the Project Manager, Installer, and Geosynthetics CQA Representative. The Geosynthetics CQA Representative will record the identification code, dimensions, weather conditions, time, location, and date of installation for each field panel.

The Geosynthetics CQA Representative will establish a table or chart showing correspondence between roll numbers, factory panels, and field panel identification codes. The field panel identification code will be used for all requisite quality assurance documentation.

3.7.2.2 Field Panel Placement

The Geosynthetics CQA Representative will verify that field panels are installed in the manner indicated in the geomembrane seam layout plan, as approved or modified.

Field panels will be placed one at a time, and each field panel will be seamed immediately after its placement (in order to minimize the number of unseamed field panels exposed to wind).

Geomembrane placement will not proceed at an ambient temperature below 40°F (5°C) or above 100°F (38°C) unless otherwise authorized by the Project Manager. Geomembrane placement will not be conducted during precipitation events, in an area of ponded water, or in the presence of excessive winds as determined by the Geosynthetics CQA Representative or Project Manager. The Geosynthetics CQA Representative will verify that the above conditions are fulfilled. The Geosynthetics Site CQA Manager will inform the Project Manager if the above conditions are not fulfilled.

The Geosynthetics CQA Representative will visually observe each panel, after placement and prior to seaming, for damage. The Geosynthetics Site CQA Manager will advise the Project Manager which panels, or portions of panels, should be rejected, repaired, or accepted. Damaged panels or portions of damaged panels which have been rejected will be marked and their removal from the work area recorded by the Geosynthetics CQA Representative. Repairs will be made according to procedures described in Section 3.7.4.

3.7.3 Field Seaming

3.7.3.1 Seam Layout

The Installer will provide the Project Manager and the Geosynthetics CQA Representative with a seam layout drawing, i.e., a drawing of the facility to be lined showing all expected seams. The Geosynthetics CQA Representative will review the seam layout drawing and verify that it is consistent with the accepted state-of-practice and this CQA Plan. Seams not specifically shown on the seam layout drawing may not be constructed without the Project Manager's prior approval. A seam numbering system compatible with the panel numbering system will be agreed upon at the Resolution and/or Pre-Construction Meeting.

3.7.3.2 Seaming Equipment and Products

Approved field seaming processes are fillet extrusion seaming and double-track fusion seaming. Proposed alternate processes will be documented and submitted to the Project Manager for approval. Only seaming apparatus which have been specifically approved by make and model will be used. The Installer will ensure that all seaming equipment used on this project are in good working order including accurate temperature gauging devices.

The Project Manager will submit all seaming documentation provided by the Installer to the Geosynthetics CQA Representative for his concurrence.

Extrusion Process

The extrusion seaming apparatus will be equipped with gauges giving the relevant temperatures of the apparatus such as the temperatures of the extrudate, nozzle, and preheat. The Installer will verify equipment operating temperature with a pyrometer to ensure that accurate temperatures are being achieved throughout the course of the geomembrane installation.

The Geosynthetics CQA Representative will record machine operating temperatures, extrudate temperatures, and ambient temperatures at appropriate intervals. Ambient temperatures will be measured approximately 6 in. (150 mm) above the geomembrane surface.

Fusion Process

The fusion-seaming apparatus must be automated vehicular-mounted devices. The fusion-seaming apparatus will be equipped with gauges indicating operating temperatures. Pinch roller pressure settings will be adjusted by the Installer as required.

The Geosynthetics CQA Representative will record ambient temperatures, seaming apparatus temperatures, and speeds. Ambient temperatures will be measured approximately 6 in. (150 mm) above the geomembrane surface.

3.7.3.3 Seam Preparation

The Geosynthetics CQA Representative will monitor the preparation of the geomembrane for seaming operations to assure that:

- prior to seaming, the seam area is clean and free of moisture, dust, dirt, debris of any kind, and foreign material;
- if seam overlap grinding is required, the process is completed according to the Geomembrane Manufacturer's instructions within one hour of the seaming operation, and in a way that does not damage the geomembrane;
- the abrading does not extend more than 0.5 in. (12 mm) on either side of the extruded weld; and
- seams are aligned to minimize the number of wrinkles and "fishmouths."

The Geosynthetics Site CQA Manager will inform the Project Manager if the conditions identified above are not met.

3.7.3.4 Trial Seams

Trial seams will be made using extraneous pieces of VFPE geomembrane to verify that seaming conditions are adequate. Such trial seams will be made at the beginning of each seaming period, and at least once every five hours, for both fusion and extrusion seaming apparatus used during the seaming period. A trial seam will also be made in the event that the ambient temperature varies more than 18°F (10°C) since the last passing trial seam test. The ambient temperature will be measured approximately 6 in. (150 mm) above the liner. Also, each seaming technician will make at least one trial seam for each seaming period. Trial seams will be made under the same conditions as actual seams. If any seaming apparatus is turned off for any reason, a new passing trial seam must be completed for that specific seaming apparatus.

If a trial seam specimen fails according to the criteria identified in the project specifications, the entire trial seam testing operation should be repeated. If a specimen fails in the subsequent testing, the seaming apparatus and seamer will not be accepted and will not be used for seaming until the deficiencies are corrected and two consecutive successful full trial seams are achieved.

Additional testing of trial seams may be conducted if agreed upon between the parties involved. Any such agreements will be documented by the Geosynthetics CQA Representative. After completion of the testing described above, the remainder of the trial seam sample may be cut into three pieces and distributed, one to be retained in the City's archives, one to be given to the Installer, and one to be provided to the Geosynthetics CQA Laboratory for the additional testing, as required. If a trial seam sample fails a test conducted by the Geosynthetics CQA Laboratory, then a destructive sample will be taken from each of the seams completed by the seaming technician and apparatus subsequent to the successful field trial seam test. The conditions of this paragraph will be considered as met for a given seam if a corresponding destructive sample has already been taken and meet or exceed the requirements of the project specifications and this CQA plan.

3.7.3.5 Nondestructive Testing

Concept

The Installer will nondestructively test all field seams over their full length using a vacuum test, spark test, air pressure test (for double-track fusion seams only), or other approved method. Vacuum testing and air pressure testing are described in the *Vacuum Testing* and the *Air Pressure Testing* of this section, respectively. The purpose of nondestructive tests is to check the continuity of seams. It does not provide any information on seam strength. Nondestructive testing will be carried out as the seaming work progresses, not at the completion of all field seaming. Nondestructive testing will not be permitted without adequate illumination unless the Installer demonstrates capabilities to do so to the satisfaction of the Project Manager.

The Geosynthetics CQA Representative will:

- observe all nondestructive testing;

- record location, date, test unit number, name of tester, and outcome of all testing; and

- inform the Installer and Project Manager of any required repairs.

The Installer will complete any required repairs in accordance with Section 3.7.4.

In some cases, seams may be inaccessible for nondestructive testing due to the design of the closure system. Provisions may be made to prefabricate portions of the geomembrane to allow nondestructive testing of seams that would otherwise be inaccessible. Once tested, the prefabricated portions may be installed. In those cases where no provisions can be made to nondestructively test a seam, the seam must be capped following the method described in Section 3.7.4.3. The seaming and capping operation will be observed by the Geosynthetics CQA Representative for uniformity and completeness.

The seam number, date of observation, name of tester, and outcome of the test or observation will be recorded by the Geosynthetics CQA Representative.

Vacuum Testing

The equipment for seam vacuum testing will consist of the following:

- a vacuum box assembly consisting of a rigid housing, a transparent viewing window, a soft neoprene gasket attached to the bottom, port hole or valve assembly, and a vacuum gauge;

- a vacuum tank and pump assembly equipped with a pressure controller and pipe connections;

- a pressure/vacuum hose with fittings and connections;
- an approved applicator; and
- a soapy solution.

The following procedures will be followed:

- if vacuum testing a fusion seam, the flap must be removed prior to testing;
- energize the vacuum pump to maintain a tank pressure of approximately 5 psi (34 kPa) gauge;
- with a soapy solution, wet a strip of geomembrane which is 6 in. (150 mm) larger in area than the vacuum box;
- place the box over the wetted area;
- close the bleed valve and open the vacuum valve;
- ensure that a leak tight seal is created;
- for a period of not less than 10 seconds, examine the geomembrane seam through the viewing window for the presence of leaks indicated by soap bubbles;
- if no leak indications appear after 10 seconds, close the vacuum valve and open the bleed valve. Before moving the box over the next adjoining area, place a mark (with an approved marker) on the geomembrane at the leading edge of the viewing window, then move the box over the next adjoining area so that the last mark on the geomembrane is at the rear of the viewing window, and repeat the process; and

- all areas where leaks appear will be marked by the vacuum testing technician and repaired by the Installer in accordance with Section 3.7.4.3.

Air Pressure Testing (For Double-Track Fusion Seams Only)

The following procedures are applicable to those processes which produce a double seam with an enclosed air channel space.

The equipment will be comprised of the following:

- an air pump equipped with a pressure gauge capable of generating and sustaining a pressure between 25 to 30 psi (175 and 210 kPa) and mounted on a cushion to protect the geomembrane;

- a hose with fittings and connections; and

- a sharp hollow needle, or other approved air pressure feed device and pressure gauge.

The following procedures will be followed:

- insert a protective cushion between the air pump and the geomembrane;

- seal both ends of the seam to be tested;

- insert the needle or other approved pressure feed device into the channel created by the fusion seam;

- insert the needle with the pressure gauge into the channel at the opposite end of the seam where the pressure feed device is located;

- energize the air pump to a pressure between 25 and 30 psi (175 and 210 kPa), close the valve, and sustain the pressure for a minimum period of 5 minutes;

- if any loss of pressure exceeds 2 psi (15 kPa) on the gauge at the opposite end of the seam to the pressure feed device or if the pressure does not stabilize, locate the faulty area and repair it in accordance with Section 5.8.4.3;

- verify the relief of the air pressure of the end of the seam opposite the pressure gauge; and

- remove the needles or other approved pressure feed devices and repair all holes created during the test procedures.

3.7.3.6 Destructive Testing

Concept

Destructive seam tests will be performed at selected locations. The purpose of these tests is to evaluate seam strength. Seam strength testing will be conducted as the seaming work progresses, not at the completion of production seaming.

Location and Frequency

The Geosynthetics Site CQA Manager will select locations where seam samples will be cut out for laboratory testing. Those locations will be established as follows:

- A minimum average frequency of one test per 500 lineal ft (150 lineal m) of seam length. This minimum frequency is to be determined as an average taken over the total length of the geomembrane seams constructed for the final cover system.

- A maximum frequency will be agreed upon by the Installer, Project Manager and Geosynthetics Site CQA Manager at the Resolution and/or Pre-Construction Meeting.

- Test locations will be determined during seaming at the Geosynthetics Site CQA Manager's discretion. Selection of such locations may be prompted by suspicion of excess crystallinity, contamination, offset seams, or any other potential cause of inadequate seaming.

The Installer will not be informed in advance of the locations where the seam samples will be taken.

Sampling Procedure

Samples will be marked by the Geosynthetic CQA Representative and removed by the Installer for field and laboratory testing as the seaming progresses. This procedure will allow review of laboratory test results before the geomembrane is covered by another material. The Geosynthetics CQA Representative will:

- observe sample removal;

- assign a number to each sampling location, and mark the sample removed from that location accordingly;

- record the sample location on the layout drawing; and

- record the reason for taking the sample at this location (e.g., statistical routine, suspicious feature of the geomembrane).

All holes in the geomembrane resulting from the destructive sampling procedures will be immediately repaired by the Installer in accordance with repair procedures described in Section 3.7.4.3. The continuity of the new seams constructed as part of the repaired area will be tested according to the *Vacuum Testing* of Section 3.7.3.5.

Prior to the removal of a sample, two specimens for field testing should be taken. Each of these specimens will be 1 in. (25 mm) wide by 8 in. (200 mm) long, with the seam centered parallel to the width. The distance between these two specimens will be 44 in. (1.1 m). If both specimens pass the field peel tests described in the *Field Testing* of Section 3.7.3.6, a sample for laboratory testing will be taken. If either specimen fails the testing, the seam should be repaired in accordance with the procedures identified in Section 3.7.4.3.

Size and Distribution of Samples

The sample for laboratory testing will be located between the two specimens removed for field testing as described in the *Sampling Procedure* of Section 3.7.3.6. The destructive sample will be 12 in. (0.3 m) wide by 42 in. (1.1 m) long with the seam centered lengthwise. The sample will be cut into three parts and distributed as follows:

- one portion, measuring 12 in. x 12 in. (0.3 m x 0.3 m), to the Installer for laboratory testing (if required);
- one portion, measuring 12 in. x 12 in. (0.3 m x 0.3 m), to the City for archive storage; and
- one portion, measuring 12 in. x 18 in. (0.3 m x 0.45 m), for Geosynthetic CQA Laboratory testing.

Final determination of the destructive sample dimensions and distribution will be made at the Pre-Construction Meeting.

Field Testing

The two 1 in. (25 mm) wide specimens mentioned in the *Sampling Procedure* of Section 3.7.3.6 will be tested in the field for peel. The testing will be conducted using a gauged tensiometer which has been calibrated within the last six months. If any field test sample fails to pass the criteria identified in the specifications, then the procedures outlined in the *Procedures for Destructive Test Failures* of Section 3.7.3.6 will be followed.

The Geosynthetics CQA Representative will witness all field destructive testing and record the date, seam number, panel numbers, location, the assigned destructive sample number, and the results of the field tests.

Geosynthetics Construction Quality Assurance Laboratory Testing

Destructive test samples will be packaged and shipped, if necessary, by the Geosynthetics CQA Representative in a manner that will not damage the test sample. The Project Manager will verify that packaging and shipping conditions are acceptable. The Project Manager will be responsible for storing the archive samples. This procedure will be fully outlined at the Resolution and Pre-Construction Meetings. Destructive samples will be tested by the Geosynthetics CQA Laboratory. The Geosynthetics CQA Laboratory will be selected by the Geosynthetics CQA Representative with the concurrence of the City.

Testing will include "Seam Strength" (ASTM D 4437 as modified in NSF 54 Appendix A), and "Peel Strength" (ASTM D 4437 as modified in NSF 54, Appendix A). Modifications to the testing procedures and the minimum acceptable values to be obtained in these tests are indicated in the Specifications. At least five specimens will be tested for each test method. Specimens will be selected alternately by test from the samples (i.e., peel, shear, peel, shear...).

The Geosynthetics CQA Laboratory will provide test results to the Geosynthetic Site CQA Manager no more than 24 hours after receipt of the samples. The Geosynthetics Site CQA Manager will review laboratory test results as soon as they become available and make appropriate recommendations to the Project Manager.

Acceptable seams must be bounded by two locations which meet the following criteria: (i) where destructive samples have passed all laboratory tests; (ii) the entire production seam length and seaming apparatus in question is capped; and (iii) constructed by the seamer. Whenever a reconstructed seam length exceeds 150 ft (50 m), a sample will be taken from the zone in which the seam has been reconstructed. This sample must pass destructive testing or the procedure outlined in this section must be repeated.

The Geosynthetics CQA Representative will document all actions taken in conjunction with destructive test failures.

3.7.4 Defects and Repairs

3.7.4.1 Identification

Seams and non-seam areas of the geomembrane will be examined by the Geosynthetics CQA Representative for identification of defects, holes, blisters, undispersed raw materials and any sign of contamination by foreign matter. The surface of the geomembrane will be clean at the time of examination. The geomembrane surface will be swept or washed by the Installer if debris of any kind inhibits examination.

3.7.4.2 Evaluation

Each suspect location both in seam and non-seam areas will be nondestructively tested using the methods described in the *Vacuum Testing* of Section 3.7.3.5. Each location which fails the nondestructive testing will be marked by the Installer or the Geosynthetics CQA Representative and repaired by the Installer. Work will not proceed with any materials which will cover geomembrane locations that have been repaired until laboratory destructive test results have been approved by the Geosynthetic CQA Representative.

3.7.4.3 Repair Procedures

Any portion of the geomembrane exhibiting a flaw or failing a destructive or nondestructive test will be repaired. Several procedures exist for the repair of these areas. The final decision as to the appropriate repair procedure will be agreed upon between the Project Manager, Installer, and Geosynthetics Site CQA Manager. The procedures available include:

- patching, used to repair large holes, tears, undispersed raw materials, and contamination by foreign matter;
- grinding and reseaming, used to repair small sections, less than 1 ft (0.3 m) of extruded seams;
- spot seaming, used to repair small tears, pinholes, or other minor, localized flaws; and
- capping, used to repair failed seams.

In addition, the following provisions will be satisfied:

- surfaces of the geomembrane that are to be repaired will be abraded no more than one hour prior to the repair;
- all surfaces must be clean and dry at the time of the repair;
- all seaming equipment used in repairing procedures must have passed the most recent seaming periods of trial seam testing;
- the repair procedures, materials, and techniques will be approved in advance of the specific repair by the Project Manager, Geosynthetic Site CQA Manager, and Installer;
- patches or caps will extend at least 6 in. (150 mm) beyond the edge of the defect, and all corners of patches will be rounded with a radius of at least 3 in. (75 mm); and
- the geomembrane below large caps should be appropriately cut to avoid water or gas collection between the two sheets.

3.7.5 Geosynthetic Final Cover System Acceptance

The Installer will retain all responsibility for the installed geosynthetics until accepted by the City.

The installed geosynthetics will be accepted by the City when:

- the installation is finished;
- verification of the adequacy of all seams and repairs, including passing nondestructive and destructive tests, are complete;
- Installer's representative furnishes the Project Manager with certification that the VFPE geomembrane was installed in accordance with the Manufacturer's recommendations as well as the plans and specifications;
- all documentation of installation is completed; and
- the Geosynthetics CQA Representative's Final Report and Record Drawings, sealed by a Professional Engineer registered by the State of Illinois, have been received by the City.

4. GEOTEXTILE CONSTRUCTION QUALITY ASSURANCE

4.1 Design

A copy of the geotextile construction drawings and project specifications prepared by the Engineer will be given to the Geosynthetic CQA Representative. The Geosynthetic CQA Representative will review these items for familiarity. This review should not be considered as the peer review of the design. Peer review should have been conducted at an earlier stage.

4.2 Manufacturing

The Geotextile Manufacturer (Manufacturer) will provide the Project Manager with a list of certified "minimum average roll value" properties for the type of geotextile to be delivered. The Manufacturer will also provide the Project Manager with a written certification signed by a responsible representative of the Manufacturer that the materials actually delivered have "minimum average roll values" properties which meet or exceed all certified property values for that type of geotextile.

The Geosynthetic CQA Representative will examine all the Manufacturers' certifications to ensure that the property values listed on the certifications meet or exceed those specified for the particular type of geotextile. Any deviations will be reported to the Project Manager.

4.3 Labeling

The Manufacturer will identify all rolls of geotextile with the following:

- Geotextile Manufacturer's name;
- product identification;
- lot number;
- roll number;
- roll weight; and
- roll dimensions.

The Geosynthetic CQA Representative will examine rolls upon delivery and any deviation from the above requirements will be reported to the Project Manager.

4.4 Shipment and Storage

During shipment and storage, the geotextile will be protected from ultraviolet light exposure, precipitation or other inundation, mud, dirt, dust, puncture, cutting or any other damaging or deleterious conditions. To that effect, geotextile rolls will be shipped and stored in relatively opaque and watertight wrappings. The Geosynthetic CQA Representative will observe rolls upon delivery to the site and any deviation from the above requirements will be reported to the Project Manager. Any damaged rolls will be rejected and replaced at no cost to the Owner.

4.5 Conformance Testing

4.5.1 Tests

Upon delivery of the geotextile rolls, the Geosynthetic CQA Representative will ensure that samples are removed and forwarded to the Geosynthetic CQA Laboratory for testing to ensure conformance to both the design specifications and the list of guaranteed properties.

As a minimum, the following tests will be performed on geotextiles in accordance with the referenced ASTM Standards:

- mass per unit area (ASTM D 3776);
- grab strength (ASTM D 4632);
- tear strength (ASTM D 4533);
- burst strength (ASTM D 3786); and
- puncture strength (ASTM D 3787).

4.5.2 Sampling Procedures

Upon delivery of the geotextile rolls, the Geosynthetics CQA Representative will ensure that samples are obtained from individual rolls at the frequency specified in this CQA plan. The samples will be forwarded to the Geosynthetics CQA Laboratory for testing to ensure conformance to both the design specifications and the list of physical properties certified by the Manufacturer.

Samples will be taken across the entire width of the roll and will not include the first linear 3 ft (1 m). Unless otherwise specified, samples will be 3 ft (1 m) long by the roll width. The Geosynthetic CQA Representative will mark the machine direction on the samples with an arrow. Samples will be taken at a rate of one per manufactured lot or one per 100,000 ft^2 (9,300 m^2), whichever requires the greater number of samples.

4.5.3 Test Results

The Geosynthetic CQA Representative will examine all results from laboratory conformance testing and compare results to the project specifications. The criteria used to determine acceptability are presented in the Specifications. The Geosynthetic CQA Representative will report any nonconformance to the Project Manager.

4.5.4 Conformance Test Failure

The following procedure will apply whenever a sample fails a conformance test that is conducted by the Geosynthetic CQA Laboratory:

- The Manufacturer will replace every roll of geotextile that is in nonconformance with the specifications with a roll that meets specifications.
- The Installer will remove conformance samples for testing by the Geosynthetic CQA Laboratory from the closest numerical rolls on

both sides of the failed roll. These two samples must conform to the specifications. If either of these samples fail, the numerically closest rolls on the side of the failed sample that is not tested, will be tested by the Geotextile CQA Laboratory. These samples must conform to the specifications. If any of these samples fail, every roll of geotextile on site from this lot and every subsequently delivered roll that is from the same lot must be tested by the Geosynthetic CQA Laboratory for conformance to the specifications. This additional conformance testing will be at the expense of the Manufacturer.

The Geosynthetics CQA Representative will document actions taken in conjunction with conformance test failures.

4.6 <u>Handling and Placement</u>

The Installer will handle all geotextiles in such a manner as to ensure they are not damaged in any way. The Installer will comply with the following:

- In the presence of wind, the geotextile will be weighted with sandbags or the equivalent. Sandbags will be used during installation only and will remain until replaced with the appropriate protective cover soils.

- The geotextile will be kept continually under tension to minimize the presence of wrinkles in the geotextile.

- The geotextile will be cut using an approved geotextile cutter only. If in place, special care must be taken to protect other materials from damage which could be caused by the cutting of the geotextile.

- The Installer will take any necessary precautions to prevent damage to the underlying VFPE geomembrane during placement of the geotextile.

- During placement of geotextile, care will be taken not to entrap stones, excessive dust, or moisture that could damage the geotextile, cause clogging, or hamper subsequent seaming.

- A visual examination of the geotextile will be carried out over the entire surface, after installation to ensure that no potentially harmful foreign objects are present.

The Geosynthetics CQA Representative will note any noncompliance and report it to the Project Manager.

4.7 Geotextile Seams and Overlaps

All geotextile seams will be sewn using thread approved by the Manufacturer and which is resistant to ultraviolet radiation. Spot sewing is not permitted. Thermal bonding is not permitted without written approval of the Engineer. Geotextiles shall be overlapped a minimum of 6 in. (150 mm) prior to seaming. No horizontal seams will be allowed on side slopes steeper than 20 percent (i.e. seams will be along, not across, slopes steeper than 5H:1V), except as part of a patch or for seams connecting the ends of two panels of geotextile deployed parallel to the slope (referred to as cross seams). Cross seams shall not be continuous across two or more panel widths.

4.8 Geotextile Repair

Any holes or tears in the geotextile will be repaired using a patch made from the same geotextile. Geotextile patches will extend a minimum of 1 ft (0.3 m) beyond the damaged area. Geotextile patches will be sewn into place no closer than 1 in. (25 mm) from any panel edge. Should any tear exceed 50 percent of the width of the roll, that roll will be removed from the slope and replaced. Care will be taken to remove any soil or other material which may have penetrated the torn geotextile.

The Geosynthetic CQA Representative will observe any repair, note any noncompliance with the above requirements and report them to the Project Manager.

4.9 Placement of Soil Materials

The Earthwork Contractor will place all soil materials located on top of a geotextile in such a manner as to ensure:

- no damage to the geotextile;
- minimal slippage of the geotextile on underlying layers; and
- no excess tensile stresses in the geotextile.

Any noncompliance will be noted by the Geosynthetic CQA Representative and reported to the Project Manager.

5. SOILS CONSTRUCTION QUALITY ASSURANCE

Soils CQA will be performed on all soil components used during construction of the final cover. The criteria to be used for the determination of acceptability of the construction work will be as identified in Tables 5-1 and 5-2.

5.1 Monitoring

The Soils CQA Consultant will monitor and document the construction of all soils components. Monitoring the construction work includes the following:

- monitoring the quality of the material stockpiles, obtaining borrow soil samples for conformance testing;
- testing to determine the moisture content and unit weight of each lift during placement and compaction of soil used in construction of the foundation, low-permeability soil barrier, and vegetative layers;
- recording test results and locations;
- noting any deficiencies;
- monitoring the thickness of lifts as loosely placed and as compacted;
- monitoring that the total thickness of the foundation, low-permeability soil barrier, and vegetative layers is as indicated on the construction plans;

- monitoring the action of the compaction and heavy hauling equipment on the construction surface (i.e., penetration, pumping, cracking, etc.); and

- monitoring the repair of nonconforming areas and testing perforations.

Monitoring the earthwork for the foundation layer specifically includes the following:

- monitor clearing, grubbing, and stripping of the existing interim cover surface;

- monitor the scarification of the interim cover surface to a depth of 6 to 8 in. (150 to 200 mm) and recompaction;

- reviewing documentation of quality control test results;

- visually monitoring the physical condition of the material during placement; and

- visually monitoring the foundation layer stability under the action of the compaction equipment.

Monitoring the earthwork for the compacted low-permeability soil barrier layer specifically includes the following:

- reviewing documentation of the quality control test results;

- monitoring the soil for deleterious material;

- monitoring moisture conditioning and preprocessing, if any, of the borrow soil material;

- monitoring the thickness of lifts during placement of the material;

- monitoring that the surface of each lift is scarified to a depth of 2 to 4 in. (50 to 100 mm) prior to placement of the following lift;

- recording the construction equipment used for material placement;

- performing BAT hydraulic conductivity tests and recording the test results and location;

- monitoring the protection of the final surface of the low-permeability soil barrier layer from excessive moisture loss prior to placement of the vegetative cover layer; and

- monitoring preparation and smoothness of the surface prior to the installation of the VLDPE geomembrane in 'C' Canyon.

Monitoring the earthwork for the vegetative layer specifically includes the following:

- reviewing documentation of the quality control test results;

- monitoring soil for deleterious material;

- monitoring the thickness of lifts during placement of the materials;

- monitoring wrinkles that may appear in the underlying geotextile cushion on VLDPE geomembrane during placement of the vegetative layer in 'C' Canyon; and

- recording field density and field moisture content measurement at location of each test on test logs.

5.2 Laboratory and Field Tests

The laboratory and field test methods, laboratory and field testing frequencies, and criteria used to determine acceptability are presented in Table 5-1. A special testing frequency will be used at the discretion of the Landfill Engineer or the Soils CQA Consultant when visual observations of construction performance indicate a potential or recurring deficiency.

5.3 Survey

The top of the low-permeability soil barrier shall be surveyed before the installation of the immediately overlying vegetative cover layer. The thickness of the low-permeability soil barrier shall be determined by comparing the survey of the finished foundation layer and the top of the low-permeability soil barrier layer.

5.4 Deficiencies

5.4.1 General

If a defect is discovered in the earthwork product, the Soils Site Monitor will immediately inform the Soils CQA Managing Engineer or his designated representative. The Soils Site Monitor, in consultation with the Soils CQA Managing Engineer, will determine the extent and nature of the defect. If the defect is indicated by an unsatisfactory test result, extent of the deficient area will be determined by additional tests, observations, a review of records, or other means that the Soils CQA Managing Engineer deems appropriate.

If the defect is related to adverse site conditions, such as overly wet soils or surface desiccation, the Soils Site Monitor, in consultation with the Soils CQA Managing Engineer, will define the limits and nature of the defect.

5.4.2 Notification

After determining the extent and nature of a defect, the Soils CQA Site Manager will notify the Landfill Engineer and Landfill Manager and schedule appropriate retests when the work deficiency is to be corrected.

5.4.3 Corrective Action

At locations where the field testing of the soil indicates that the compacted unit weight, moisture content, or field or laboratory hydraulic conductivities do not meet the requirements presented in Table 5-1, the failing area will be reworked as indicated below:

- If the results of any in-situ moisture or dry density, or field hydraulic conductivity value fails to meet the specified criteria presented in Table 5-1, two additional tests of the same type will be performed in the vicinity of the failed test. If either of the two additional tests results in a failure, then this area of the low-permeability soil barrier will be considered in nonconformance and will be removed, reworked, and recompacted to meet the requirements specified in Table 5-1.

- Perform in-place density and moisture content testing in the vicinity of a nonconforming area to evaluate deficiency in-place density and moisture content.

- Obtain samples of low-permeability soil liner material from nonconforming areas for potential laboratory testing to evaluate differences in soil properties that could contribute to the nonconforming test results.

Criteria to be used for determination of acceptability will be as identified herein. Other tests conducted on hydraulic conductivity samples will consist of Atterberg limits and grain size distribution.

5.4.4 Repairs and Retesting

The City's work force will correct the deficiency to the satisfaction of the Soils CQA Consultant. If a project specification criterion cannot be met, or unusual weather conditions hinder work, then the Soils CQA Consultant will develop and present to the Landfill Engineer suggested solutions for approval.

All retests recommended by the Soils CQA Consultant must verify that the defect has been corrected before any additional work is performed by the City's work force in the area of the deficiency. The Soils CQA Consultant will also verify that all installation requirements are met.

Penetrations into the compacted low-permeability soil barrier resulting from sampling or other activities shall be properly backfilled with hand-tamped select low-permeability material and/or bentonite powder. CQA personnel will repair nuclear density, sand cone, and BAT hole perforations. The City's work force shall repair perforations and/or excavations resulting from CQA sampling and testing. All repairs will be inspected by the Site Soils Monitor for compliance.

TABLE 5-1

FOUNDATION LAYER CONFORMANCE TESTING
FINAL COVER SYSTEM
LOPEZ CANYON SANITARY LANDFILL

TEST METHOD	MINIMUM TESTING FREQUENCY	ACCEPTANCE CRITERIA
Grain Size Distribution (ASTM D 422)	1 test per 10,000 yd^3 (7,650 m^3)	Maximum particle size of 6 in.
Modified Proctor (ASTM D 1557)	1 test per 10,000 yd^3 (7,650 m^3)	N/A
In-Place Moisture-Density Nuclear Method (ASTM D 2922/3017)	1 test per 1,000 yd^3 (765 m^3)	Dry density no less than 90% of the max. dry density for top 6 to 8 inches of the foundation layer, no less than 85% of the max dry density for the vegetative layer moisture content no less than the optimum moisture content, as measured by ASTM D 1557.
In-Place Moisture/Density Sand Cone Method (ASTM D 1556) or Drive Cylinder Method (ASTM D 2937)	1 test per 10,000 yd^3 (7,650 m^3)	Dry density no less than 90% of the max. dry density for the foundation layer, no less than 85% of the max dry density for the vegetative layer moisture content no less than the optimum moisture content, as measured by ASTM D 1557.

TABLE 5-2

LOW-PERMEABILITY SOIL BARRIER LAYER CONFORMANCE TESTING FINAL COVER SYSTEM LOPEZ CANYON SANITARY LANDFILL

TEST METHOD	MINIMUM TESTING FREQUENCY	ACCEPTANCE CRITERIA
Grain Size Distribution (ASTM D 422)	1 test per 4,000 yd^3 (3,060 m^3)	Minimum fines content of 50%. Maximum particle size of 3 in. (75 mm).
Atterberg Limits (ASTM D 4318)	1 test per 4,000 yd^3 (3,060 m^3)	Plasticity index: 20 minimum
In-Place Moisture/Density Nuclear Method (ASTM D 2922)	1 test per 250 yd^3 (190 m^3) Minimum of 5 tests per week	Each lift shall be compacted to at least 90 percent of the maximum dry density at +/-2% of the optimum moisture content, as measured by ASTM D 698.
In-Place Moisture/Density Sand Cone Method (ASTM D 1556) or Drive Cylinder Method (ASTM 2937)	1 test per 2,500 yd^3 (1,900 m^3) (for correlation) (minimum of 1 test per week)	Each lift shall be compacted to at least 90 percent of the maximum dry density at +/-2% of the optimum moisture content, as measured by ASTM D 698.
Moisture-Density Compaction Curve (ASTM D 698)	1 test per 5,000 yd^3 (3,820 m^3)	N/A
Field Permeability Test (BAT Permeameter, Manufacturer's Specifications)	1 test per 2,000 yd^3 (1,530 m^3)	Maximum saturated hydraulic conductivity of 1×10^{-6} cm/s.
Laboratory Permeablity Test[1] (ASTM D 5084)	1 test per 4,000 yd^3 (3,060 m^3) (on Shelby tube soil samples)	Maximum saturated hydraulic conductivity of 1×10^{-6} cm/s at a confining pressure of 3 psi.

6. GEOSYNTHETIC CLAY LINER (GCL) QUALITY ASSURANCE

During the installation of the GCL, the CQA CONSULTANT will monitor and document that material handling and storage, deployment, seaming, anchoring and protection, and repairs are in conformance with the Contract Drawings and the Technical Specifications. The Site CQA Manager will review the Geosynthetics CONTRACTOR's submittals and provide recommendations to the OWNER. Monitoring activities will be documented, as will all deviations from the Contract Drawings and the Technical Specifications, and their resolutions. Any nonconformance identified by the CQA CONSULTANT will be reported to the OWNER and the Geosynthetics Contractor. The GCL CQA activities are described in greater detail in the following sections.

6.1 Geosynthetic Clay Liner (GCL) Conformance Testing

CQA personnel will sample the GCL at the manufacturer's plant and/or after delivery to the construction site. The samples will be forwarded to the Geosynthetics CQA Laboratory for testing to assess conformance with the Technical Specifications. The test methods and minimum testing frequencies are indicated in Table 6-1.

Samples will be taken across the entire width of the roll and will not include the first 3 ft (0.9 m) if the sample is cut onsite. Unless otherwise specified, samples will be 3 ft (0.9 m) long by the roll width. The CQA CONSULTANT will mark the machine direction with an arrow and the manufacturer's roll number on each sample.

6.2 GCL Delivery and Storage

Upon delivery to the site, the CQA CONSULTANT will check the GCL rolls for defects (e.g., tears, holes) and for damage. The CQA CONSULTANT will report to OWNER and the Geosynthetics CONTRACTOR:

- any rolls, or portions thereof, which should be rejected and removed from the site because they have severe flaws; and

- any rolls which include minor repairable flaws.

The GCL rolls delivered to the site will be checked by the CQA CONSULTANT to ensure that the roll numbers correspond to those on the approved manufacturer's quality control certificate of compliance.

6.3 GCL Installation

The CQA CONSULTANT will monitor and document that the GCL is installed in accordance with the Contract Drawings and the Technical Specifications. The Geosynthetics CONTRACTOR shall provide the CQA CONSULTANT a certificate of subgrade acceptance prior to the installation of the GCL as outlined in the Technical Specifications. The GCL installation activities to be monitored and documented by the CQA CONSULTANT include:

- monitoring that the GCL rolls are stored and handled in a manner which does not result in any damage to the GCL;
- monitoring that the GCL is not exposed to UV radiation for extended periods of time without prior approval;
- monitoring that placement and compaction of soil does not cause damage, create large wrinkles, or induce excessive tensile stresses to the GCL;
- monitoring that the GCL are seamed in accordance with the Technical Specifications and the manufacturer's recommendations;
- monitoring and documenting that the GCL is installed on an approved subgrade, free of debris, protrusions, or uneven surfaces;
- monitoring that no needles are in the GCL or bentonite powder using a metal detector;

- monitoring that the GCL is not installed on a saturated subgrade or standing water and is not exposed such that it is hydrated prior to completion of the side-slope liner system; and

- monitoring that any damage to the GCL is repaired as outlined in the Technical Specifications.

TABLE 6-1
GEOSYNTHETIC CLAY LINER CONFORMANCE TESTING

PROPERTY	TEST METHOD	MINIMUM TESTING FREQUENCY
Dry Mass per Unit Area	ASTM D 3776	40,000 ft^2 (3,715 m^2) or one per lot[(2)]
Puncture Strength, Unhydrated GCL	ASTM D 4833	40,000 ft^2 (3,715 m^2) or one per lot[(2)]
Bentonite Free Swell	USP NF XVII	40,000 ft^2 (3,715 m^2) or one per lot[(2)]
Hydraulic Conductivity[(1)]	ASTM D 5084	100,000 ft^2 (9,290 m^2) or one per lot[(2)]

Notes: (1) Performed at a confining stress of 5 psi.
(2) A lot is defined as a series of consecutively numbered rolls from the same manufacturing line.

APPENDIX

M E M O R A N D U M

TO: Mr. Dan Meyers, City of Los Angeles, Bureau of Sanitation

FROM: Mr. Mike Snow, P.E., GeoSyntec Consultants

DATE: 19 September 1996

SUBJECT: **Very Flexible Polyethylene Tensile Testing Design Yield Point Concept Clarification**

This letter is an explanation clarifying the methodology used by GeoSyntec Consultants (GeoSyntec) to identify the design yield point (DYP) of very flexible polyethylene (VFPE) geomembranes (i.e., VFPE's include very low density polyethylene (VLDPE) and linear low density polyethylene (LLDPE)). A methodology to identify the design yield point of VFPE is necessary because, unlike high density polyethylene (HDPE) which has a distinct yield point (at approximately 10 to 12 percent strain), VFPE's have more of a yielding region (at approximately 10 to 20 percent strain).

The point on the VFPE tensile stress vs. tensile strain curve, where the tangent modulus first decreases to 290 psi, was selected as the DYP based on a review of a large number of test results. The corresponding stress is called the design yield stress while the corresponding strain is called the design yield strain. A typical tensile stress vs. strain curve for VLDPE is attached. The minimum values of design yield stress and strain associated with the DYP are then specified along with the other properties of the VFPE. There are no know references that present this methodology, however, it has been used previously by GeoSyntec on projects using VFPE's.

The stress-strain curve used to determine the DYP will be obtained in accordance with ASTM D 638, which is a more appropriate test method than ASTM D 882.

CLOSURE

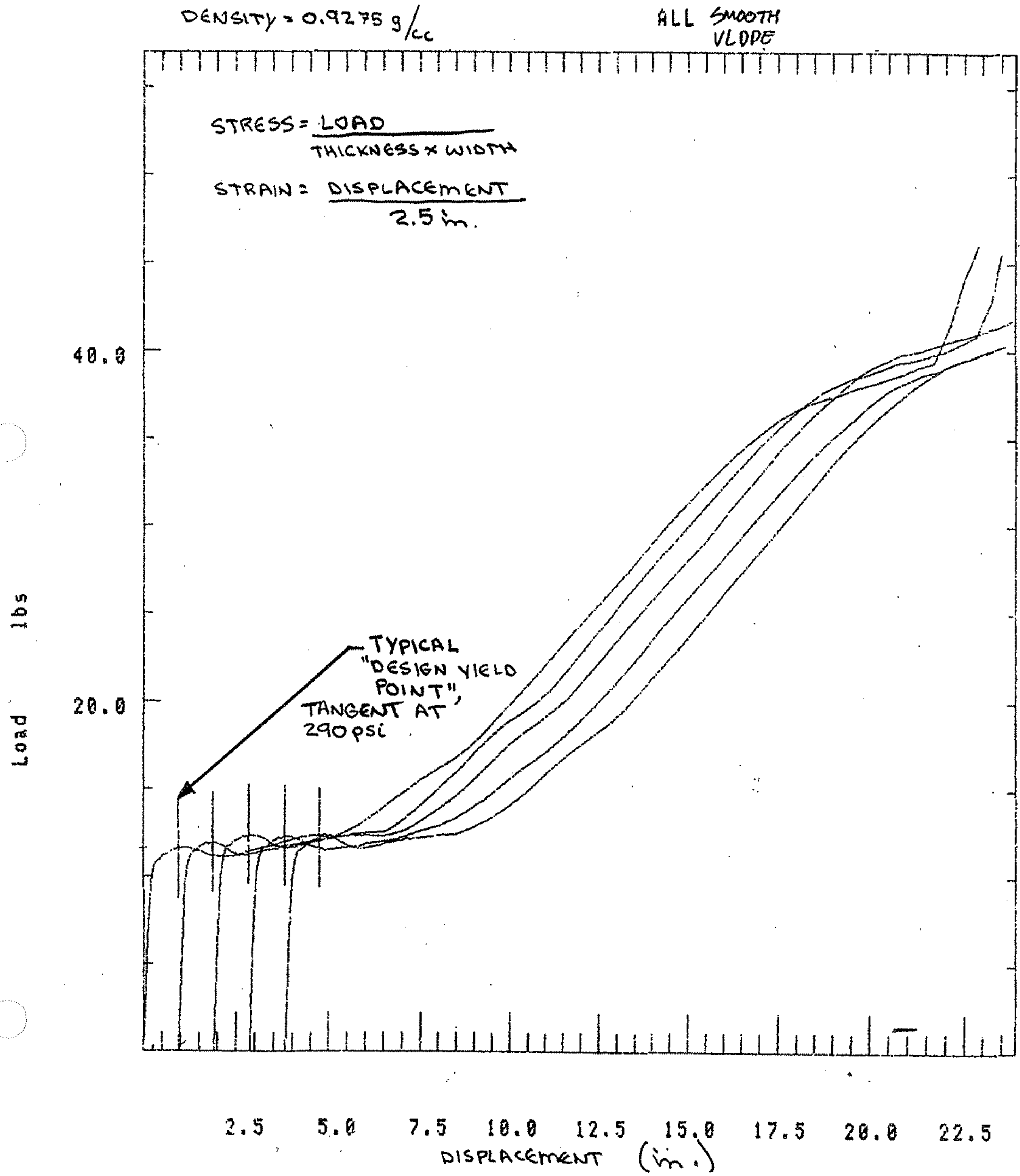
GeoSyntec hopes this memorandum serves as clarification of the methodologies used to identify the DYP of VFPE geomembranes. If you have any questions or comments, please do not hesitate to contact Mike Snow.

* * * * *

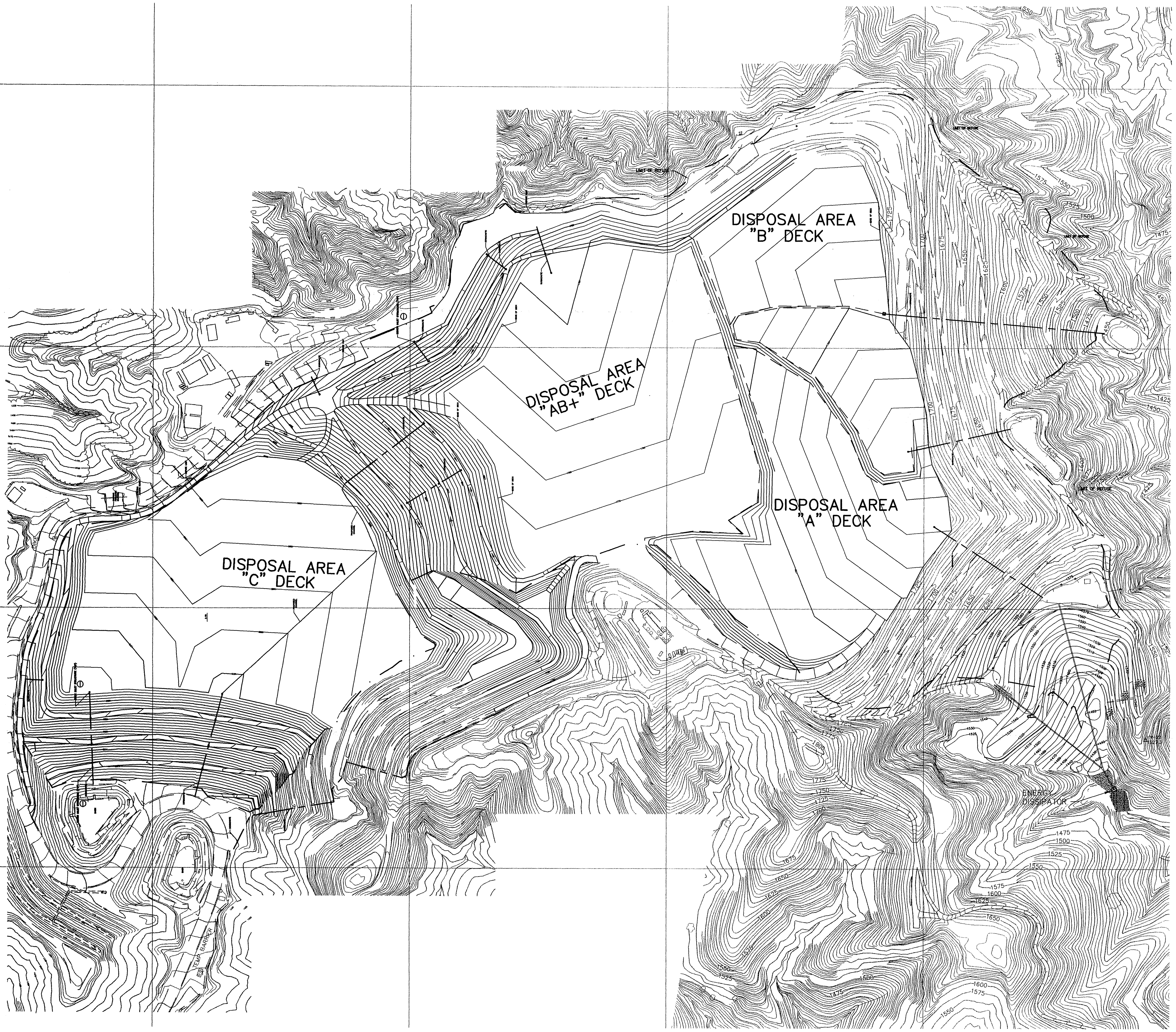
VLDPE LOAD/DISPLACEMENT PLOTS

SAMPLE = 0.25" WIDE

THICKNESS = 41 mil
DENSITY = 0.9275 g/cc



ATTACHMENT H



LEGEND

1725	EXISTING CONTOUR
1725	PROPOSED FINAL GRADE CONTOURS
	DOWNCHUTE
	PROPOSED DIVERSION CHANNEL
	EXISTING PERIMETER CHANNEL
	FLOW LINE
	RIDGE
	EXISTING ACCESS ROAD
	PROPOSED BENCHES
	PROPOSED DECK INLET STRUCTURES
HV-8 1366.2	BENCHMARKS
	REFUSE LIMITS

200 100 0 200 400
SCALE IN FEET

		DATE
DESIGNED	BHC	05-17-96
DRAWN	JT	05-17-96
CHECKED	BHC/MSS	05-17-96
SUPERVISED	EK	05-17-96
PROJECT ENGR.	R.C.E. NO.	
DIV./DIST. ENGR.	R.C.E. NO.	

GEOSYNTEC CONSULTANTS
16541 Gothard Street, Suite 211
Huntington Beach, California 92647
Telephone: (714) 843-6866

LOPEZ CANYON LANDFILL

NO.	REVISION DESCRIPTION	ENGR.	DATE

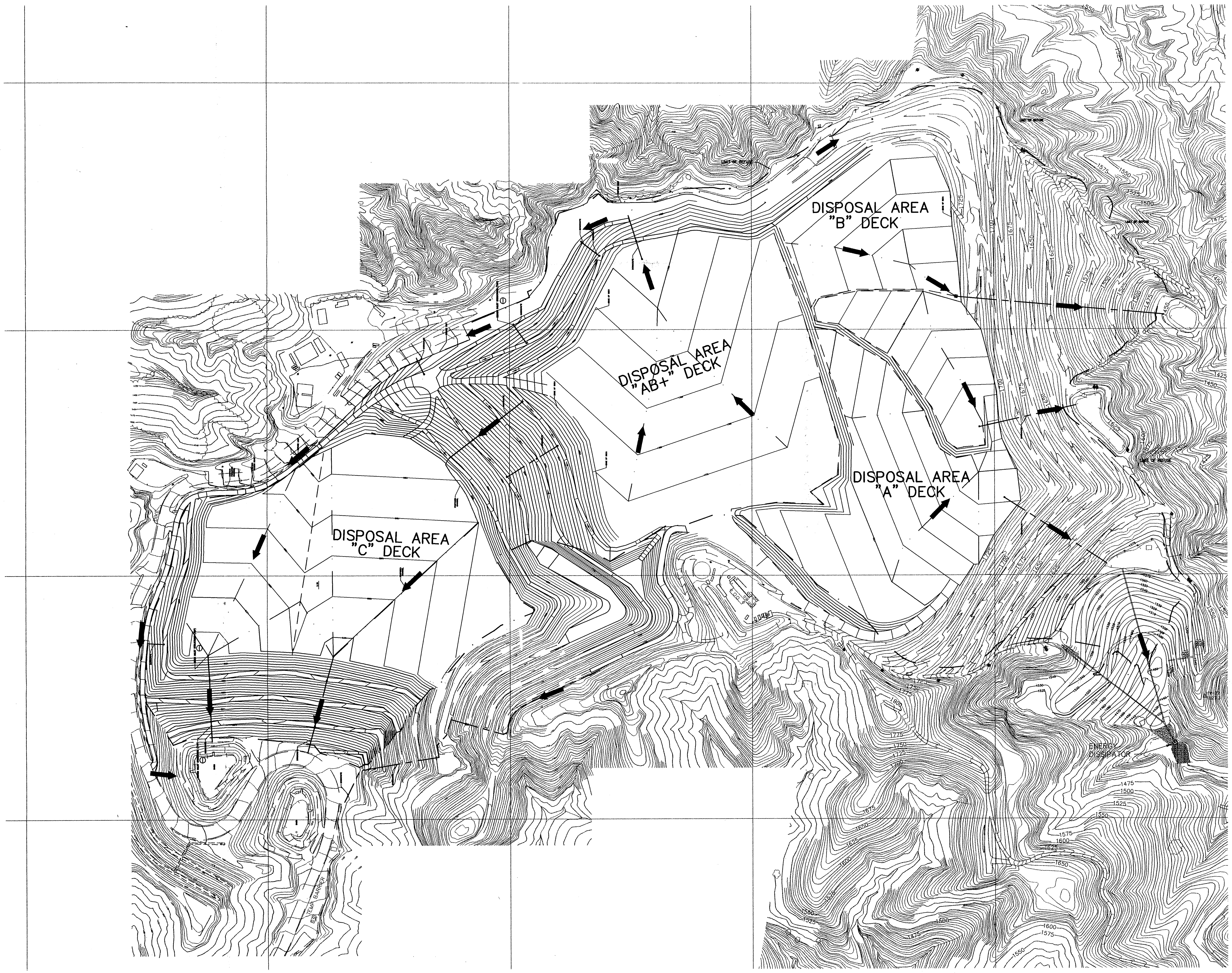
CITY OF LOS ANGELES
BUREAU OF SANITATION DELWIN A. BIAGI, DIRECTOR
DATE ________ 19___
STEPHEN A. FORTUNE R.C.E. NO. 21737
DIV. DIST. ENGR.

SCALE AS SHOWN
SHEET NO.
DWG. NO. 5
JOB NO. CE4100-04

REFUSE BOUNDARY PLAN

DWG: 4100-413 199701161104 AC

ATTACHMENT I



LEGEND

- 1725 EXISTING CONTOUR
- 1725 PROPOSED FINAL GRADE CONTOURS
- DOWNCHUTE
- PROPOSED DIVERSION CHANNEL
- EXISTING PERIMETER CHANNEL
- FLOW LINE
- RIDGE
- EXISTING ACCESS ROAD
- PROPOSED BENCHES
- PROPOSED DECK INLET STRUCTURES
- HV-8 1366.2 BENCHMARKS
- FLOW DIRECTION
- REFUSE LIMITS

200 100 0 200 400
SCALE IN FEET

DESIGNED	BHC	05-17-96
DRAWN	JT	05-17-96
CHECKED	BHC/MSS	05-17-96
SUPERVISED	EK	05-17-96
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LOPEZ CANYON LANDFILL

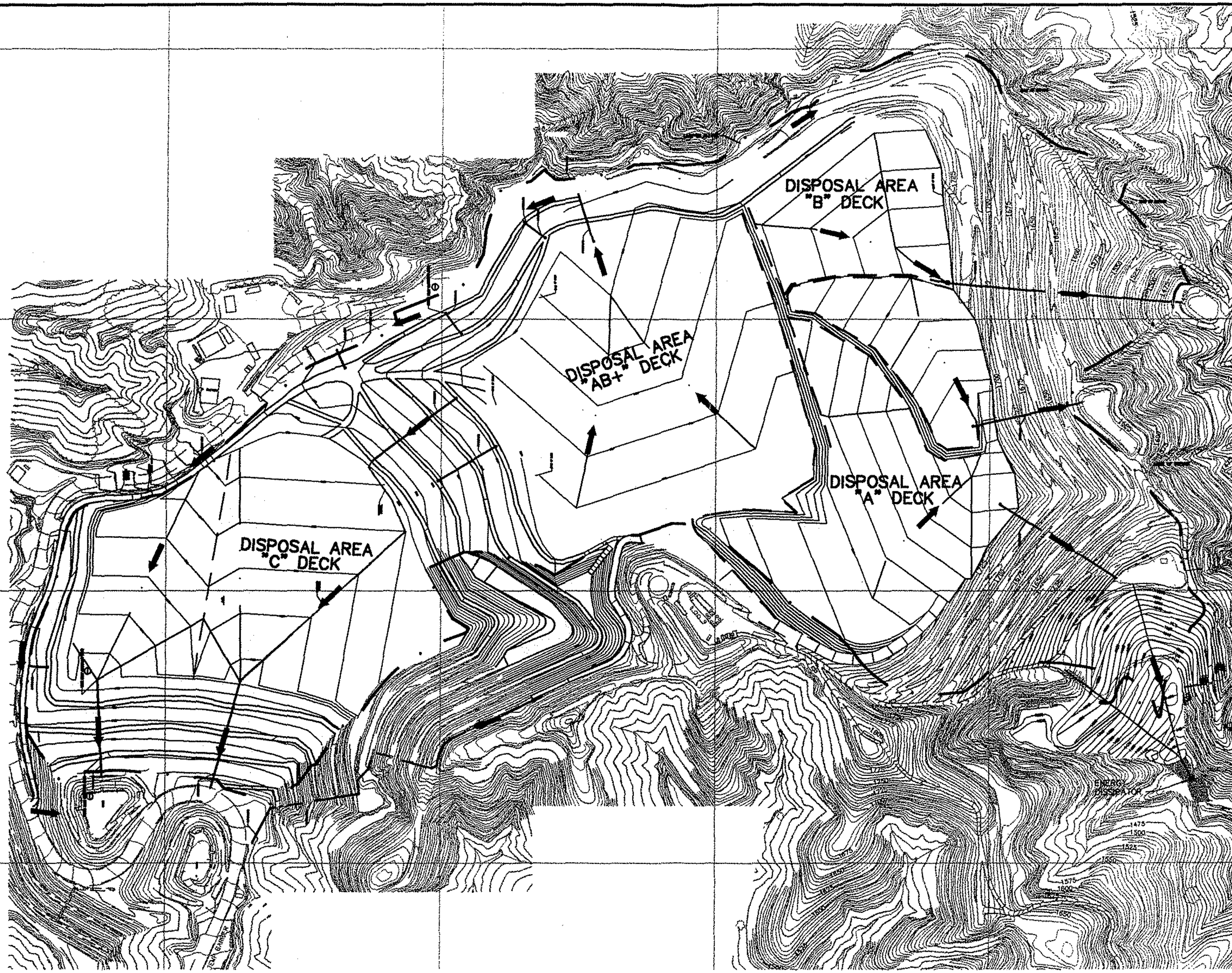
NO.	REVISION DESCRIPTION	ENGR.	DATE

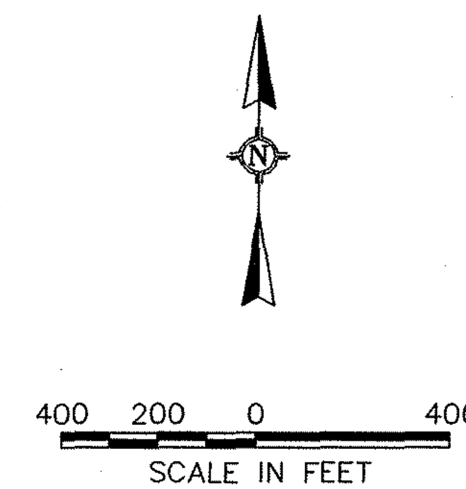
CITY OF LOS ANGELES
BUREAU OF SANITATION DELWIN A. BIAGI, DIRECTOR
DATE ____ 19__
STEPHEN A. FORTUNE R.C.E. NO. 21737
DIV. DIST. ENGR.

SCALE AS SHOWN
SHEET NO.
DWG. NO. 1
JOB NO. CE4100-04

FINAL GRADING AND SURFACE WATER DRAINAGE

DWG: 4100-412 199605151301 JT





LEGEND

- 1725 — EXISTING CONTOUR
- 1725 — PROPOSED FINAL GRADE CONTOURS
- DOWNCHUTE
- PROPOSED DIVERSION CHANNEL
- EXISTING PERIMETER CHANNEL
- FLOW LINE
- RIDGE
- EXISTING ACCESS ROAD
- PROPOSED BENCHES
- PROPOSED DECK INLET STRUCTURES
- HV-8 1366.2 BENCHMARKS
- FLOW DIRECTION
- REFUSE LIMIT

GeoSyntec Consultants

REVISED GRADING AND SURFACE WATER DRAINAGE PLAN DISPOSAL AREA A, B, AB+, AND C
LOPEZ CANYON SANITARY LANDFILL
LOS ANGELES, CALIFORNIA

FIGURE NO.	3-1
PROJECT NO.	CE4100-04
DATE:	DEC-10-96

DWG: 4100F114 199701160841 AC

ATTACHMENT J

7. REVISED LANDFILL GAS CONTROL SYSTEM

7.1 General

The original landfill gas control system was installed at the Lopez Canyon Sanitary Landfill in 1989 and was upgraded in 1992. Initial start up of the system was conducted in December 1989. The landfill gas control system design consists of horizontal and vertical landfill gas wells, lateral collectors, and headers over a large portion of the landfill. The current flare station consists of nine flares. The collected landfill gas is delivered to the flare station where it is disposed of by combustion. Monitoring of the landfill gas control system is performed with perimeter monitoring probes and a landfill gas surface monitoring grid. The landfill gas monitoring system is unchanged from that presented in the FCP.

Revisions to the landfill gas control system presented in the FCP were required as a result of the modifications to the final grading plans in Disposal Area C. Revisions were made only to the layout of the landfill gas control system in this area. The specific components of the system (e.g., headers, wells, etc.) are unchanged from those described in the FCP. The revised layout of the landfill gas control system is presented as Figure 7-1 and Drawing No. 4 of this amendment. Descriptions of the system components are presented below.

7.2 Landfill Gas Control System

7.2.1 General System Layout

The existing landfill gas control system in Disposal Areas A,B, and AB+ was installed prior to the placement of final cover and consists of vertical and horizontal landfill gas wells buried in the intermediate cover which are designed to allow landfill gas condensate to flow to the sumps located at low points around the site. The system modifications described in the following sections will effectively incorporate Disposal Area C into the existing landfill gas control system and will accommodate any increased condensate volumes the system may experience when Disposal Area C has been added. Any additional modifications made to the landfill gas control system during the closure and post-closure maintenance period will be submitted to the LEA and the CIWMB for approval in accordance with §17783.(d) of Title 14.

7.2.2 Disposal Area C

The design of the landfill gas control system for Disposal Area C incorporates a series of horizontal gas wells and collection header lines (see Figure 7-1 and Drawing No. 4 of this amendment). Horizontal wells and collection header lines are installed as the waste is placed.

As Disposal Area C is filled, a system of horizontal landfill gas wells will be installed. A total of five levels of horizontal landfill gas wells will be installed under the Disposal Area C deck. The horizontal spacing between adjacent landfill gas wells lines will be approximately 100 ft (30 m). The vertical distance between each layer of horizontal landfill gas wells will be approximately 40 ft (12 m). The top layer of horizontal landfill gas wells will be approximately 20 ft (6 m) below the final cover.

Each horizontal landfill gas well outlet line will be individually valved and connected to a main landfill gas collection header. The main purpose of the horizontal landfill gas wells is to allow for collection of landfill gas from the center of the landfill. Their chief advantages are lower cost and compatibility with ongoing fill operations.

7.3 Energy Recovery Facility

An energy recovery facility, fueled by landfill gas (LFG) produced by the Lopez Canyon Landfill, is proposed to be constructed and operated at the landfill. The proposed system would be designed, permitted, built, and operated by Minnesota Methane LLC, under a contract with the Lopez Canyon Energy Partners and the City of Los Angeles. The landfill itself will continue to be maintained by the Bureau of Sanitation.

The Proposed Energy Recovery Facility (ERF) would be located on the southwest side of the landfill, adjacent to Disposal Area "C" and existing landfill gas conveyance lines. A site map (Figure 7-2), showing the proposed location of the facility is attached.

The proposed project would have a capacity to use up to approximately 2950 scfm of LFG at 50% methane content. The gas will be used to drive two engines and generate approximately six megawatts (MW) of electrical power. The power will be sold to Southern California Edison Company (SCE). The amount of power generated will be enough to supply approximately 3,000 homes.

7.3.1 Main Features of the Proposed Project

The proposed project will include the following main features:

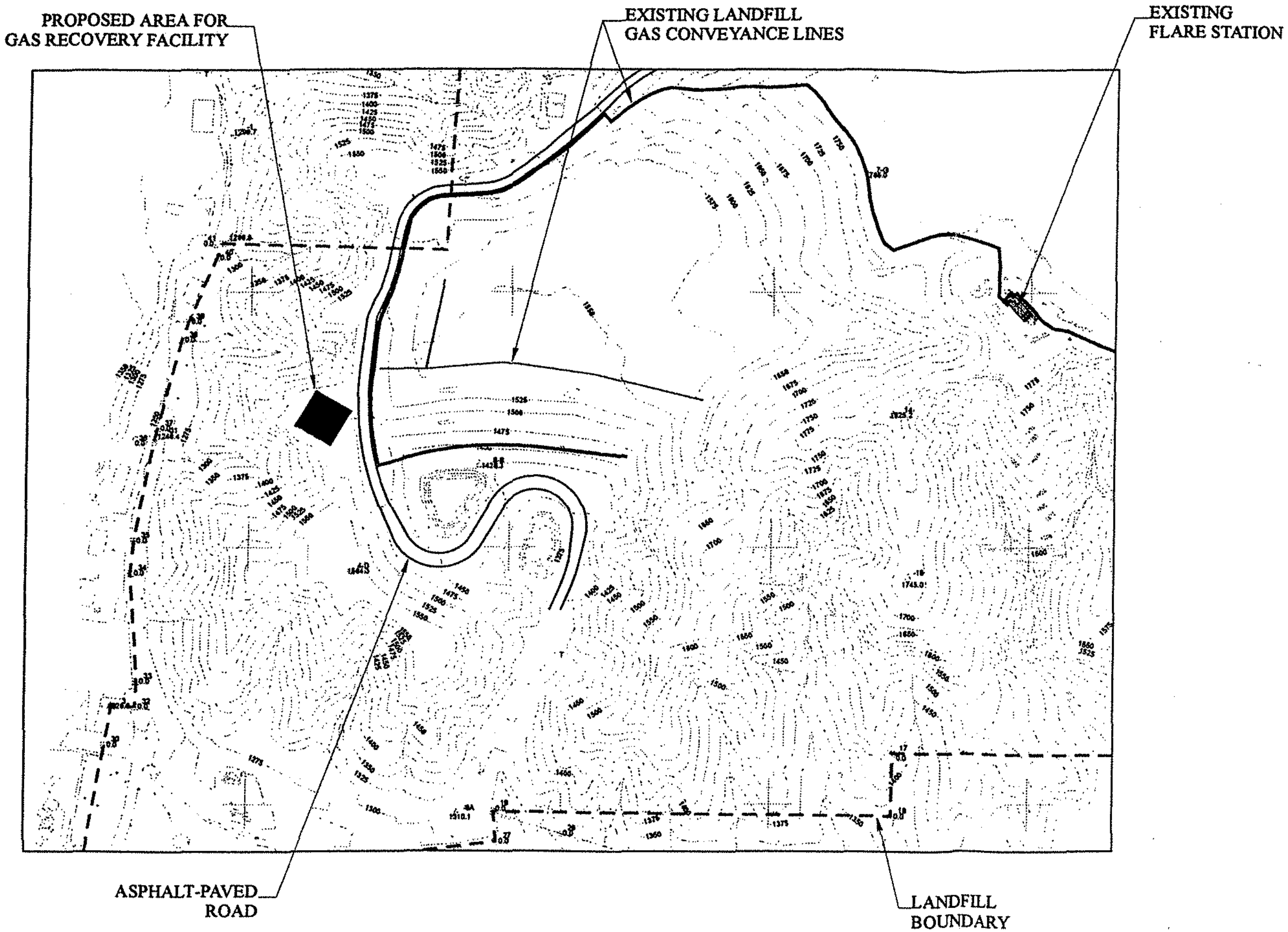
1. Construction of a building and installation of two engines, generators, afterburners, and associated controls and equipment;

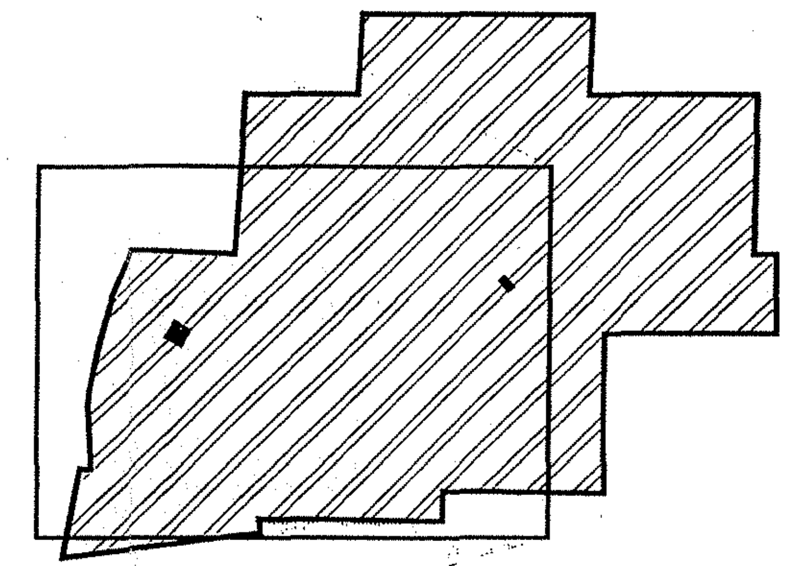
2 Installation of a conveyance line and if needed, blower system to transport the gas from the existing gas conveyance line to the proposed ERF; and

3. Modifications to the existing flares to serve as an emergency backup system.

The proposed floor plan for the ERF, and a Process and Instrumentation Drawing diagram are attached as Figures 7-3 and 7-4.

ATTACHMENT K





LOPEZ CANYON LANDFILL

SOURCE: BASE MAP CAD FILE, CITY OF LOS ANGELES BUREAU OF SANITATION

GATEWAY
SCIENCE & ENGINEERING
CONSULTANTS AND CONTRACTORS
16509 Saticoy Street Van Nuys, CA 91402 1-818-902-0925

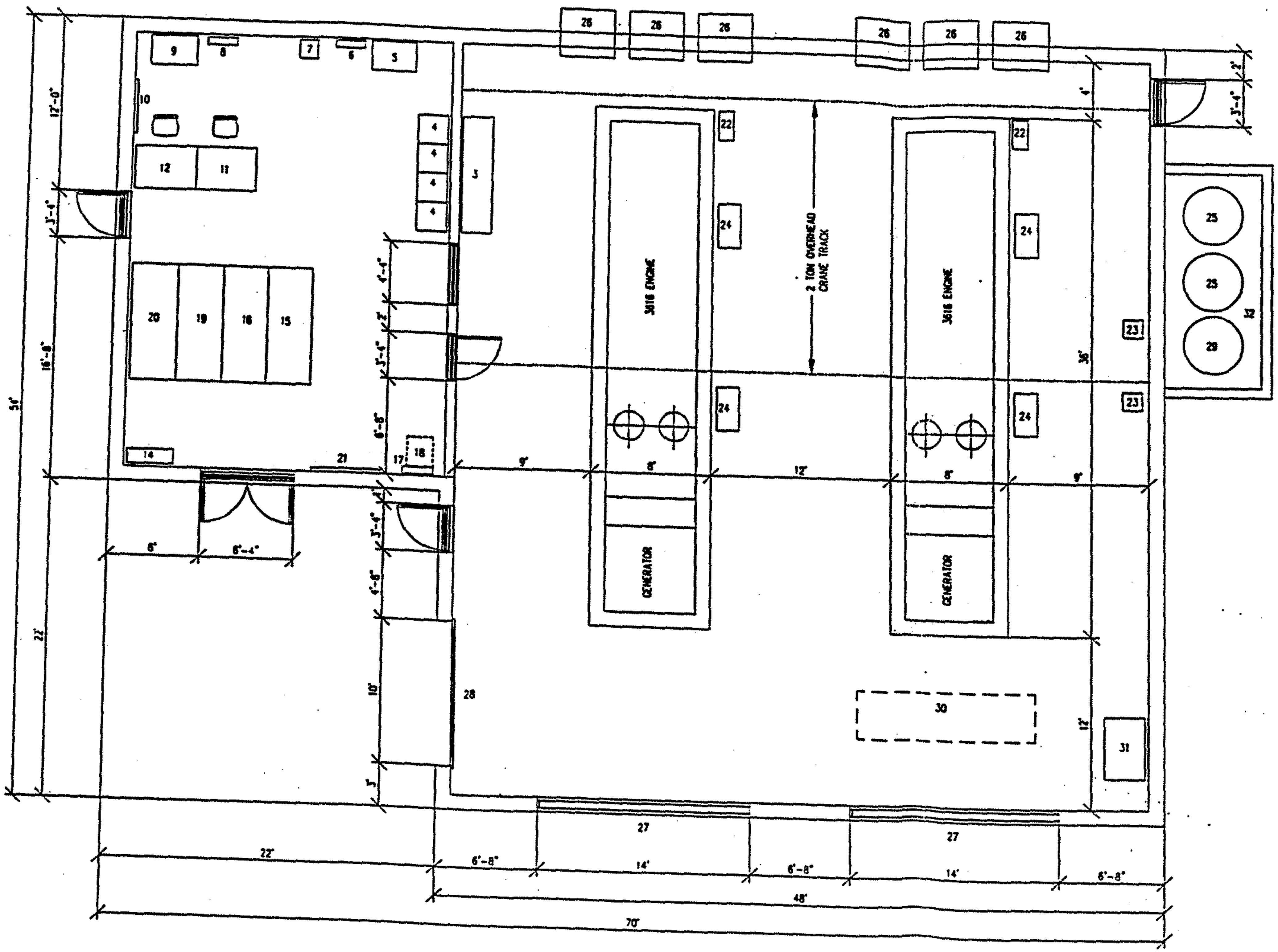
Lopez Canyon Landfill
6 MW Power Plant
Energy Resource Recovery Facility

Drawn By:	B. Price
Approved By:	D. Chakraborty
Date:	10-2-96
Job No:	4013
File Name:	4013-2.cdr
Scale in Feet:	0 500 1000

NORTH

SITE MAP

FIGURE 7-2



1/A2 FLOOR PLAN
SCALE: 1/8" = 1'-0"

N

LEGEND

1. GENERATOR NO. 1
2. GENERATOR NO. 2
3. STATION BATTERIES
4. M C C
5. STATION BATTERY CHARGER
6. D. C. PANEL
7. METHANE MONITOR
8. 120/208 VOLT PANEL
9. 480/120 VOLT TRANSFORMER
10. TELEPHONE PANEL
11. ZTR DESK
12. WORK AREA
13. OIL STORAGE
14. NEUTRAL RELAY CABINET
15. GENERATOR 1 C.B. + CONTROL
16. GENERATOR 2 C.B. + CONTROL
17. UPS PANEL
18. UPS
19. MASTER CONTROL SECTION
20. STATION SERVICE SWITCH
21. METER AREA
22. CAT ESS + ZTR BOX
23. ENGINE BATTERY CHARGER
24. ENGINE BATTERIES
25. LUBE OIL STORAGE
26. EXHAUST FAN
27. REMOVEABLE AIR INTAKE LOUVER
28. 10' X 10' OVERHEAD DOOR
29. GLYCOL TANK
30. OVERHEAD MOUNTED AIR TANK
31. MAINTENANCE AIR COMPRESSOR
32. SPILL CONTAINMENT WALLS

FIGURE 7-3

ZIEGLER INC.
ZIEGLER POWER SYSTEMS

Floor Plan

Lopez Canyon Landfill Project
City of Los Angeles, California

DESIGNED
DRAWN
CHECKED
REVISIONS

DATE 4-3-96
FILE NO. 96021
SHEET NO. A2

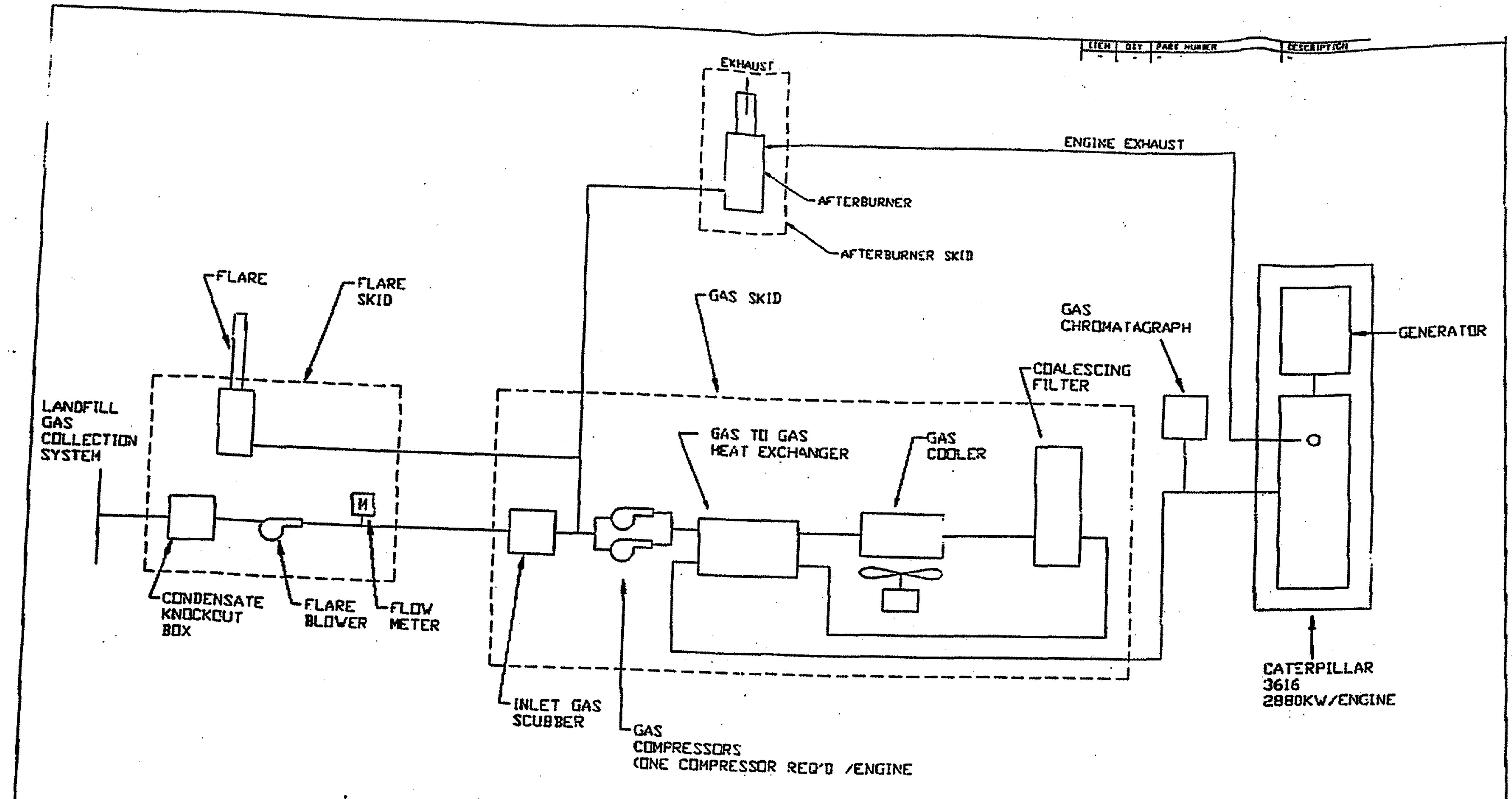
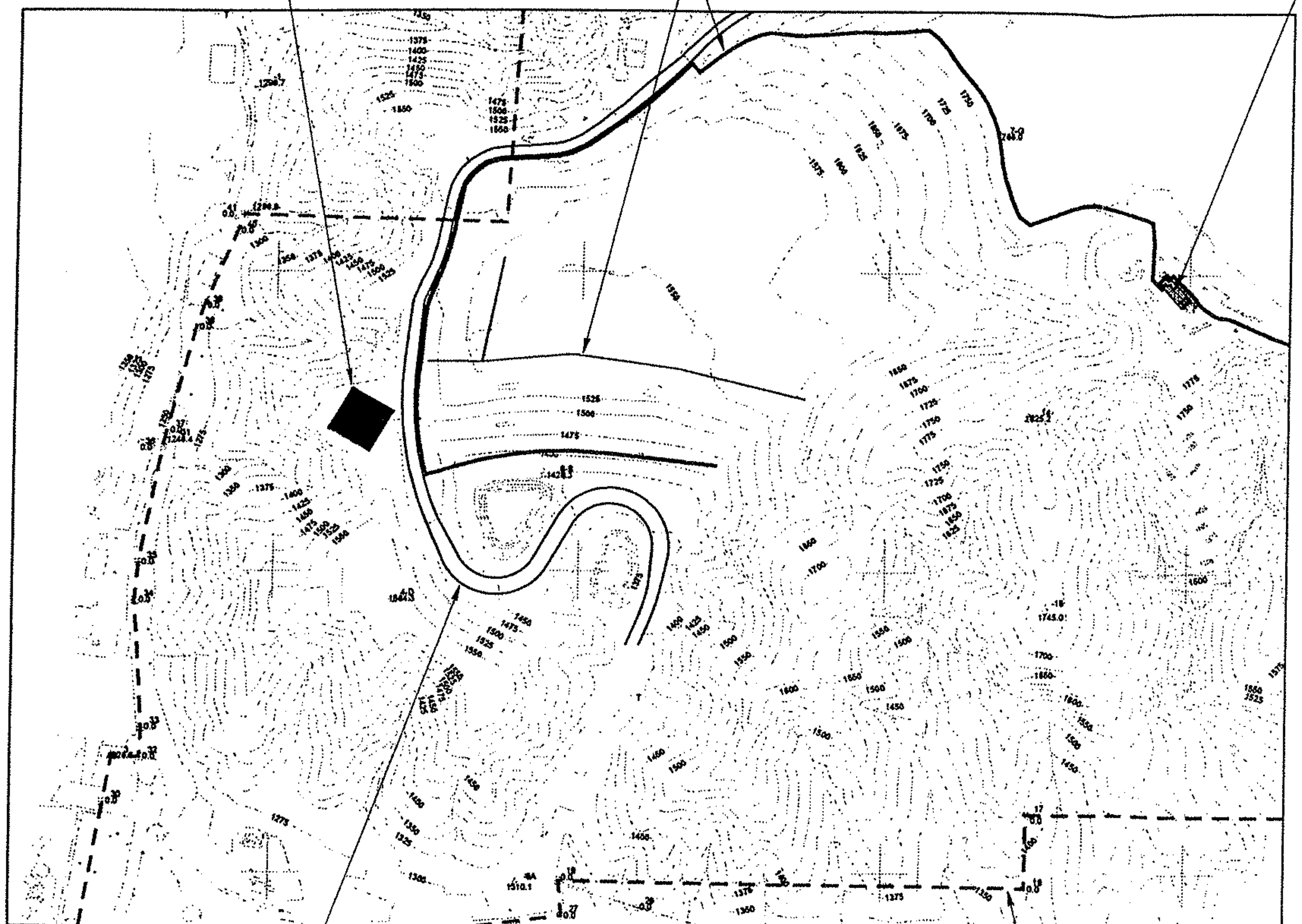


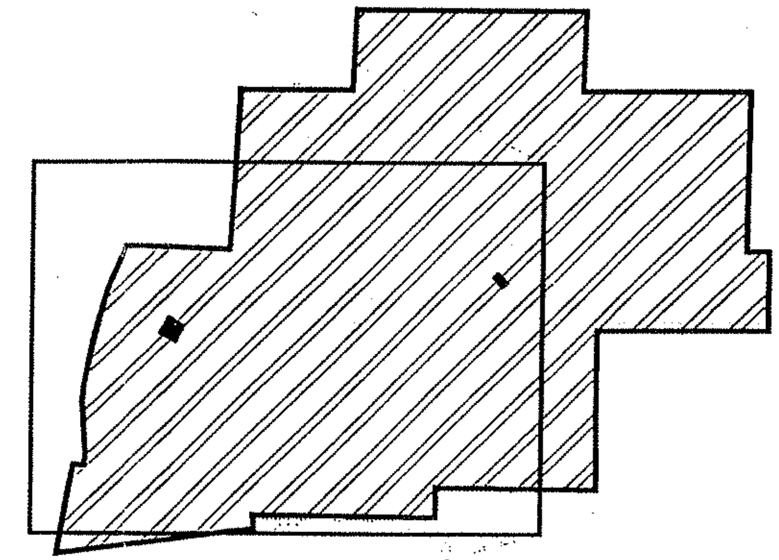
FIGURE 7-4

ZIEGLER CAT
ZIEGLER POWER SYSTEMS

ECS
PM
8/19/96
3616 PROJECTS
PROCESS FLOW DIAGRAM
1 of 1

ATTACHMENT K





SOURCE: BASE MAP CAD FILE, CITY OF LOS ANGELES BUREAU OF SANITATION

GATEWAY
SCIENCE & ENGINEERING
CONSULTANTS AND CONTRACTORS
16509 Saticoy Street Van Nuys, CA 91402 1-818-902-0925

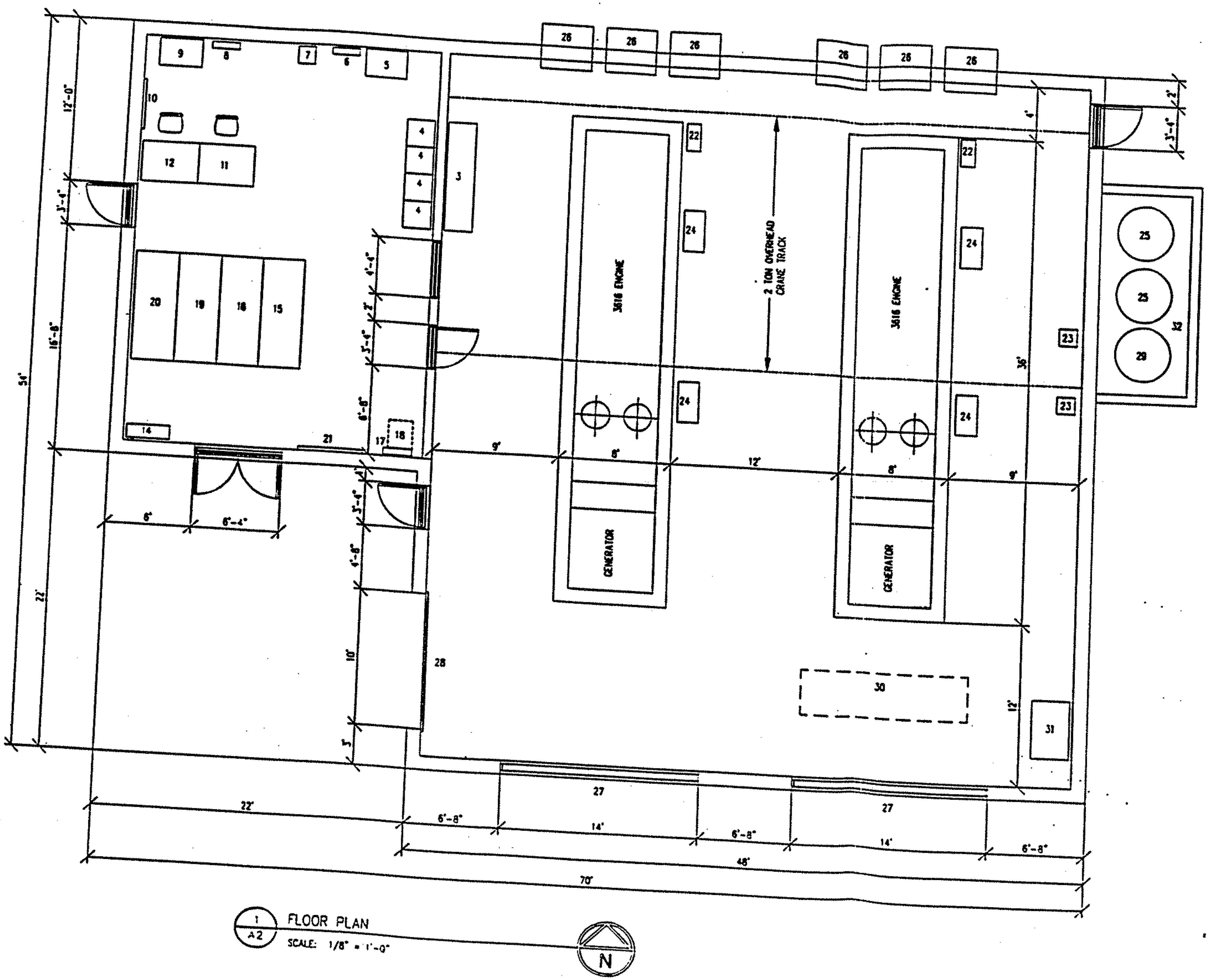
Lopez Canyon Landfill
6 MW Power Plant
Energy Resource Recovery Facility

Drawn By:	B. Price
Approved By:	D. Chakraborty
Date:	10-2-96
Job No:	4013
File Name:	4013-2.cdr
Scale in Feet:	0 500 1000

NORTH

SITE MAP

FIGURE 7-2



LEGEND

1. GENERATOR NO. 1
2. GENERATOR NO. 2
3. STATION BATTERIES
4. M C C
5. STATION BATTERY CHARGER
6. D. C. PANEL
7. METHANE MONITOR
8. 120/208 VOLT PANEL
9. 480/120 VOLT TRANSFORMER
10. TELEPHONE PANEL
11. ZTR DESK
12. WORK AREA
13. OIL STORAGE
14. NEUTRAL RELAY CABINET
15. GENERATOR 1 C.B. + CONTROL
16. GENERATOR 2 C.B. + CONTROL
17. UPS PANEL
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23. ENGINE BATTERY CHARGER
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25. LUBE OIL STORAGE
26. EXHAUST FAN
27. REMOVEABLE AIR INTAKE LOUVER
28. 10' X 10' OVERHEAD DOOR
29. GLYCOL TANK
30. OVERHEAD MOUNTED AIR TANK
31. MAINTENANCE AIR COMPRESSOR
32. SPILL CONTAINMENT WALLS

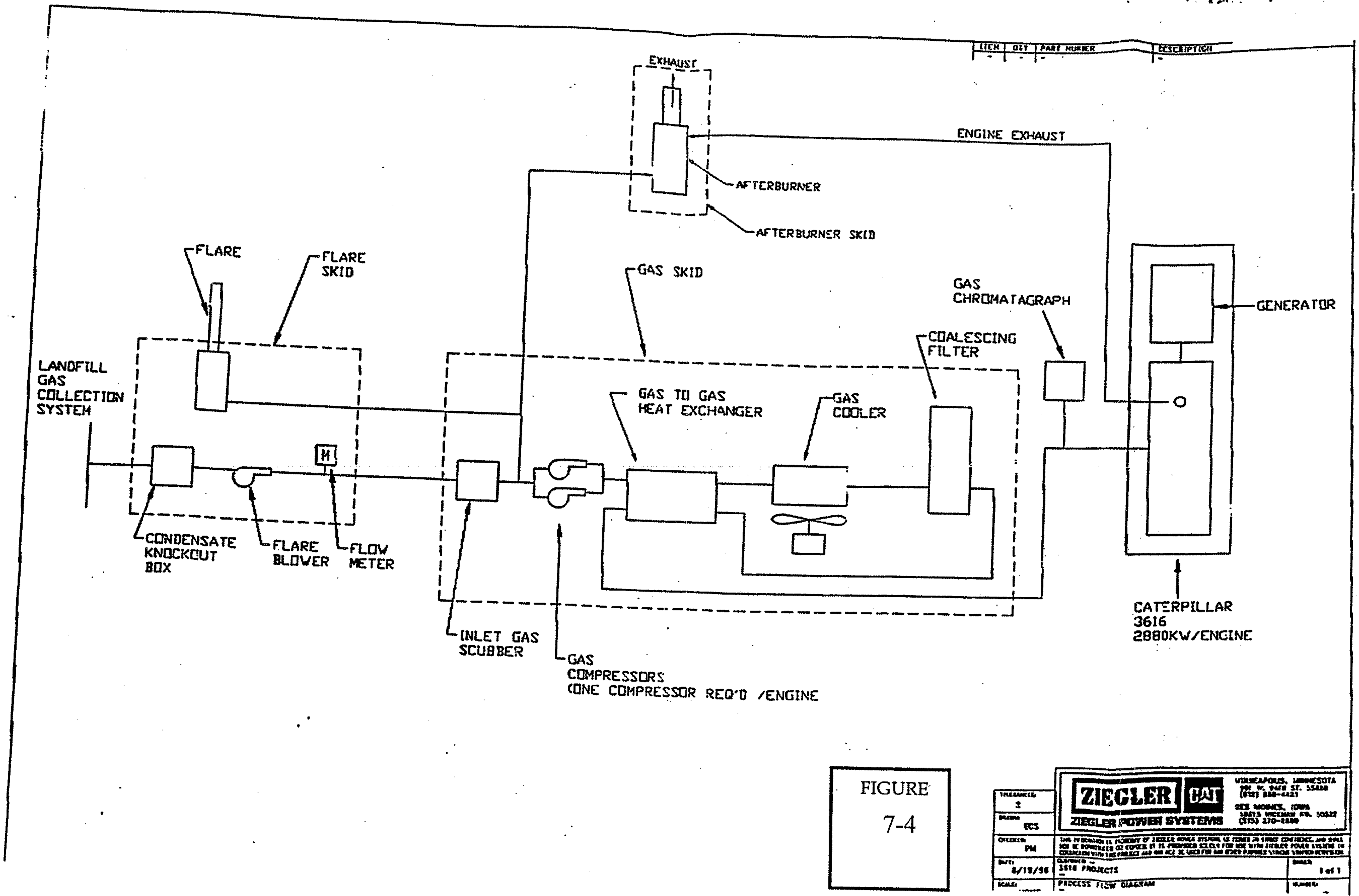
FIGURE 7-3

ZIEGLER INC.
ZIEGLER POWER SYSTEMS

Floor Plan
Lopez Canyon Landfill Project
City of Los Angeles, California

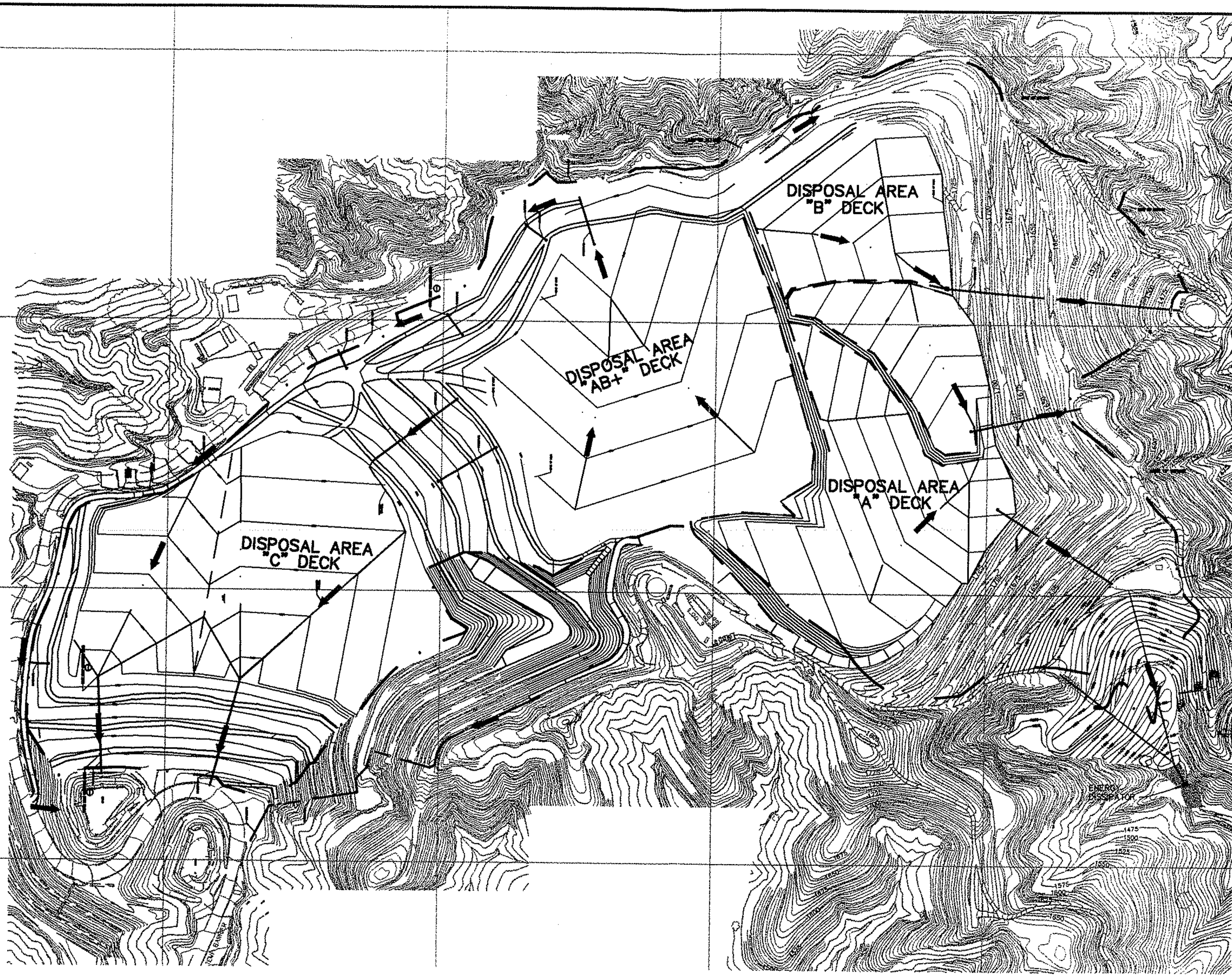
DESIGNED C.B.
DRAWN C.B.
CHECKED C.B.
REVISIONS

DATE 4-3-96
FILE NO. 96021
SHEET NO. A2



ITEM
QTY
PART NUMBER
DESCRIPTION
EXHAUST
ENGINE EXHAUST
AFTERBURNER
AFTERBURNER SKID
FLARE
FLARE SKID
GAS SKID
GAS CHROMATAGRAPH
GENERATOR
COALESCING FILTER
LANDFILL GAS COLLECTION SYSTEM
GAS TO GAS HEAT EXCHANGER
GAS COOLER
M
CONDENSATE KNOCKOUT BOX
FLARE BLOWER
FLOW METER
CATERPILLAR 3616 2880KW/ENGINE
INLET GAS SCUBBER
GAS COMPRESSORS (ONE COMPRESSOR REQ'D /ENGINE
ZIEGLER
CAT
ZIEGLER POWER SYSTEMS
MINNEAPOLIS, MINNESOTA
DES MOINES, IOWA
ECS
PM
PROCESS FLOW DIAGRAM
1 of 1

FIGURE 7-4



400 200 0 400
SCALE IN FEET

LEGEND

- 1725 EXISTING CONTOUR
- 1725 PROPOSED FINAL GRADE CONTOURS
- DOWNCHUTE
- PROPOSED DIVERSION CHANNEL
- EXISTING PERIMETER CHANNEL
- FLOW LINE
- RIDGE
- EXISTING ACCESS ROAD
- PROPOSED BENCHES
- PROPOSED DECK INLET STRUCTURES
- HV-8 1386.2 BENCHMARKS
- FLOW DIRECTION
- REFUSE LIMIT

GeoSyntec Consultants

REVISED GRADING AND SURFACE WATER DRAINAGE PLAN DISPOSAL AREA A, B, AB+, AND C
LOPEZ CANYON SANITARY LANDFILL
LOS ANGELES, CALIFORNIA

FIGURE NO. 3–1
PROJECT NO. CE4100-04
DATE: DEC-10-96

DWG: 4100F114 199701160841 AC

FINAL CLOSURE PLAN
LOPEZ CANYON SANITARY LANDFILL
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FINAL CLOSURE PLAN
LOPEZ CANYON SANITARY LANDFILL
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LOPEZ CANYON SANITARY LANDFILL
TABLE OF CONTENTS (continued)
VOLUME IV OF IV REPLACEMENT - AMENDMENT TO FINAL CLOSURE PLAN

FINAL CLOSURE PLAN
LOPEZ CANYON SANITARY LANDFILL
TABLE OF CONTENTS (continued)
VOLUME IV OF IV REPLACEMENT - AMENDMENT TO FINAL CLOSURE PLAN

FINAL CLOSURE PLAN
LOPEZ CANYON SANITARY LANDFILL
TABLE OF CONTENTS (continued)
VOLUME IV OF IV REPLACEMENT - AMENDMENT TO FINAL CLOSURE PLAN

FIGURES (continued)

FINAL CLOSURE PLAN
LOPEZ CANYON SANITARY LANDFILL
TABLE OF CONTENTS (continued)
VOLUME IV OF IV REPLACEMENT - AMENDMENT TO FINAL CLOSURE PLAN

APPENDICES

FINAL CLOSURE PLAN
LOPEZ CANYON SANITARY LANDFILL
TABLE OF CONTENTS (continued)
VOLUME IV OF IV REPLACEMENT - AMENDMENT TO FINAL CLOSURE PLAN

DRAWINGS