January 27, 1988

REVISIONS TO PARAGRAPH 5(G), EXHIBIT M444
JOINT WITNESS STATEMENT
OF DR. DAVID CHARLESWORTH AND DR. R. KERRY ROWE

(g) The proposed landfill will have minimal impact on deep groundwater quality, i.e. groundwater in the shale; thus, in the event that there was a discharge of this groundwater into the shallow groundwater system, i.e. the freshwater lens southeast of Highway 403, the impact of the landfill on the shallow groundwater system, i.e. freshwater lens , would also be minimal. Furthermore, upward movement of deep groundwater, i.e. groundwater in the shale, through the till would result in additional attenuation. Indeed, in the event that the deep groundwater did discharge in this manner, it is expected that the impact of the naturally poor quality of this groundwater would be far more significant than any impact due to contaminants in the groundwater which originated from the proposed Halton landfill.

(The term "deep groundwater" as used in this paragraph and also in Dr. Rowe's report on Site F, Exhibit M440, refers to the shale groundwater in the upper zone of the shale. The thickness of this upper zone is denoted by the symbol T in his Site F report. The "deep groundwater" referred to by Dr. Rowe, is also schematically indicated on Exhibit M445R as the Upper Zone in Shale.)

WITNESS STATEMENT

INTRODUCTION

1. Dr. Rowe's two reports dated October 1987, "An Examination of the Potential Impact on Groundwater Quality ..." at the two proposed sites and his curriculum vitae were previously distributed. A separately bound booklet referencing his publications, awards and research grants related to contaminant transport is being delivered concurrently with this Witness Statement. This booklet also contains a copy of some of his publications relevant to the methodology used in the October 1987 reports.

2. The two reports by Geocon, October 1987, concerning the proposed sites were previously distributed.

3. Appended to this Witness Statement are copies of a series of overhead projections to be used during evidence-in-chief. Some of these overheads are figures already found in Dr. Rowe's October 1987 Reports. Others illustrate concepts and data either presented in the reports or will be used to discuss evidence and exhibits introduced to date.

SITE "F"

4. Dr. Charlesworth and Dr. Rowe will review their

respective October 1987 Reports regarding Site F.

5. Dr. Rowe's general conclusions arising from his October 1987 Report concerning Site F are the following:

(a) Assuming there was no matrix diffusion or dispersion, then contaminants moving along the fractures might be expected to move at the same velocity as the groundwater; however, this is not a realistic assumption for the shale at Site F.

(b) If matrix diffusion is considered at Site F, the speed of contaminant movement is much slower than the average linearized ground water velocity (sometimes also called seepage velocity).

(c) All other things being equal, the fracture porosity and average linearized groundwater velocity have very little effect on the rate of contaminant migration. For a given flow per unit area through the rock (expressed as the "Darcy velocity"), two orders of magnitude variation in fracture porosity and average linearized groundwater velocity had no significant effect on the rate of contaminant movement.

(d) The Darcy velocity (or flow per unit area) is one of the key variables controlling the concentration of contaminant at a given point downgradient of the

landfill at a given time.

(e) Fracture spacing and the diffusion co-efficient are of importance at small times and at locations close to the landfill, however, at larger times and at distance from the landfill, reasonable uncertainty regarding the magnitude of either of these quantities had very little effect on the calculated impact.

(f) Considering the effect of matrix diffusion, there is substantial attenuation of contaminant as the flow occurs through fractures in the shale; the concentration of contaminant 300 or more metres from the landfill is very small.

(g) The proposed landfill will have minimal impact on deep groundwater quality; thus, in the event that there was a discharge of this groundwater into the shallow groundwater system southeast of Highway 403, the impact of the landfill on the shallow groundwater system would also be minimal. Furthermore, upward movement of deep groundwater through the till would result in additional attenuation of and improvement to the deep groundwater quality. Indeed, in the event that the deep groundwater did discharge in this manner, it is expected that the impact of the naturally poor quality of this groundwater would be far more

significant than any impact due to contaminants in the groundwater which originated from the proposed Halton landfill.

6. Dr. Rowe will indicate he has reviewed three possible scenarios applicable to Site F:

Scenario "A"

Assumes there is no "weakly fractured massive shale unit" and, therefore, that contaminant can readily move down through the fractured rock (this is the worst case and is the case considered in his October 1987 Report);

Scenario "B"

Assumes a fractured zone 25 m wide at the south end of both the Burlington and proposed Site F landfills through which contaminant passes down vertically 15 m and then travels laterally through a 25 metre wide zone;

Scenario "C"

Assumes a 15 metre thick "weakly fractured massive shale unit" beneath both the Burlington and proposed Site F landfills, which is underlain by a 5 metre thick fractured zone where lateral migration occurs (this roughly corresponds to THCL's conceptual model).

Scenario "A"

7. Scenario "A" represents the worst case. This assumes no "weakly fractured massive shale unit" and, therefore, contaminant can readily move down through the fractured rock. This is the case considered in his October 1987 Report on Site F. He will review his Site F report and in summary his basic conclusions are the following:

(a) For a Darcy velocity of 0.125 m/a, which could correspond to:

- mounding to the top of the landfill (increases the gradient between landfill and 300 m south to less than 0.0875) and a hydraulic conductivity of 4.5×10^{-6} cm/s; or
- a gradient of 0.035 and a hydraulic conductivity of 1.1×10^{-5} cm/s

after 10-15 years one would expect to find conservative contaminants ($c < 45\% c_o$) 25 m south of the Burlington landfill. If the fracture spacing was about 5 cm, then one would expect to find organics such as benzene 25 m south of the landfill. If the fracture spacing was of the order of 30 cm then one might find benzene at concentrations less than 17 ug/L (i.e. $c < 11\% c_o$) at 25 m south and conservative species at concentrations (5% c_o) up to 65 m south.

(b) For a Darcy velocity of 0.125 m/a the impact of Site F 300 m downgradient would be less than a 15% increase in concentration for conservative species and about 3% increase in organics such as benzene (i.e. 4.5 ug/L for benzene). Peak impact would take 200-900 years at 300 m. The impact 800 m downgradient would be less than a 10% increase in concentration of conservative species and 2% for organics such as benzene (i.e. 3 ug/L) and it would take 525-2700 years for peak impact to occur.

(c) For a Darcy velocity of 0.125 m/a, the proportion of leachate not collected and, therefore, passing into the deep groundwater system and moving laterally off-site is 0.8%. .8% leachate.

(d) For a Darcy velocity of 0.05 m/a, which could correspond to a lateral gradient of 0.035 and a hydraulic conductivity of the rock of 4.5×10^{-6} cm/s, after 10-15 years one would expect to find conservative contaminants at low concentrations ($c < 10\% c_o$) 25 m south of the landfill if the fracture spacing was about 30 cm. For a fracture spacing of about 5 cm one would not expect to be able to detect conservative species. Organics such as benzene would not be detected (at 1 ug/L) 25 m south of the landfill.

(e) For a Darcy velocity of 0.05 m/a, the impact of Site F 300 m downgradient would involve increases in conservative species and organics (such as benzene) concentrations of less than 7% and 1.5% (e.g. 2 ug/l for benzene) respectively and peak impact would take 450-2200 years. At 800 m downgradient peak impact of conservative species and organics (benzene) is less than 4% c_o and 1.5% c_o respectively and peak impact would take 1100-6500 years.

(f) For a Darcy velocity of 0.05 m/a, the proportion of leachate not collected and, therefore, passing into the deep groundwater system and moving laterally off-site is approximately 0.3%.

Scenario "B"

8. Scenario "B" assumes that there is fractured zone 25 m wide at the south end of both the Burlington and proposed Site F landfills through which contaminants pass down vertically for a distance of 15 m and then laterally through a 25 m wide zone. This case considers time of migration down to the fracture zone of similar dimensions to that postulated by THCL. Dr. Rowe's conclusions under this scenario are:

(a) that for a Darcy velocity of 0.125 m/a, after 10-15 years migration one would expect to find conservative

contaminant at concentrations less than 20% c_o, 25 m south of the landfill but probably not at points 65 m or more from the landfill. One might just find benzene 25 m south at concentrations less than 2.5% c_o, (i.e. 4 g/L) if fracture spacing is of the order of 30 cm.

(b) For a Darcy velocity of 0.125 m/a or 0.05 m/a, the impact of Site F would be slightly less than indicated in Scenario "A" for these two Darcy velocities.

Scenario "C"

9. Scenario "C" assumes a 15 m thick "weakly fractured massive shale unit" beneath both the Burlington and proposed Site F landfills, and assumes these are underlain by a 5 m thick fractured zone where lateral migration occurs. This scenario approximates THCL's conceptual model.

In discussing this scenario, Dr. Rowe will comment on the THCL "Better Fit Model" (Exhibit H221) and compare it to THCL's original modelling. He will indicate that the "Better Fit Model" is indeed a better fit in respect of Site F, although it is not necessarily the best fit.

He will indicate that based on flows given by THCL's "Better Fit Model", he has performed an analysis using a two-dimensional finite layer model (i.e. the same computer program used in his analysis of Site D). The results of this modelling indicate:

(a) The peak concentration at the downgradient edge of the proposed Site F landfill is less than 85 mg/L (approximately 5% of initial concentration c_o) after about 550 years. This concentration would decrease due to matrix diffusion as contaminants migrated downgradient. This, as with all the calculations in the Site F report, assumes a mass of chloride of 7800 tonnes. This is substantially higher than the mass of 4400 tonnes assumed in the analysis of Site D. If this calculation was repeated for a mass of 4400 tonnes, the peak concentration at the downgradient edge of the landfill is reduced from 85 mg/L to 50 mg/L (i.e. 3% of c_o).

(b) The amount of leachate moving off-site under the assumptions of the Better Fit Model would be less than .3% (i.e. 99.7% of leachate will be collected).

(c) One would not expect either conservative species or organics to be detected in the highly fractured shale 25 m south of the Burlington landfill after only 15 years migration.

Dr. Rowe will conclude with respect to this scenario that the impact would be less than indicated for the worst case, Scenario "A" and also less than in Scenario "B".

10. In response to cross-examination, Mr. Feenstra indicated that THCL considered a "conservative failure scenario" in their impact calculations of Site D (Vol. 43, p. 9713). Similarly, he indicated that they considered a "large scale failure" (Vol. 46, p. 10336).

In fact, Dr. Rowe will indicate that although THCL's modelling involved a mound of approximately 6 m above the base of the landfill as indicated by Mr. Feenstra at page 9712, they did not consider the implications of uncertainty regarding the hydraulic conductivity of the upper till or the effect of the leachate mound on the head in the Upper Granular Unit.

In summary on this point, Dr. Rowe will indicate that nowhere in THCL's impact calculation is a mound of more than approximately 6 m or a hydraulic conductivity of the upper till greater than 8×10^{-8} cm/s considered. Dr. Charlesworth as well as Dr. Rowe will indicate that any prudent impact assessment would consider reasonable failure scenarios, including reasonable worst case scenarios. Dr. Rowe has done this in his October 1987 report on Site D. The range of mounding considered is from 3.7 m to 15 m and hydraulic conductivities of the upper till from 8×10^{-7} cm/s to 8×10^{-8} cm/s. For example, Dr. Rowe's report indicates that even with failure in 3 collection tiles in one cell, a mound of 4.6 m would be produced. This is less than the

mound considered by THCL, however, only half an order of magnitude error in the hydraulic conductivity of the upper till creates an impact greater than THCL's worst case scenario.

11. Dr. Rowe will indicate that in considering the phenomenon of matrix diffusion and in modelling its effects at Site F, he is using the same concept Mr. Feenstra utilized when Mr. Feenstra calculated or modelled the effect of diffusion through the matrix of the weakly fractured massive unit at Site F. In fact exactly the same governing equation and parameters control the migration from fractures into the matrix of the rock as was used by Mr. Feenstra in modelling the migration from the landfill through the massive shale matrix.

12. Dr. Rowe will comment on Exhibit N-253 as to whether the porosities reported in this analysis are consistent with his test data and assumptions.

13. Dr. Rowe will discuss the evidence by Mr. Feenstra in Vol. 42, p. 9451 where he states that in general the degree of attenuation depends very, very significantly on the actual rate of groundwater flow through the cracks, the aperture or opening size of the cracks and the effect of the diffusion coefficient. Dr. Rowe will discuss each of these

aspects and will demonstrate that reasonable uncertainty with respect to either the groundwater velocity or the fracture opening size has negligible effect upon the level of attenuation of contaminant that is achieved downgradient of the landfill.

14. With respect to Mr. Feenstra's response to Mr. Tidball's question found at Vol. 46, p.10402 that we don't know precisely which fractures carry the bulk of the groundwater, Dr. Rowe would agree but would point out that if it's only a few fractures carrying the contaminant, then there is unlikely to be any significant impact. The only way there can be a substantial impact is if there is a large volume of contaminant moving through the shale over a large area. Given the hydraulic characteristics of the site, as commented upon by Dr. Charlesworth, Dr. Rowe is of the opinion that contaminant would have to move over a wide front of the order of 100 or more metres and through a thickness of the order of 10's of metres to have a significant impact downgradient of the landfill. The monitoring data which is available would suggest that this is not the case.

15. With respect to Mr. Feenstra's response to a question from Mr. Tidball at Vol. 46, p. 10415, as to whether the presence of very high concentrations of methyl ethyl ketones and tetrahydrofurans would be conclusive proof that

the organic contamination in the wells was due to glues, Dr. Rowe would comment that (a) dilution of these species along with the other organic species may have occurred due to past sampling at these wells; and (b) these contaminants are likely to diffuse into the matrix of the rock. He will caution that it should not necessarily be anticipated that they would still be at very high concentration levels and that these two processes are likely to complicate interpretation of any test data.

16. Dr. Charlesworth will comment with respect to Dr. Kingham's questions to Mr. Feenstra (Vol. 46, pp. 10497-98) with respect to the potential for migration of leachate at Site F in the weathered shale. He will indicate that to the extent that any leachate at that site may migrate through the weathered shale, it can be caught by relatively straightforward engineering devices such as tow drains that have been installed at both the Bayview and Burlington Park landfill sites.

17. Dr. Rowe will comment on other portions of the testimony of Mr. Sobanski and Mr. Feenstra, including:

(a) Dr. Rowe agrees with Mr. Sobanski (Vol. 36, p. 7943) that the effect of changing the hydraulic conductivity of the highly fractured unit will be very dependent on how

much water can be forced through the upper or moderately fractured massive shale units;

(b) With respect to Mr. McQuaid's question on page 7947 as to what the effect of a two order of magnitude difference in the hydraulic conductivity of the highly fractured shale unit, Dr. Rowe will indicate this would have relatively little effect on THCL's calculation of the flow from the landfill since the flows off-site are primarily being controlled by the weakly fractured massive shale unit. He will indicate that a simple calculation would show that a two order of magnitude variation in the hydraulic conductivity of the highly fractured unit would only increase the flow by something of the order of 15%.

(c) Dr. Rowe will comment on the question by Ms. Vigod and Mr. Feenstra's response found in Vol. 46, p. 10323, that appears to imply that sand could absorb leachate whereas rock cannot. Dr. Rowe will indicate that the example quoted by Ms. Vigod is not relevant when one is dealing with a rock such as the shale at Site F.

18. Dr. Rowe will indicate that matrix diffusion is not the only possible explanation for the fact that no significant quantities of contaminant have been detected downgradient of the Burlington landfill. Matrix diffusion

may only be one of several factors which are resulting in significant attenuation. For example, if a weakly fractured massive shale unit, postulated by THCL does exist, then one would expect that this would further retard the migration of contaminant from the landfill and also provide an additional opportunity for dilution of contaminant as it migrates into the more fractured zone. If this zone is present, then it will serve a similar function for waste migrating from the proposed landfill as it does for waste migrating from the existing landfills.

19. Dr. Rowe agrees with Mr. Feenstra's response to Ms. Vigod at Vol. 46, p. 10316:

> "... the simulations performed at Site F suggest that given the hydrostratographic conceptualization of the flow system, the potential for absorption of these volatile organic chemicals and the relatively long flow path length to the discharge zone, there will be a fairly significant degree of attenuation irrespective of matrix diffusion. Now, what the observations at the Bayview and Burlington sites suggest to us is that due to the effect of matrix diffusion, our simulations as we performed at Site F in 1986 are probably excessively conservative..."

20. Dr. Charlesworth will, in reviewing Ms. Hewetson's thesis, Exhibit E214, indicate that for many of the same reasons given by Mr. Feenstra in evidence, for example at Vol. 44, pp. 9876-9989, there is no basis to conclude the

contaminants reported by her south of Bayview Park are due to landfilling. Dr. Charlesworth will indicate that in addition to his opinion expressed at page 5-3 of his October 1987 report on Site F, it is likely that the volatile organic chemical concentrations detected by Hewetson are the result of other sources of contamination, such as solvents used in the construction of a peizometer or inadequately cleaned pipes used in the borehole installation. Dr. Charlesworth will indicate that he agrees with Mr. Feenstra that, for the other reasons set out in Mr. Feenstra's witness statement at pages 15-16, there is no basis to conclude that contaminants reported by her were due to landfilling.

21. With respect to Dr. Charlesworth comments in his October 1987 Site F report, pp. 4-7 to 4-8 regarding the hydraulic conductivity of the "highly fractured unit", Dr. Charlesworth will refer to Mr. Feenstra's opinion at Vol. 46, p. 10438, to the effect that THCL's method of calculating hydraulic conductivity is incorrect and that the value derived from the pumping test should be at least one order of magnitude lower than that given by THCL.

22. Dr. Charlesworth will comment on Mr. Feenstra's testimony with respect to a monitoring program for Site F (Vol. 46, pp. 10453-54) and agree that he is confident, as Mr. Feenstra is at page 10460, that a monitoring program

could be designed to detect contaminants moving at the boundary.

Dr. Charlesworth will also agree with Mr. Feenstra's comment with respect to monitoring Site D, that there could be "off-site migration through the granular units in a variety of directions ... we are not precisely sure in what direction it would go". This he noted is to be contrasted with Site F where he noted, it is clear in which direction the monitoring should occur.

Dr. Charlesworth will agree with Mr. Feenstra, Vol. 46, pp. 10467-70, that the same monitoring concerns that would be applicable to Site F are applicable to Site D, i.e. the need for stainless steel pipes, the need to monitor with the same frequency, the need to monitor the same parameters, etc.

SITE "D"

23. Dr. Charlesworth and Dr. Rowe will discuss their respective October 1987 Reports with regard to Site D.

24. Dr. Charlesworth will comment with respect to a question by Mr. McQuaid and response by Mr. Sobanski at Vol. 34, pp. 7394-7400 with respect to the direction of movement of water in the Upper Granular Unit as to whether or not wells along First Line which derive their water from that unit may be affected by leachate reaching that unit. He will

indicate, as is noted at page 5-3 of his report on Site D, October 1987, that because THCL's data on this is problematic or lacking, it is reasonable to assume that flow direction in the Upper Granular Unit would show some similarities to the flow pattern in the upper till (Exhibit H39, Drawing 17) i.e. have a westerly component within part of the unit. Further, mounding within the landfill would increase the gradient in that direction and hence the flow.

25. Dr. Charlesworth will also comment that, as reported in his October 1987 Site D report, there are upward gradients from the bedrock, it becomes critical in assessing the impact at Site D to be able to define the extent, location and role of the granular units within the groundwater flow system. Contrary to Exhibit B207 which suggests a large proportion of the flow from Site D would be to the shale bedrock, a vast majority of the flow, under present conditions, will be through surficial units. Even with a landfill at this site, which produced mounding, Dr. Rowe will indicate that less than 1.5%, and typically about 0.7% of the leachate would actually pass to the shale.

26. Dr. Charlesworth will refer to Mr. Crysler's description of ponds and surface drainage at and around Site D (Vol. 41, pp. 9164-84) and will refer to Exhibit M102. Dr. Charlesworth will indicate that the streams and ponds shown

on this aerial photograph could be fed by groundwater discharge even if they go dry at certain times of the year, i.e. they could be fed by flow from the weathered upper till and/or the Upper Granular Unit.

27. Dr. Rowe will indicate that he agrees with Mr. Hiraishi's evidence to the effect that "a prudent engineer considers what are the implications in the event of a failure" of an engineering system. Dr. Rowe's analysis as contained in his October 1987 Report on Site D and the further information contained in this Witness Statement are an attempt to consider reasonable failure scenarios in accordance with that dictum.

28. Dr. Rowe will indicate that, with respect to Site D, he modelled three values for hydraulic conductivity of the upper till. One of these (8×10^{-8} cm/s) is the value used by THCL. Both Dr. Rowe and Dr. Charlesworth will indicate that for various reasons, such as: smearing of the soil around peizometers; variability of the upper till; the accuracy of Hvorslev values, it is prudent to consider other values for hydraulic conductivity of the upper till.

29. With respect to modelling conducted by Dr. Rowe at Site D, Table 2 of his October 1987 Report contains data which, among other things, indicates the percentage of flow

to the shale bedrock, if one assumes that the shale acts as an underdrain to the system. In stipulating the percentage flows to the shale in Table 2 of his Report, Dr. Rowe had not directly modelled the flow in the shale itself, but rather, simply assumed that the shale has a capacity to act as a drain for whatever volume of groundwater arrived at the shale bedrock. Subsequently, however, Dr. Rowe has in fact modelled the flow through the shale using the hydraulic conductivity and thickness of the shale given by THCL. His modelling indicates less than 1.5% (and typically 0.7% or less) of the flow or recharge goes to the shale. The vast majority of any leachate recharging to the site from the landfill will pass from the site through the Upper Granular Unit (approximately 95% will discharge through the Upper Granular Unit under the cases modelled).

30. Dr. Rowe, in discussing his analysis with respect to Site D, will summarize his findings with respect to three independent scenarios discussed in his October 1987 report. He will also indicate the time it would take until impact occurs off-site, including consideration of the time it would take to develop the leachate mound. In addition, he will summarize how long it will take for contaminant transport off-site once the contaminants reach the Upper Granular Unit.

31. He will show that for the worst case scenarios,

even though it might take fourteen years to fully develop the mound, contaminant can be escaping off-site during the period that the mound is developing and, in fact, for the worst case scenario, contaminant impact could be detected off-site within only fourteen years of the beginning of the development of the leachate mound.

32. His analysis also indicates that once leachate arrives at the Upper Granular Unit, the time for it to travel 100 metres from the fill area is, in some cases, very short (the range being from 6 months to 12 years, with the majority of cases analyzed being 3 years or less.)

33. Dr. Rowe will discuss the length of time during which the hydraulic trap must continue to function as designed (and assuming the nominal hydraulic conductivity of the waste is 10^{-4} centimeters per second or greater) if off-site impacts are to be prevented in terms of additional chloride concentrations being introduced to adjoining wells. He will indicate that, having regard to the chloride values already present in adjoining wells (see Table 1, Geocon October 1987 Report on Site D) which are in excess of Ministry of the Environment drinking water objectives, and having regard to the Ministry of the Environment Reasonable Use Policy (Exhibit E154) which provides that where groundwater quality is poorer than drinking water objectives

but the groundwater is nonetheless in use as a drinking water source, "no decrease in water quality will be acceptable" due to the establishment of a landfill. Accordingly, the leachate collection system must continuously function as designed for more than 80 years to reduce an assumed initial chloride concentration of 1500 mg/L in the landfill to 10 mg/L at the site boundary.

Assuming 25 mg/L additional concentration was acceptable, it would take approximately 65 years for an initial landfill concentration of 1500 mg/L to be attenuated.

34. If the initial chloride concentration in the landfill were, for example, 3000 mg/L it would take more than 80 years to reduce the boundary impact to 10 mg/L and 75-80 years to reduce it to 25 mg/L.

35. The time periods given above do not reflect a worst case situation in terms of the length of time required to reduce initial chloride concentrations to the levels specified. The times referred to were calculated assuming a hydraulic conductivity of the upper till given by THCL, i.e. 8×10^{-8} cm/s. The time the hydraulic trap would be required to operate to achieve the reductions contemplated would be longer if the hydraulic conductivity of the upper till was greater.

For example, assuming the hydraulic conductivity of

the upper till was 4×10^{-7} cm/s and assuming the initial chloride concentration was 1500 mg/L, it would take approximately 70-75 years for that initial concentration to be reduced to the point where the off-site impact would be less than 25 mg/L and more than 80 years to reduce it to less than 10 mg/L.

36. Dr. Rowe will address the topic of the clogging of drains such as are to be used in the leachate collection systems at either proposed site. Dr. Rowe will indicate that the potential for clogging is of greater significance at Site D than at Site F. He will indicate that the potential for the clogging of drains is an important engineering consideration applicable to a wide range of geotechnical applications. He will indicate that drains and filters for a landfill are one of the most difficult design environments; and that very limited research has been conducted into long-term performance of landfill drainage systems. Accordingly, there is a need to consider the potential for failure of such systems and to develop a design, construction and maintenance program which will minimize the potential for failure and which will minimize the impact of a failure.

37. He will indicate that clogging of drains may occur (1) in the drainage pipe; and/or (2) in the granular filter around the pipe.

38. Clogging of either a granular filter or a geotextile wrap around the leachate collection pipe may occur due to:

(1) the migration of fines from the waste;

(2) precipitation (both chemical and biochemical)

(3) biological growth.

Dr. Rowe will indicate that it is very difficult to "clean" the filter around the pipe.

39. He will also indicate that clogging of the pipes may occur due to:

(1) structural failure;

(2) sedimentation;

(3) precipitation (both chemical and biochemical); and

(4) biological growth.

40. Drains have been observed to clog (due to iron clogging) within one month of being laid (e.g. Puig et al (1986) - see Appendices to this Witness Statement).

41. The hydraulic conductivity of filters may be substantially reduced due to migration of fines from the waste. Geotextile filters have had reductions of hydraulic conductivity of 5 orders of magnitude when tested with leachate passing through (Cancelli and Cazzuffi, 1987 - see

Appendices to this Witness Statement).

42. Dr. Rowe will indicate that good engineering practice would be to:

(a) Carefully design the filter medium using test data obtained by using leachate which contains fines from the proposed daily cover material as well as particulate matter from leachate itself;

(b) Preferably use a leachate collection system which is gravity controlled;

(c) Lay pipes in straight lines, preferably with openings every 30 to 50 metres through which a fire hose can be inserted to flush the pipe;

(d) Provide manholes at changes in direction and at intervals of 100 to 150 metres along straight sections;

(e) Have as high a gradient as possible within the drain (2% or greater is preferred);

(f) Design a system such that if some or all of the drains fail, the impact will be as small as possible.

43. In the context of discussing good engineering practice in respect of drains and leachate collection systems, Dr. Rowe will review applicable portions of Exhibit M267 and THCL's design for Site D. For example, based on Exhibit H41, Table 7 and Exhibit H41, Figure 2, if leachate is not pumped until the critical elevations set out are

reached, leachate will be backed up in all of the collection tiles and granular filters over the entire site. If this occurred, it would present a scenario similar to that which is reported for the Boone County, Kentucky landfill where clogging of the granular filter was reported after leachate had been allowed to back up in the granular filter (Exhibit M257, Tab 1, p. 21).

44. Dr. Rowe will also comment on Mr. Hiraishi's evidence with respect to the proposed leachate collection systems. Dr. Rowe will indicate that with respect to Mr. Hiraishi's response to Dr. Kingham (Vol. 4, p. 10641) as to whether total failure of the underdrain system can occur inside the pipe and also within the granular filter material, Dr. Rowe will indicate he agrees with Mr. Hiraishi that failure could occur in both such that the leachate system in effect becomes non-functional.

45. Dr. Rowe will indicate that to the extent the operations at Site D would not continuously involve the pumping and removal of leachate from the leachate collection system, this could produce conditions conducive to mechanisms of clogging of the leachate collection system, particularly in the absence of an effective slope in the pipes.

46. Dr. Rowe will refer to Mr. Hiraishi's testimony

(Vol. 48, pp. 11024-5) with regard to Mr. Hiraishi's experience at the Oakville landfill site where biological clogging of a vertical leachate collection system pipe occurred. Dr. Rowe will indicate that this exemplifies the need to be concerned about clogging of the leachate collection system when that leachate collection system is essential to the prevention of off-site impacts as it is at Site D.

47. Dr. Rowe will indicate that he agrees with the statement:

> "There are currently no techniques available for the maintenance of the drainage envelopes and filter layers. The effect of flushing is limited to the pipe"

EPA 267 Tab 3 p15

and that he is of the opinion that it is highly speculative to suggest, as opined by Mr. Hiraishi (Vol.49, p. 11113) that one could obtain effective flushing through a backwashing process.

48. With respect to Mr. Hiraishi's response in re-examination (Vol. 51, p. 11711) that there is redundancy in his proposal at Site D with respect to oversizing of leachate collection system pipes, Dr. Rowe will indicate that oversizing of the pipes, while a positive measure, would not prevent clogging of the granular or geotextile filter around

the pipes; if the filter mechanism becomes clogged, the pipes are rendered useless.

49. Dr. Rowe will indicate that he agrees with Mr. Hiraishi that if the maximum mounding above the base of the landfill is to be maintained at a head of 1.5 metres, there is no redundancy in the spacing of the leachate collection pipes as supposed. (Vol. 48, p. 10995). Further, Dr. Rowe will indicate that while a maximum head of approximately 2.6 metres of leachate could mound at the base of the landfill before the hydraulic trap would be reversed, nevertheless, if just one of the collection tiles in a particular cell clogged, such that the spacing between two collection tiles was approximately 60 metres, the maximum head at that location would be approximately 3 metres and accordingly the hydraulic trap would not be functioning at that location. Accordingly, Dr. Rowe will conclude in respect of that topic, contrary to Mr. Hiraishi's opinion (Vol. 48, p. 10994) that there is no significant "safety factor" with respect to clogging based on THCL's design assumptions.

50. With respect to Mr. Hiraishi's response, Vol.51 , p. 11714) that the leachate collection system would collect leachate without a slope and that accordingly any slope at Site D is a "redundancy" because mounding would still cause the leachate collection system to collect the leachate, Dr.

Rowe will indicate that it is that very scenario of "mounding" that he is concerned about and that it is not conservative to assume "mounding" can occur without having negative implications for reversing the hydraulic trap and creating off-site contamination.

51. With respect to Mr. Hiraishi's response (Vol. 51, p. 11714) that by providing access to leachate collection system pipes he has designed into the system "another redundancy", Dr. Rowe will review Exhibit H-41-D-2, Mr. Hiraishi's drawing where Mr. Hiraishi himself indicated that approximately one-third of the site where garbage would be deposited would not be accessible by way of manholes (in particular with regard to cells 2, 3 and 4, and even cell 5 as designed).

52. With respect to Mr. Hiraishi's comment (Vol. 51, p. 11717) that there is a need to provide proper quality assurance and quality control as well as construction techniques that will limit and minimize problems with pipe breakage and premature clogging, Dr. Rowe will indicate his agreement with that statement. He will also indicate that this is a concern which is much more crucial at Site D than at Site F given that the leachate collection system must operate properly to prevent off-site contamination at Site D.

53. With respect to Mr. Hiraishi's comments (Vol. 51, p.11718) that regular preventative maintenance is required and that this program "may include cleaning", Dr. Rowe will indicate that:

(a) it is difficult if not almost impossible to envision proper or adequate cleaning of the filter material around the pipe; and

(b) that while it is desirable at both sites, a regular preventative maintenance program would again be much more critical at Site D than at Site F.

54. With respect to Mr. Hiraishi's comment (Vol. 51, p. 11720) that monitoring for Site D will occur "as part of monitoring the manholes", Dr. Rowe will indicate:

(a) that he agrees with Mr. Hiraishi that observing flow in manholes will not allow conclusive determination of whether failure of the collection tiles has in fact occurred;

(b) that he agrees with Mr. Hiraishi's comments (Vol. 49, p. 11187) that:

> "It would be very difficult to know whether in fact the flows that you see are in fact the flows that you had anticipated to see. There are no guarantees on what you observe in fact are what you anticipated in the first place."

(c) that monitoring of the manholes is unlikely to detect

most of the failure scenarios examined by Dr. Rowe in his October 1987 Report on Site D.

55. Dr. Rowe accordingly will indicate he does not agree with Mr. Sobanski (Vol. 29, p. 6173) where Mr. Sobanski indicates that in his view looking at leachate volumes would give an indication of total failure. Dr. Rowe will indicate that even with a total failure of all collection tiles in, for example, cells 4 and 5, there still could be leachate being collected over the failed section of the landfill in a range of 10 to 76% for the cases considered, and that as a percentage of the total leachate being generated between 58% and 98% would be collected in the manholes.

56. Dr. Rowe will reach the following conclusions with respect to Site D:

(a) If the hydraulic trap functions as intended and if the hydraulic conductivity of the waste is 10^{-4} cm/s or greater and if the collection tiles continue to work for more than 80 years without significant blocking or clogging, then given the data provided by THCL and the current state of knowledge, one would not expect the landfill to impact on off-site groundwater quality;

(b) If the hydraulic conductivity of the waste is 10^{-5} cm/s then significant off-site contamination of

groundwater is to be expected (530-760 mg/L for chloride, assuming an initial concentration of 1500 mg/L)*; or

(c) If the leachate collection tiles which cannot be cleared (i.e. between manholes) block/clog (e.g. in cell 4), then significant contamination of off-site drinking water could be expected (390-590 mg/L chloride assuming an initial concentration of 1500 mg/L)*; or

(d) If all the collection tiles fail in cells 4 and 5 (but subheaders continue to function) one would expect significant contamination of off-site groundwater (810-1260 mg/L chloride, assuming an initial concentration of 1500 mg/L*;

(e) If the Upper Granular Unit is not continuous then contaminant could move off site without being detected;

(f) Having regard to conclusions (b) to (e), there is a reasonable likelihood that groundwater contamination will occur off site due to the proposed landfill at Site D.*

---

* Assuming that contaminant has not been detected in the Upper Granular Unit and that effective contingency measures have not been implemented.

COMPARISONS - SITE "D" & SITE "F"

57. Through overhead projections, Drs. Rowe and Charlesworth will present a comparison of the two sites based on THCL's data and design (see appendices to this Witness Statement) and also provide a comparison of impacts calculated at both sites.

58. With regard to Site F, the maximum impact of chloride at 800 metres downgradient (close to the CNR tracks) would be about 150 mg/L, and this assumes a maximum mass of contaminant of 7,800 tonnes. This peak concentration is not reached until about 525 years after failure occurs.

59. Dr. Rowe will comment on an issue raised by Dr. Kingham with respect to comparing Exhibits B259 and Figure 3-32 from Mr. Feenstra's report, Exhibit H86. Dr. Rowe will indicate that a recalculation of the chloride concentration at Site F based on THCL's better fit model and ignoring the effects of the diffusion into the matrix, would produce a peak of less than 85 milligrams per litre at a time of about 550 years. Applying matrix diffusion at a distance 800 metres downgradient of the landfill, the concentration would be less than 60 milligrams per litre. Dr. Rowe will indicate that Exhibit B259 is equally applicable to chloride concentrations at the site boundary in the case of Site F,

assuming the better fit model.

60. In contrast, at Site D, the range of peak concentration at the site boundary is between 390 and 1260 mg/L, and this assumes a total mass of contaminant of only 4,400 tonnes. Further, the time it would take to reach peak concentrations at Site D is almost an order of magnitude shorter (typically 35-66 years).

61. Based on available information and their own reports concerning the proposed sites, Drs. Rowe and Charlesworth will elaborate on their opinion that Site F is superior to Site D from the perspective of potential impact on groundwater quality. They will indicate that the performance of the two existing landfills adjacent to Site F and the results of the various studies relating to the proposed site indicate that the impact of the proposed landfill on groundwater quality at points south of Highway 403 would, even in the absence of an underdrain leachate collection system, be negligible.

62. In contrast, the proposed design at Site D will only prevent contamination of groundwater if the so-called hydraulic trap can be developed and maintained under the entire site for at least 80 years (and perhaps much longer) However, there are numerous ways in which this system or a

part of it could fail. Three possible failure scenarios are outlined in Dr. Rowe's October 1987 report and this witness statement.

The contingency measures proposed by the THCL for Site D are intended to deal with contamination of the groundwater not if, but when, the collection system and/or hydraulic trap fail. Given the complexity of the hydrogeologic system at Site D and the uncertainty about the likely flow paths, the proposal to develop Site D presents reasonable concerns that contaminants will impact off-site before any detection of failure has occurred. Unlike at Site F, undetected off-site migration at Site D will have a direct adverse impact on groundwater downgradient of the landfill.

At Site F, in the event that the upper leachate collection system fails, there is a secondary leachate collection system capable of underdraining at least a portion of the site. Further, most leachate could be collected by perimeter ditches and toe drains, whereas at Site D a perimeter ditch would only collect leachate tending to go laterally through the weathered upper till. If the leachate collection system at Site D or part of it fails, the leachate plume(s) must be detected and satisfactory engineering devices installed to prevent off-site impacts.

63. Dr. Charlesworth and Dr. Rowe will comment with respect to Exhibit H-263. With regard to the three scenarios

summarized by Mr. Feenstra in that exhibit, they will indicate that with regard to Scenario 1, Site D, they disagree; under certain conditions, even with the primary engineering features in operation, there could be significant impact at Site D.

With respect to Scenario 3, their general comment is that even if there is a failure of the underdrain leachate collection system at Site F, it is unlikely that significant and complex engineering must be undertaken at Site F to prevent any off-site migration of leachate; only surface discharge must be controlled. On the other hand, at Site D, the leachate plumes must be traced and, if leachate has already reached the Upper Granular Unit, rapid and complete interception of off-site migration must occur in order to prevent off-site impacts.

They will also suggest a fourth scenario should be considered: failure of the engineering features and either failure to carry out adequate monitoring or failure of contingency measures. In this fourth case, the impacts would be similar to those for Scenario 2 in Exhibit H263, i.e. significant impact at Site D and no significant impact at Site F.

They will indicate that Scenario 3 is not as likely to occur as Mr. Feenstra indicates, not because contingency measures are impossible, but rather:

(a) The escape of leachate at Site D may not be detected

completely or at all before it reaches the Upper Granular Unit;

(b) Even if an indication of contamination is obtained through monitoring, those responsible for interpreting results may not know or believe that they are witnessing a failure (referring back to Mr. Sobanski's evidence on this topic);

(c) As Dr. Rowe's work indicates, once leachate arrives at the Upper Granular Unit, there may be relatively very little time in which to take effective remedial action;

(d) Even if remedial action is taken with respect to some detected contamination, because of the potential complexity of the Upper Granular Unit, i.e. fingering, there may be contamination occurring at other places which is not detected while the first bout of contamination is being dealt with.

64. Dr. Rowe will disagree with Mr. Feenstra's opinion given to Mr. McQuaid at Vol. 45, p. 10081, that there is reason to prefer Site D over Site F under operating conditions assumed at either site. He will disagree that at Site D "there would be no potential contaminant migration into the underlying granular unit" during operation of the site. He will also indicate that while at Site F there may be a "potential" for downward movement of contaminants, that under operating conditions the potential proportion of

leachate moving down into the shale is less than 0.3% under the "better fit model" assumptions and approximately 5% under THCL's earlier model.

65. From the standpoint of modelling contaminant migration, Drs. Rowe and Charlesworth will indicate that the proposed Site D is at least as complex as Site F and there is uncertainty regarding the value of relevant parameters at both sites. In both cases, modelling, using reasonable combinations of parameters, will provide an indication of the potential for contamination of the groundwater. At Site F, there is the unique advantage of having the documented performance of two existing landfills which can be used to calibrate the choice of parameters.

66. Drs. Rowe and Charlesworth will agree with Mr. Feenstra that it is appropriate to assume a significant level of confidence in the applicability of contaminant transport models at Site F compared to Site D:

> "The comparison of a field example or field observations with any type of calculation or model results I think would have to be considered as a reasonably significant confirmation of the estimates or calculations." [Vol. 46, p.10444]

67. They will also refer to Mr. Feenstra's response to a question by Mr. Garrod, Vol. 44, p. 9871, that the

existence of data about the plume delineation work at the Burlington site and also data at the Bayview site provides a higher level of confidence with regard to Site F than Site D in terms of its suitability for a landfill.

68. Dr. Charlesworth will comment with respect to the Ministry of the Environment Reasonable Use Policy (Exhibit E154). With respect to Site F he will indicate:

a) the most reasonable use of the groundwater regime downgradient at Site F and its environs is for waste disposal; the poor quality groundwater of this area is a good resource for the disposal of waste;

b) locating a landfill at Site F rather than at Site D will comply with the Policy that "waste disposal sites should be placed in environments where their impact will be minimized" [page 10] and the statement that it may be prudent to "discourage the use of some environments for waste disposal and encourage the use of other environments". [page 8]

69. With respect to Site D, Dr. Charlesworth will indicate that the Ministry of the Environment Reasonable Use Policy is applicable in that the groundwater resource is relied on and has been relied on for many years with respect to domestic and agricultural purposes. As indicated in his

October 1987 report on Site D, the wells analyzed in the Geocon report are at or exceed Ministry of the Environment criteria for certain parameters. Accordingly, pursuant to Section 3.5 of the Reasonable Use Policy, "no decrease in water quality will be acceptable".

70. Drs. Charlesworth and Rowe will comment on hydrogeological testimony and exhibits to date with respect to Mr. Sobanski's reply to Dr. Kingham, Vol. 40, pp. 8832-33, that:

> "The important layer is the upper unit ... there were some sand facies in one of the holes at the bottom ... but ... for the most part it was a solid clay till, without any evidence at all in our logs of any silt or sand seams that we saw."

They will indicate that a review of the THCL borehole logs makes it clear that that description of the upper till, the unit into which the refuse is to be placed, is not accurate, as discussed at page 5-10 of the Geocon report.

71. With respect to Burlington Exhibit 208, they will indicate that:

- 4 of the 10 samples were from locations not proximate to the area proposed for waste disposal;
- the relevant samples of upper till indicate the material ranges from a gravelly silt till, some

sand, some clay, to a clayey silt till, some sand, some gravel;

This unit is in fact on the borderline of being a clayey, sandy silt till. One would not expect the attenuation characteristics of this material to be as good as the clayey materials in the Sarnia area which have been referred to in the hearings.

72. Notwithstanding Mr. Sobanski's reply to Mr. McQuaid, Vol. 34, p. 7437, they will indicate that Exhibit M134, p. 397 of the book Groundwater by Freeze and Cherry, is relevant to predicting contaminant movement and to monitoring difficulties at Site D.

73. With respect to Mr. Feenstra's response to Mr. Tidball, Vol. 46, p. 10412, Drs. Charlesworth and Rowe will indicate that they agree with Mr. Feenstra when he stated:

> "I would say the fact that there are chemical concentrations in the back-ground wells, particularly WAT 2 and MB 14, the relatively low concentrations of contaminants detected in the Waterloo and Morrison, Beatty wells over a large area and a very great distance, up to a very great distance from the landfill site, combined with the fact that if we regard the data to truly reflect groundwater concentrations, I think we would be hard pressed to come up with a scenario of attenuation mechanisms that would allow the movement of trace concentrations those great

distances with very little change in concentrations. I would believe that the conclusion or interpretation is quite conclusive."

74. Similarly, Drs. Rowe and Charlesworth will agree with Mr. Feenstra's evidence (Vol. 43, pp. 9634-35) with respect to the fact that there is no particular pattern to the level of concentrations in the monitoring wells downgradient of the Burlington landfill; that the concentration of wells nearest the landfill are not higher than the concentrations detected 300, 600, or 700 metres downgradient, which would not be expected if in fact these chemicals were landfill derived. They also agree with Mr. Feenstra that if contaminant was escaping from the landfill, they would expect there to be higher concentrations closer to the landfill, particularly because the concentrations identified at the present time in the Burlington landfill leachate are very much higher than the concentrations seen in the groundwater. They would also expect to see lower concentrations at some distance from the landfill itself.

75. Dr. Charlesworth will indicate that with respect to data in Ms. Hewetson's thesis, Exhibit E214, that to the extent there were toe discharges of leachate from the Bayview landfill occurring at the time of that study, a number of monitors referred to by her, e.g. WAT3, MB11, MB12, WAT7, are in a location such that surface transport of such toe

discharge could take such leachate proximate to the locations of these monitors. Accordingly, any conclusions based on leachate "measurements" could be primarily the result of surface flow from the Bayview landfill as opposed to groundwater transport of such contaminants from the landfill. He will refer to photographs taken in 1987 of the location of those monitors to indicate how they are in low-lying areas generally along the valley floor.

76. Dr. Charlesworth will discuss exhibits such as N196a. He will indicate that given the various potential sources of low levels of volatile organics measured and the fact that there is no higher concentration of these proximate to the landfill than further away, he agrees with the conclusion, as suggested by Mr. Feenstra, that these are not necessarily related to contaminants originating in the landfill. Accordingly, measurements at locations or horizons that are suggested to be at the same depth or otherwise related but which are not in fact of the same value, does not provide a basis for concluding that the two landfill sites are behaving in a substantially different manner. He will express the opinion that it is still reasonable to predict the impact of a landfill if located at Site F based on observations in respect of the Burlington and Bayview Park landfills. Even though locally there may be variations of various kinds, it is still reasonable to predict on a large

scale from these two adjoining landfills. Local variations occur in other hydrogeologic environments, not just in the vicinity of Site F.

77. With respect to Dr. Kingham's questions (Vol. 40, pp. 8862-63) with respect to data contained in Exhibit N191 and N192, and the differences in values of sulphates and chlorides discussed, Dr. Charlesworth will indicate that variations in soluble minerals within the rock, such as gypsum, could account for widely different values for some of the different constituents. Differences in chlorides could be due to higher concentrations of water trapped in the matrix of the rock. Measurements of chloride values would be higher for water trapped in the matrix than if measured in the fractures themselves. In this regard, Dr. Charlesworth will address Dr. Kingham's question at page 8865 with regard to whether or not the variability in data does indicate "some underlying geological irregularity" (page 8865). Dr. Charlesworth will indicate, having regard to Dr. Kingham's question at page 8866 with respect to variability in local gradients and contours, that these are not the governing issue with respect to hydrogeological interpretation of the overall impact of the site.

78. Dr. Charlesworth will comment with respect to Exhibit E-109, the "Review Under the Environmental Assessment

Act" prepared by various ministries and, in particular will address the comment that the Ministry of the Environment would only accept Site F from a hydrogeological perspective "if the consultant can reasonably demonstrate that the leachate infiltrating into the shale will go to depth and will not come to the surface between the site and the lake". He will indicate that he is of the opinion this is what has been occurring around Site F and will happen at the proposed site.

79. With respect to the concern raised at page 50 of Exhibit E-109 (a concern raised prior to the THCL studies on Site F being completed) to the effect that Site F may require a liner, Dr. Charlesworth will indicate that on all of the available data it is clear that no liner is required.

80. Dr. Charlesworth will also present photographs taken in September 1987 south of Highway 403 in an area which THCL has suggested would be the "discharge area". The photographs indicate that a tributary of Indian Creek and surrounding environs are, at that time, dry. Dr. Charlesworth will indicate that if this was a discharge zone one would expect that a stream draining that zone to contain water.

81. Dr. Charlesworth will also refer to a series of

aerial photographs taken over a period of years commencing in 1934 of the area now developed as the Bayview and Burlington landfill sites. His review of these aerial photographs will indicate that large portions of the area of the present Burlington landfill site which does not have an underdrain system is developed on lands which were apparently mined for clay and for shale and, in any event, were stripped of their natural surficial till cover. These photographs also indicate the same is true with respect to the Bayview landfill site.

82. Further, a review by way of stereoscopic interpretation of aerial photographs indicates that both the southern end of the Burlington landfill site and the Bayview Park landfill site were not engineered to current standards as proposed for Site F and yet all monitoring to date indicates that they have operated with minimal impact on the groundwater regime.

APPENDICES TO JOINT WITNESS STATEMENT

		Tab
1.	Corrections to R. Kerry Rowe's Reports October 1987	1
2.	Overhead Projections to be Used in Evidence-in-Chief	2
3.	Recent Articles on Clogging of Drains:	3

Puig, J. et al "Ferric Clogging of Drains", Proceedings of the Third International Conference on Geotextiles, 1986, Vienna, Austria, p. 1179.

Cancelli, A. et al "Permittivity of Geotextiles in Presence of Water and Pollutant Fluids", Proceedings of the Geosynthetic 1987 Conference, New Orleans, U.S.A., p. 471.

CORRECTIONS TO R. KERRY ROWE'S REPORTS OCTOBER 1987

Site D

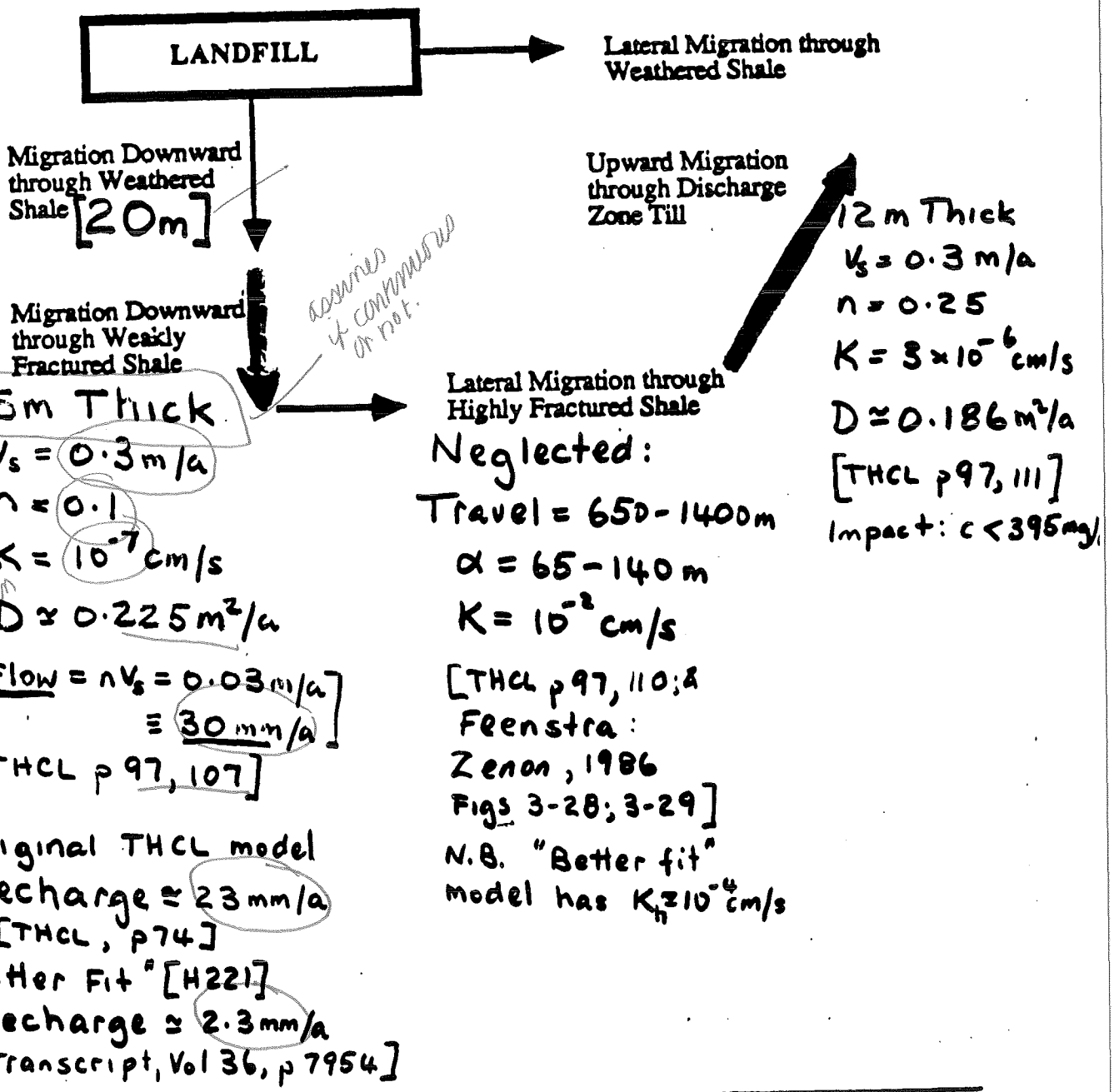
Page	Para	Line	Corrections
10	1	2	"landfills" should read landfilling
18	3	2	replace the word "piezometric" by "total"
21	1	3	replace the word "piezometric" by "total"
27	Table 2 Column 4		Delete "peizometric" 182.5 should read 185

Site F

9	4	5	insert words "if there was" after "Furthermore"
9	4	5	Insert "this" after "the till"

COMPARISON OF THCL'S ORIGINAL MODEL AND THE "BETTER FIT MODEL"

FIGURE 3-24. SITE F: SCHEMATIC OF GROUNDWATER CONTAMINANT PATHWAYS USED BY THCL



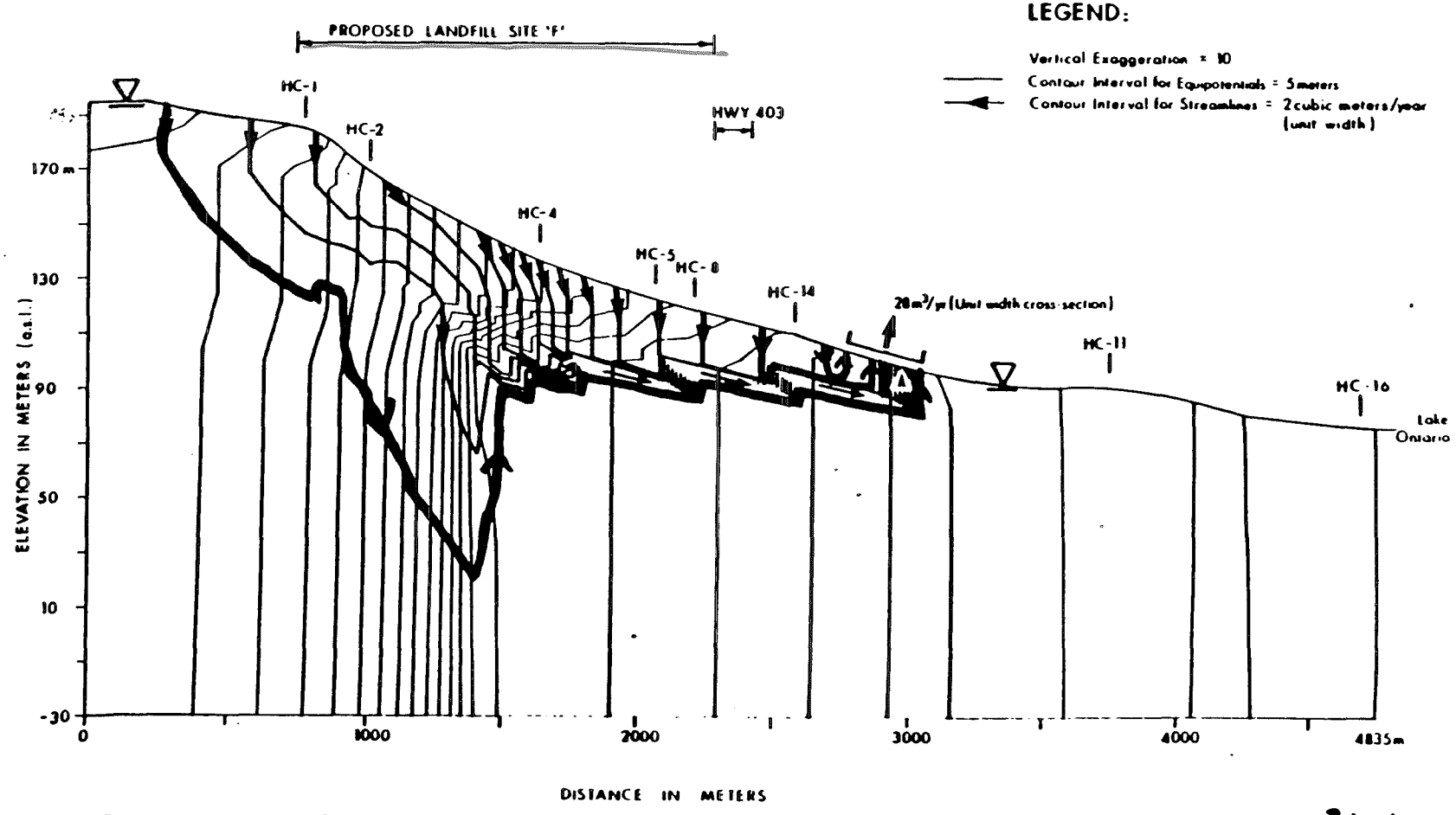
THCL -> Exhibit H40

From
Ex H40

FIGURE 6

COMPUTER SIMULATION OF GROUNDWATER FLOW SYSTEM

THCL's "Original" Simulation



Flow *Entering Flow System* and moving off site ≡ 20 $m^3/m/a$

Infiltration into landfill ≑ 0.305 × 1250 × 1 ≡ 380 $m^3/m/a$

∴ Proportion of Leachate moving offsite = $\frac{20}{380}$ = 0.05 (ie 5%)

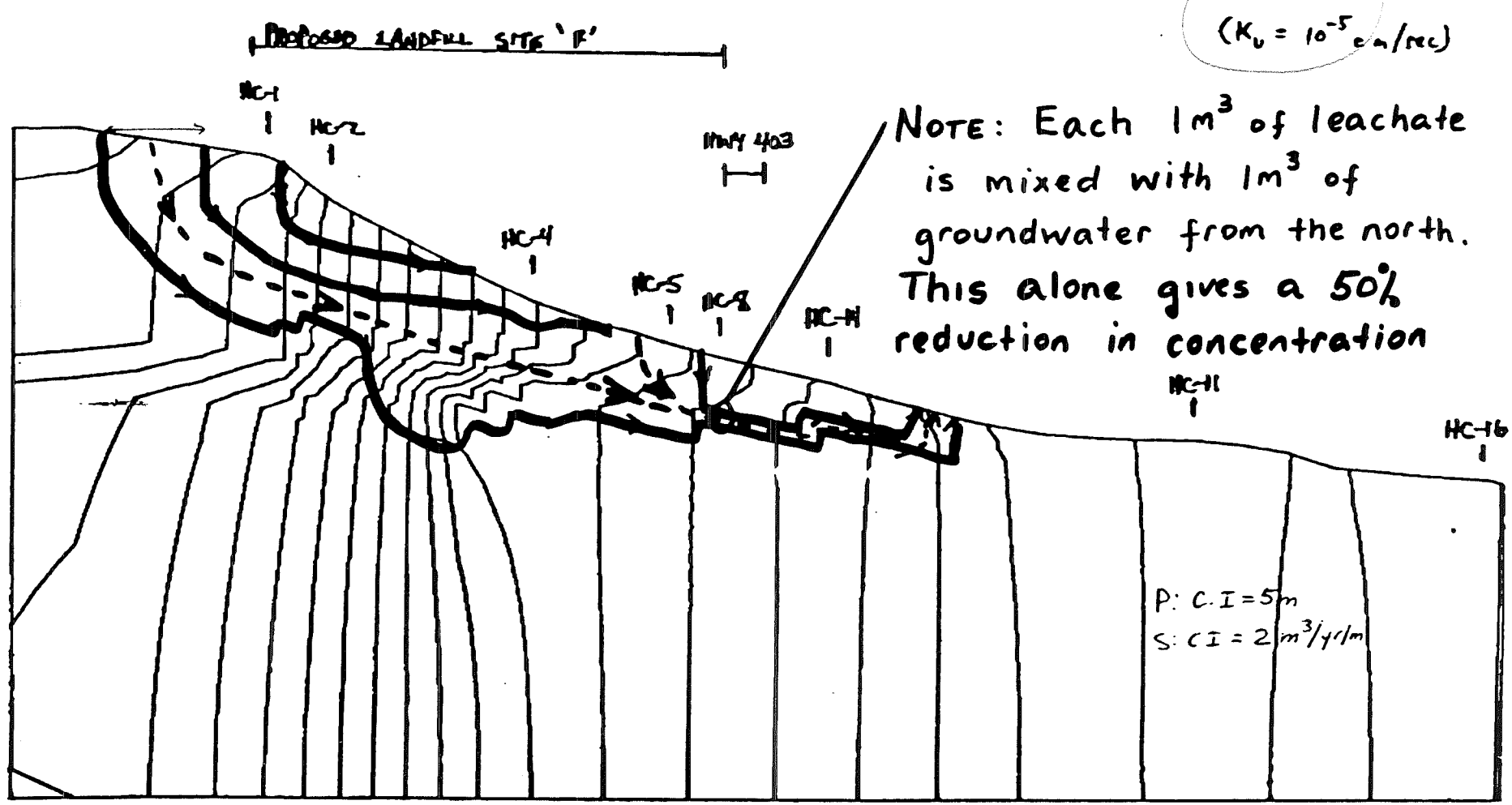
(95% Collected) won't fill

Trow

CALIBRATION WITH ANISOTROPY

THCL's "Better Fit Model"

K of highly fractured shale unit reduced to $K_h = 10^{-4}$ cm/sec ($K_v = 10^{-5}$ cm/sec)



NOTE: Each $1m^3$ of leachate is mixed with $1m^3$ of groundwater from the north. This alone gives a 50% reduction in concentration

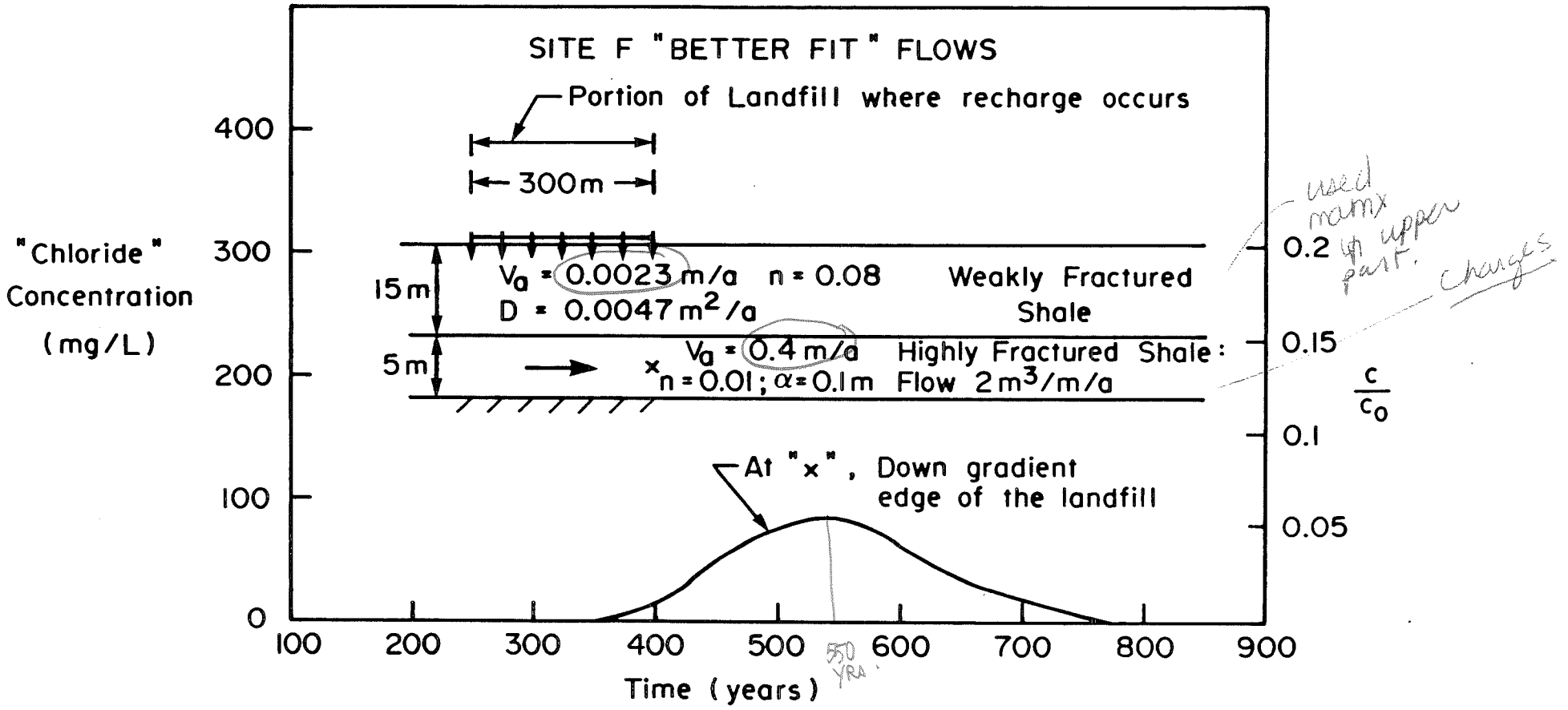
Flow entering flow system and moving off-site = $1m^3/m/a$

Infiltration into landfill $\doteq 0.305 \times 1250 \times 1$ $\equiv 380 m^3/m/a$

$\therefore$ Proportion of Leachate moving off-site $= \frac{1}{380} = 0.0026$

(ie 0.26%) 99.7% Collected

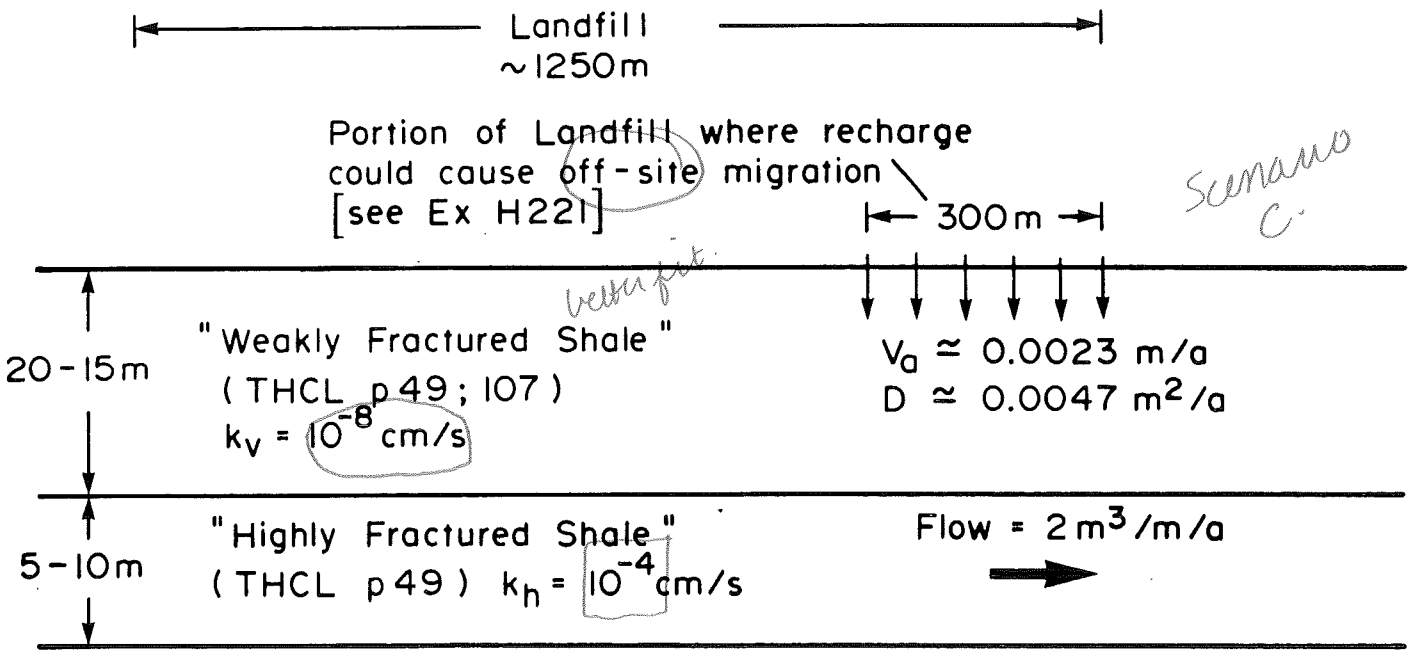
CALCULATED IMPACT OF SITE F ASSUMING FLOWS
FROM THCL'S "BETTER FIT" MODEL



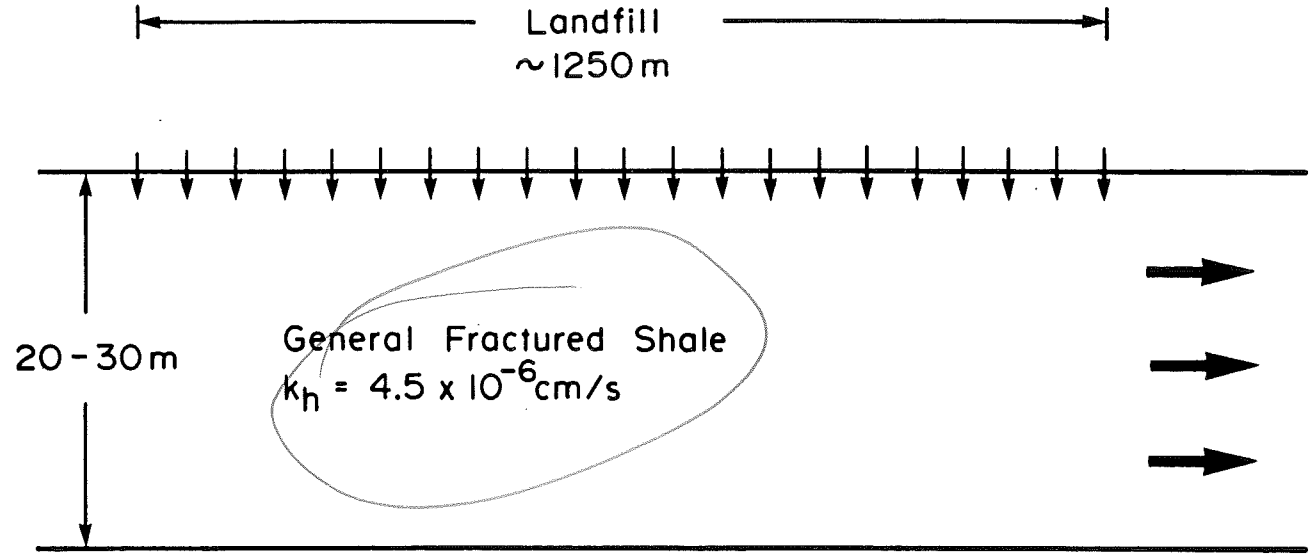
used namx ↑ upper part. changes

4400 50 ~3%
7800 85 65

TWO POSSIBLE MIGRATION SCENARIOS: SITE F



(a) Clearly defined distinct units:
Transport down through weakly fractured shale and then along "highly" fractured shale - Primary attenuation in the weakly fractured shale.



(b) General fractured shale unit:
Only consider attenuation due to matrix diffusion as contaminant migrates to the south.

GENERAL CONCLUSIONS

1. SPEED OF CONTAMINANT TRANSPORT IS MUCH SLOWER THAN THE AVERAGE LINEARIZED GROUNDWATER VELOCITY, v_F.

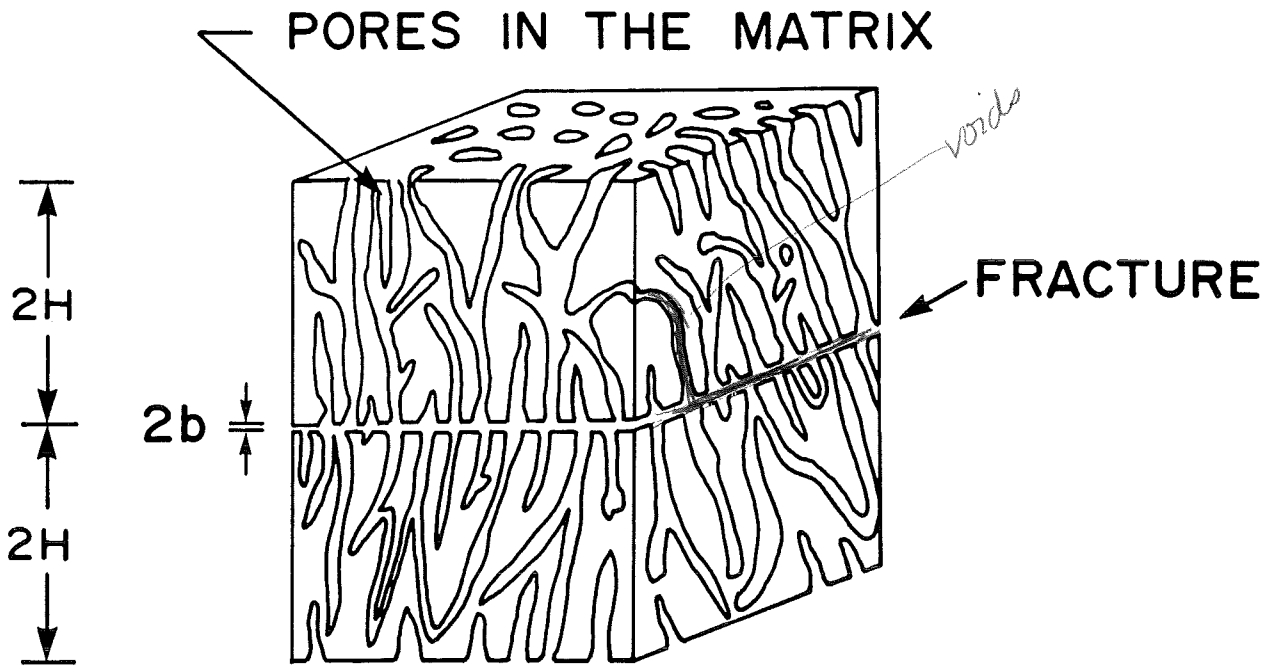
2. FRACTURE POROSITY AND AVERAGE GROUNDWATER VELOCITY HAVE RELATIVELY LITTLE EFFECT FOR TYPICAL PARAMETERS.

3. THE DARCY "VELOCITY" IS IMPORTANT. amount of flow through system.

4. THE MATRIX DIFFUSION COEFFICIENT AND FRACTURE SPACING ARE OF DECREASING SIGNIFICANCE AT LARGE TIMES REQUIRED FOR CONTAMINANT TO MIGRATE A SIGNIFICANT DISTANCE.

5. MATRIX DIFFUSION RESULTS IN SUBSTANTIAL ATTENUATION OF CONTAMINANTS.

SCHEMATIC SHOWING FRACTURES AND MATRIX PORES



Spacing between fractures = 2H

"Voids" in fracture = n_f : typical 0.001

Voids in matrix = n_m : typical 0.1*

$$\text{Porosity} = \frac{\text{Volume of Voids}}{\text{Total Volume}}$$

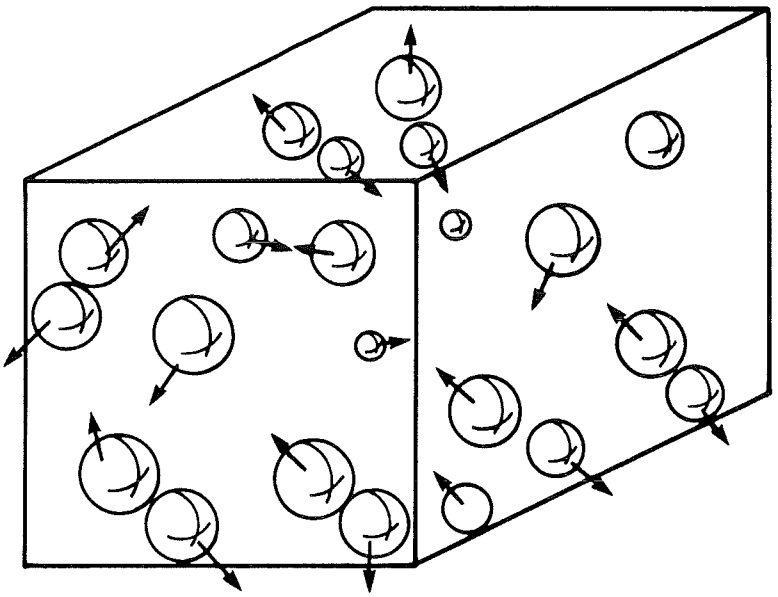
*At site F this was measured to be 0.1 - 0.11
see page 31

NOTE: If Fracture Porosity = 0.001
Matrix Porosity = 0.1

Then the matrix pores represent more than 99% of the total voids or pores in the rock.

DIFFUSION: THE PROCESS WHEREBY IONIC OR MOLECULAR CONSTITUENTS MOVE UNDER THE INFLUENCE OF THEIR KINETIC ACTIVITY IN THE DIRECTION OF THE CONCENTRATION GRADIENT

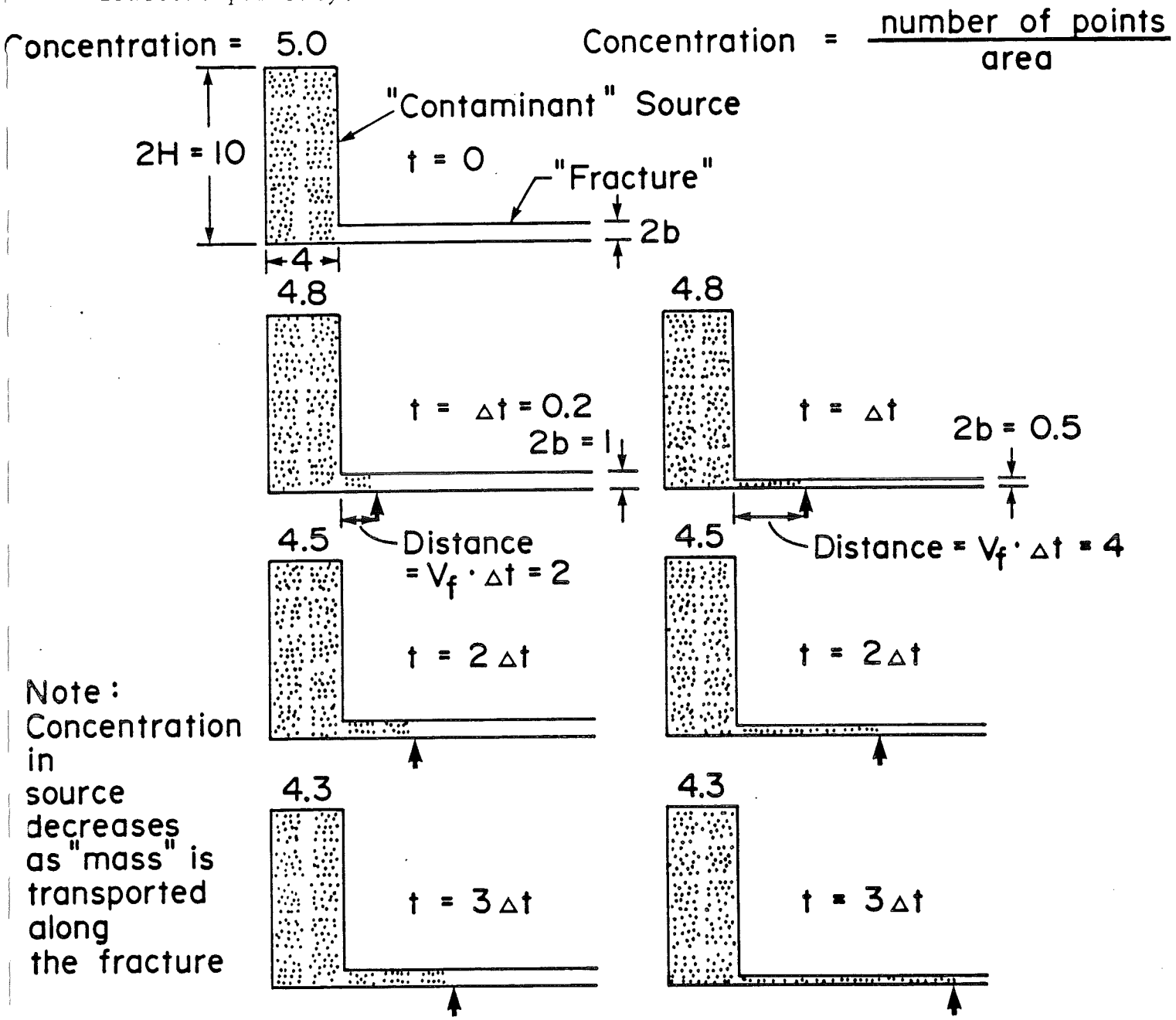
CONCENTRATION = MASS PER UNIT VOLUME



NOTE: If the concentration of contaminant in the water in a fracture is greater than the concentration of contaminant in the water in the pores of the rock then contaminant will diffuse (move) from the fractures into the pore water in the rock matrix.

SCHEMATIC SHOWING NO MATRIX DIFFUSION

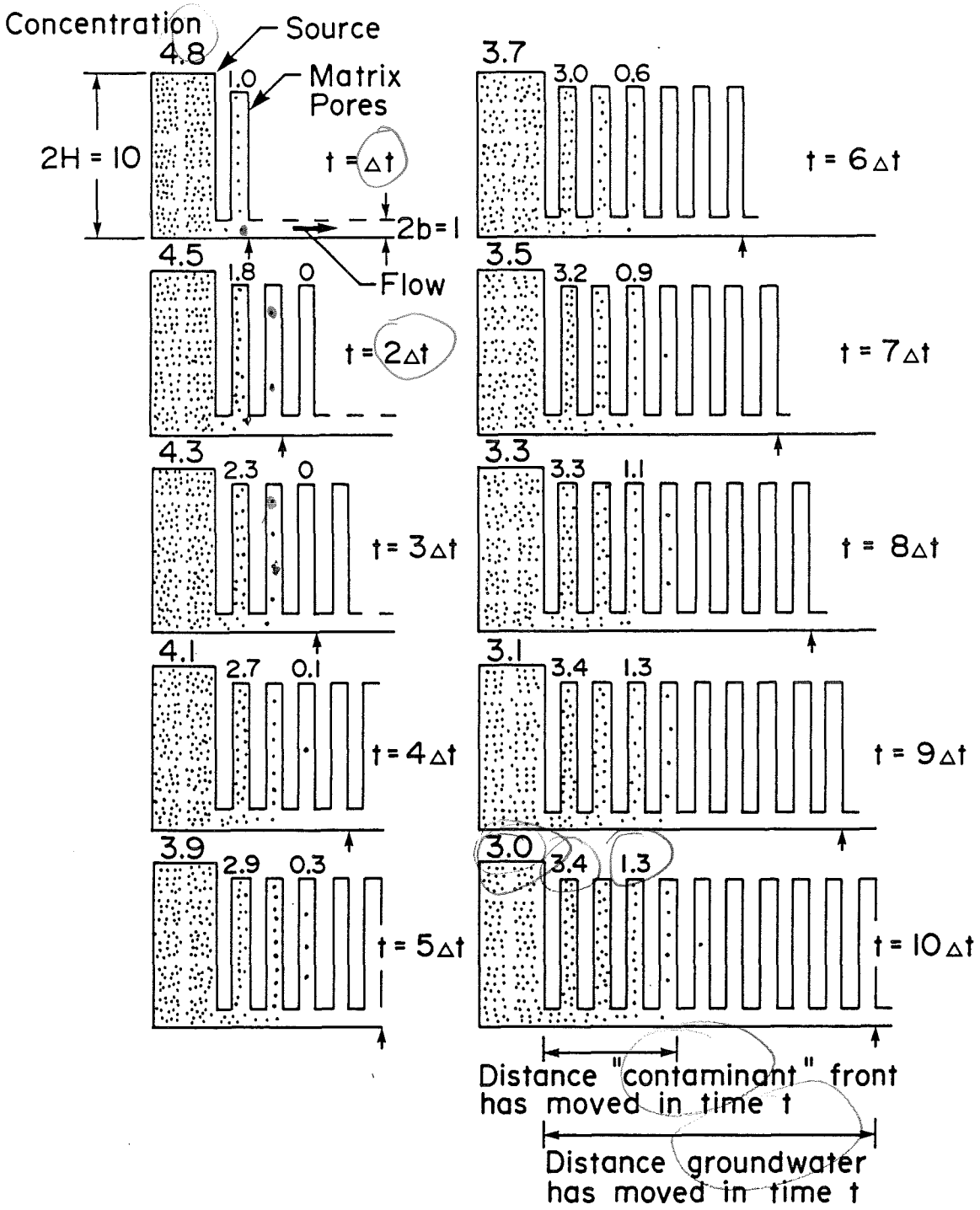
This shows that IF one neglects matrix diffusion and dispersion, THEN
(1) the speed of contaminant migration is equal to the groundwater velocity; and
(2) the speed of contaminant migration is inversely proportional to the fracture porosity.



	(a)	(b)
Darcy Velocity	$V_a = 1$	$V_a = 1$
Fracture Porosity	$n_f = \frac{2b}{2H} = \frac{1}{10} = 0.1$	$n_f = \frac{2b}{2H} = \frac{0.5}{10} = 0.05$
Groundwater Velocity	$V_f = \frac{1}{0.1} = 10$	$V_f = \frac{1}{0.05} = 20$

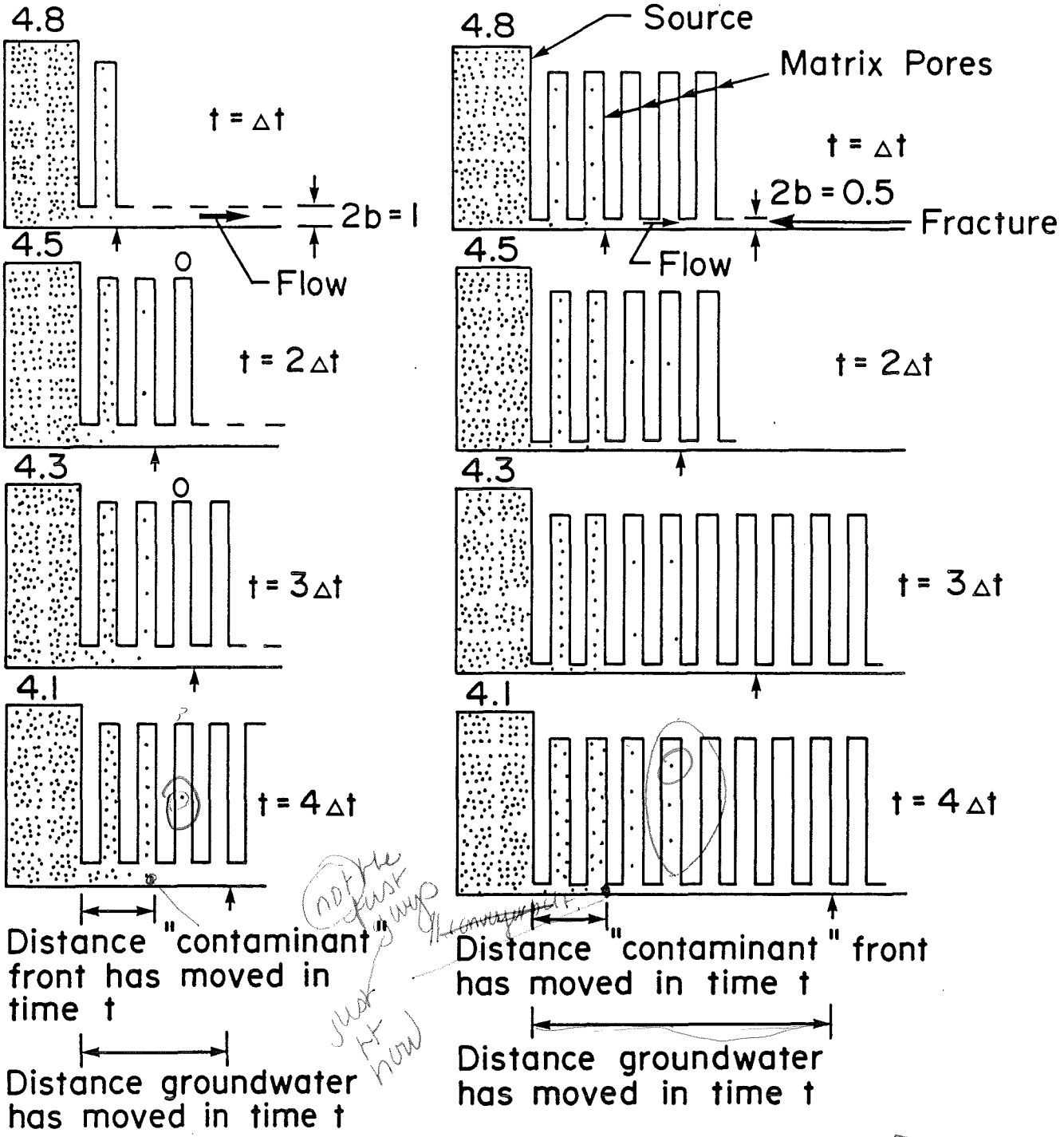
SCHEMATIC SHOWING "MATRIX DIFFUSION"

$D_m = \infty$



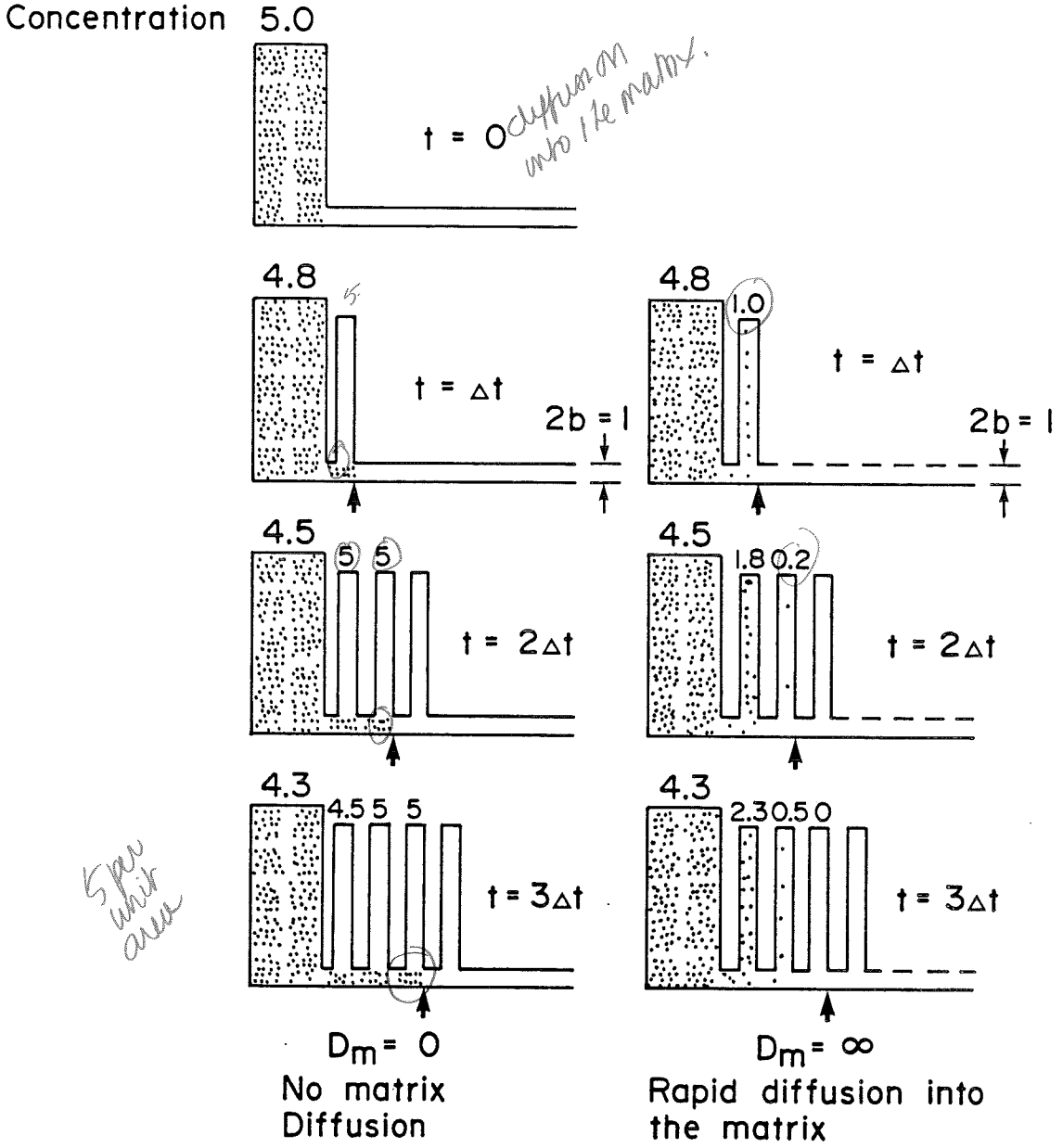
This shows that when one considers matrix diffusion contaminants DO NOT move at the same speed as the groundwater. Matrix diffusion substantially reduces the speed of contaminant movement along the fractures. Notice also that the maximum concentration in the fracture is less than the original source concentration.

SCHEMATIC SHOWING THAT IF ONE CONSIDERS MATRIX DIFFUSION THEN FRACTURE POROSITY HAS A MINIMAL EFFECT ON THE SPEED AT WHICH CONTAMINANT MOVES ALONG A FRACTURE



NOTE: Decreasing fracture porosity by a factor of two
(i) doubles the groundwater velocity
(ii) DOES NOT significantly change the location of the contaminant front or speed at which the contaminant migrates

SCHEMATIC SHOWING THE EFFECT OF MATRIX DIFFUSION COEFFICIENT



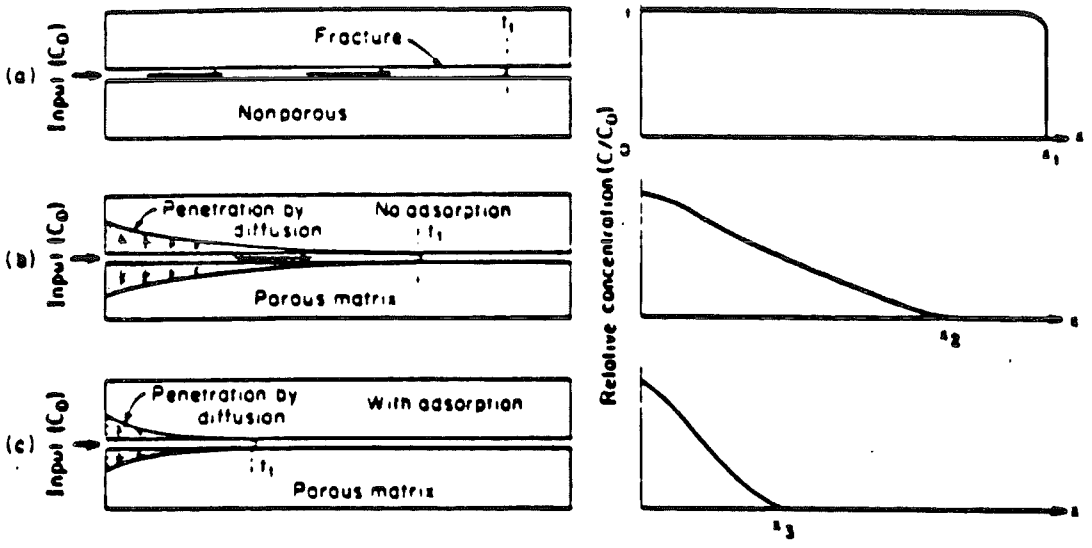
NOTE: (i) Zero matrix diffusion coefficient gives maximum concentration in the fractures.

(ii) Infinite matrix diffusion coefficient gives minimum concentration in the fracture.

(iii) It is therefore conservative to perform an analysis using a matrix diffusion coefficient less than the actual value in the field. This would result in the calculation overestimating the concentration of contaminant in the fracture and hence overestimating the potential impact on groundwater quality.

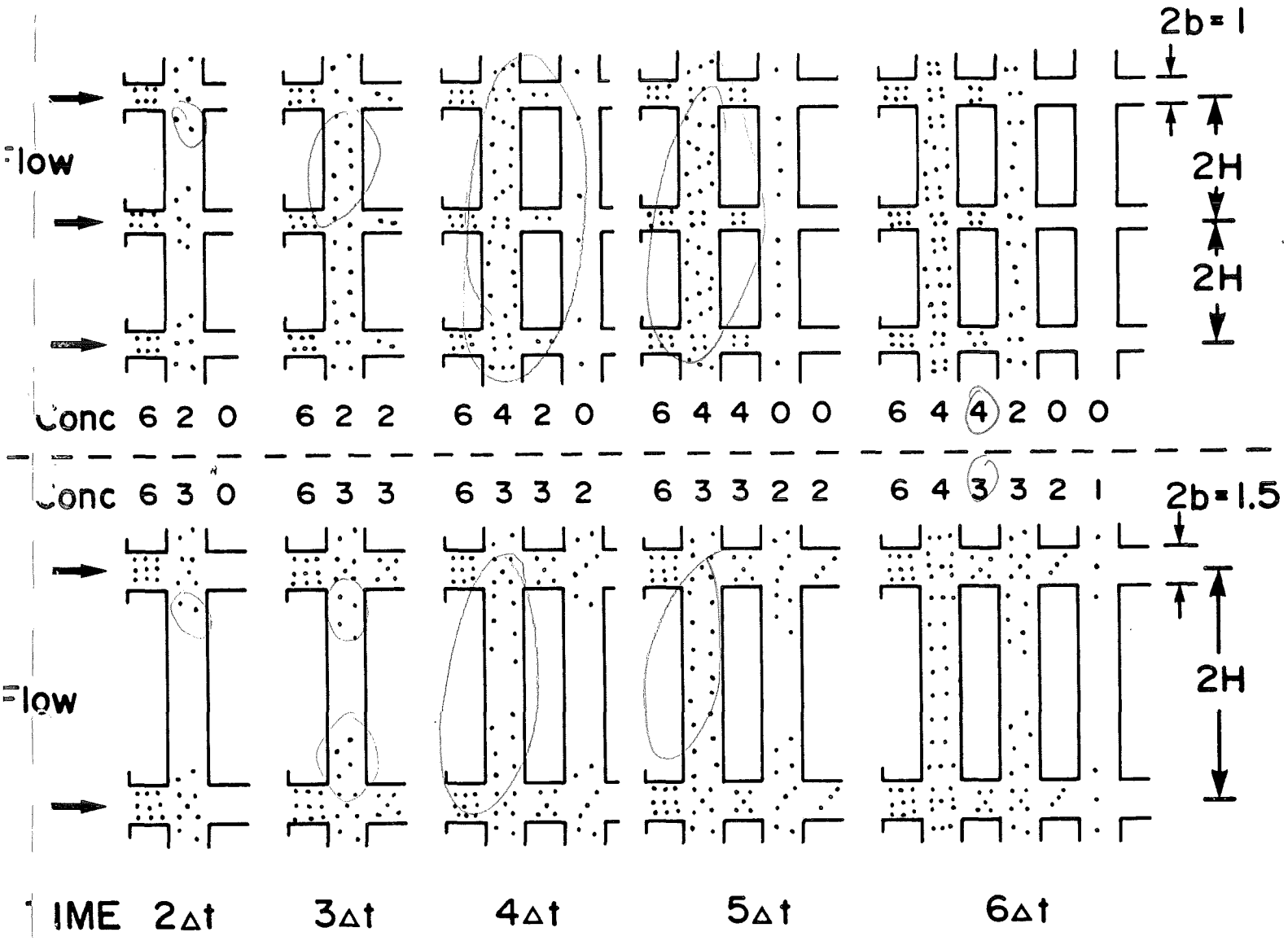
FIGURE 2 (FROM THCL, Site F Report, Vol. 1, Fig. 10, p. 88 - Original Source - Freeze and Cherry, 1979, p. 411).

ILLUSTRATION OF THE EFFECT OF MATRIX DIFFUSION (Cherry, 1979)



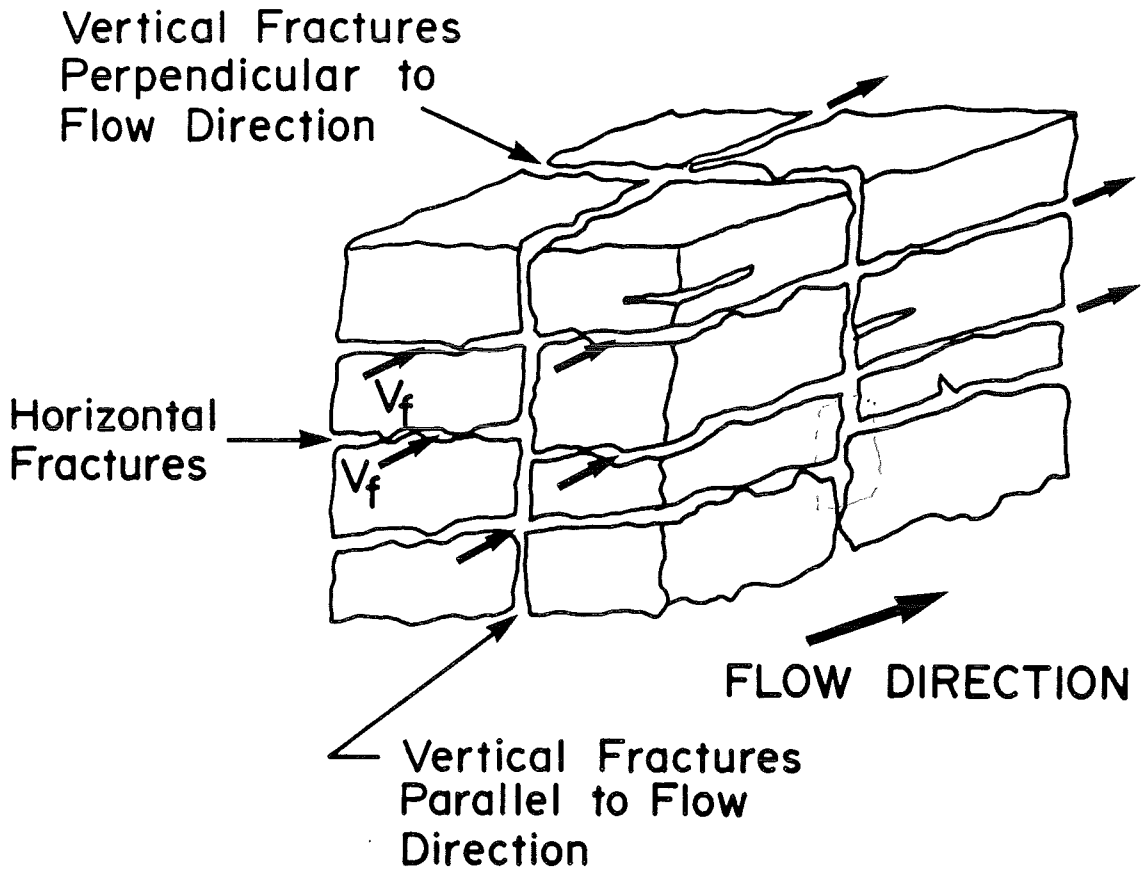
Effect of diffusion on contaminant migration in porous fractured medium. (a) Unidirectional hydraulic transport in a fracture in a nonporous medium; (b) unidirectional hydraulic transport with migration into matrix as a result of molecular diffusion; (c) unidirectional hydraulic transport with molecular diffusion and adsorption (profiles of relative concentration of reactive contaminant within fracture shown at time t_1).

Schematic Showing Effect of Fracture Spacing



NOTES:
(i) Fracture porosity is the same in both cases (however the width of the fracture is different)
(ii) This shows that a smaller fracture spacing reduces the distance the contaminant front has travelled in a given time but may give higher concentrations in the fracture than a wider fracture spacing.
(iii) After there has been sufficient time for contaminant to fully diffuse into the rock pores, fracture spacing ceases to have a significant effect on concentration in the fracture.

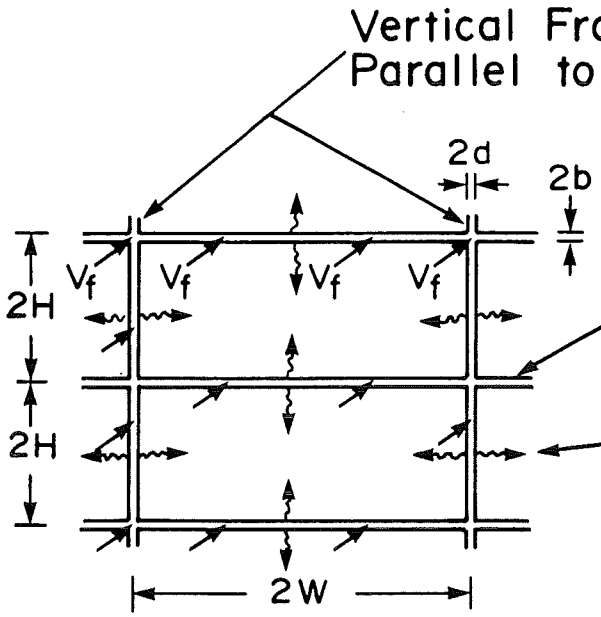
SCHEMATIC SHOWING A THREE-DIMENSIONAL FRACTURE SYSTEM



SCHEMATIC SHOWING 2D & 1D FRACTURE SYSTEMS

(Section Perpendicular to Flow Direction)

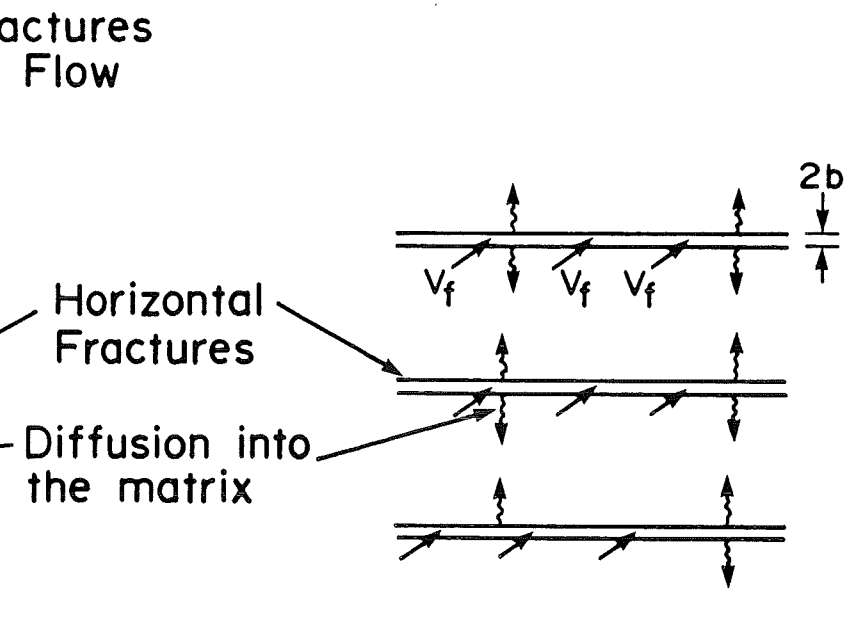
Two Dimensional Fractures (2D)



Fracture Porosity

$$n_f \approx \frac{2b}{2H} + \frac{2d}{2W}$$

One Dimensional Fractures (1D)



Fracture Porosity

$$n_f = \frac{2b}{2H}$$

NOTES:

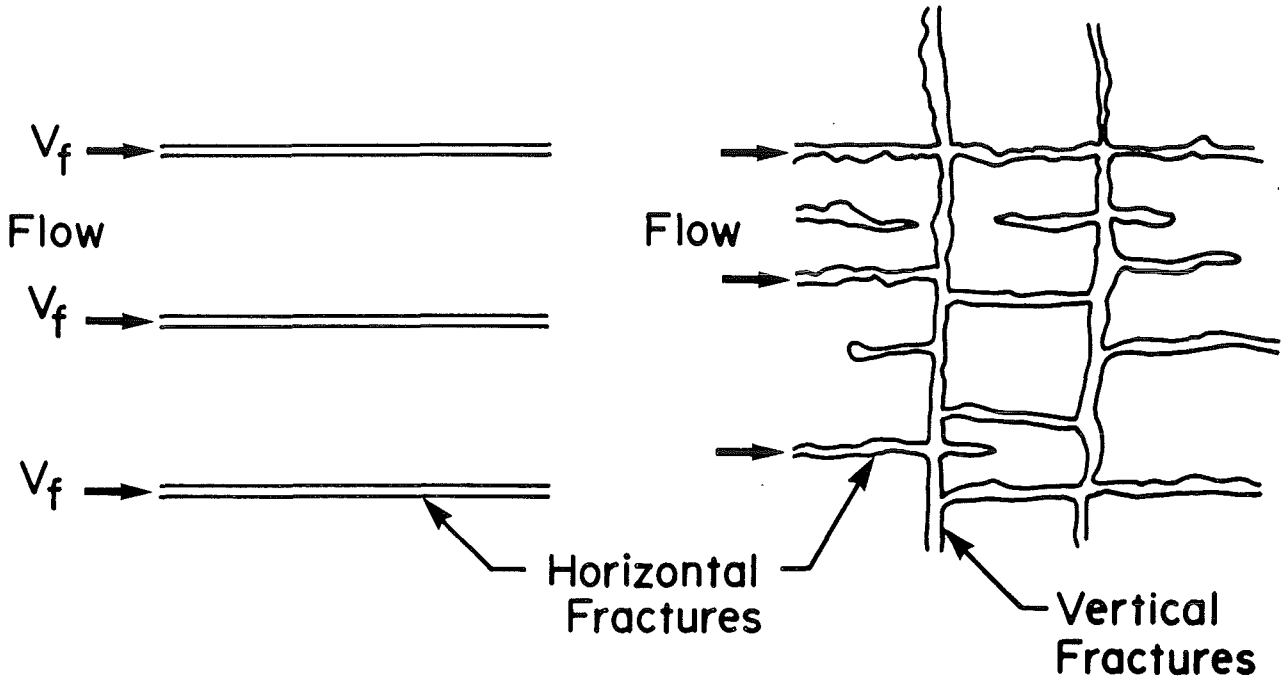
(i) For a two-dimensional system, contaminant will diffuse into the rock matrix from both the vertical and horizontal cracks. This will reduce the time it takes for contaminant to diffuse through the matrix relative to the time it would take if diffusion was only from the horizontal fractures.

(ii) The effect of two-dimensional diffusion will be similar to the effect of slightly reducing the spacing between horizontal fractures.

(iii) The difference between results for one- or two-dimensional diffusion will not be significant for times greater than that required to reach equilibrium. For spacings of 30 cm or less, the time to reach equilibrium is of the order of 25 years or less.

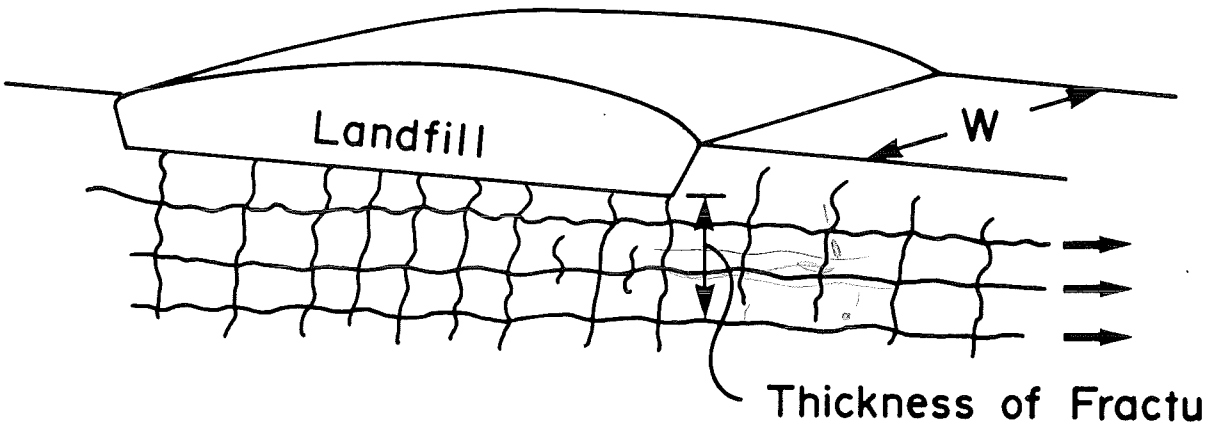
SCHEMATIC SHOWING IDEALIZED AND "MORE REALISTIC" FRACTURE PATTERNS

(Section Parallel to Flow Direction)



NOTES:
The idealized situation involves a series of parallel fractures at a constant spacing 2H, where each fracture has a constant width 2b. The real situation can be expected to be more complicated than this idealized case and will involve dead end fractures, constrictions (narrowing followed by widening of fracture) and a tortuous flow path (flow along near horizontal fractures then up or down a vertical fracture then along a near horizontal fracture). For situations where the spacing between the fractures is not uniform, one can obtain a reasonable estimate of the attenuation as contaminant migrates along the fractures by performing analyses for both the minimum and maximum reasonable spacing of the fractures. Note that the shortest distance travelled by contaminant corresponds to the parallel fracture idealization. In the real situation contaminant must travel further and would be exposed to a greater surface area through which matrix diffusion could occur.

SCHEMATIC SHOWING ASSUMED ZONE WHERE MIGRATION OCCURS



NOTES: For purposes of this study, it was assumed that

(i) contaminant could readily migrate down into the rock beneath the landfill; and
(ii) would migrate laterally through a fractured zone of thickness T = 25 m
(iii) dilution from groundwater arising to the north of the site was neglected
(iv) the concentration of contaminant in the rock beneath the landfill was reduced to a value c_o^* to take account of matrix diffusion into the rock beneath the landfill.
(v) the thickness of rock, T, used in the analysis does not significantly affect the calculated attenuation with distance from the landfill.

STEPS TAKEN IN THE STUDY

1. ESTIMATE LATERAL FLOW (DARCY VELOCITY) OF v_A = 0.7 M/A BASED ON AN ASSUMED RECHARGE OF 14 MM/A OBTAINED BY THCL IN THEIR INITIAL FLOW MODEL (EX. H40, P. 74).

2. CALCULATE CONTAMINANT PLUME AFTER 15 YEARS FOR BURLINGTON LANDFILL USING v_A = 0.7 M/A.

3. EXAMINE THE EFFECT OF UNCERTAINTY REGARDING FRACTURE OPENING SIZE AND FRACTURE SPACING.

4. ESTIMATE A LATERAL FLOW BASED ON INFORMATION PROVIDED BY GEOCON TO BE ABOUT 0.05 M/A. CALCULATE CONTAMINANT PLUME AT BURLINGTON AFTER 15 YEARS FOR v_A = 0.05 M/A.

5. EXAMINED THE POTENTIAL FOR ATTENUATION IN THE FRACTURED ZONE SOUTH OF SITE F FOR v_A = 0.05.

6. CONSIDERED "WORST CASE" SCENARIO OF A FLOW OF 0.125 M/A BOTH FOR BURLINGTON LANDFILL AND SITE F.

P26 440

SUMMARY OF PARAMETERS CONSIDERED

LANDFILL DIMENSIONS: HALTON (SITE F): L = 1250m , W = 400m

BURLINGTON: L = 800m , W = 250m

(PORTION WITHOUT UNDERDRAIN)

LATERAL FLOWS v_a = 0.7 m/a BASED ON THCL'S ORIGINAL MODEL

0.05 m/a BASED ON EXPECTED CONDITIONS

0.125 m/a BASED ON "WORST CASE" EXPECTED.

RECHARGE q_a = 14 mm/a FOR v_a = 0.7m/a AND T = 25m

1 mm/a FOR v_a = 0.05m/a AND T = 25m

2.5 mm/a FOR v_a = .125m/a AND T = 25m

FRACTURE SPACING, 2H = TYPICALLY 5-30 CM (1.2 M FOR ONE CASE)

APERTURE SIZE, 2b = 30-500 μm

FRACTURE POROSITY, n_b = 2b/2H = 0.0001-0.01

MATRIX POROSITY TESTS GAVE 0.1-0.11

VALUE USE 0.08

THCL VALUE = 0.1

DIFFUSION COEFFICIENT: TESTS GAVE 1.5×10^{-6} cm^2/s ($1.4-1.6 \times 10^{-6}$ cm^2/s)

CHLORIDE: IN SHALE: D_m VALUE USED 6×10^{-7} cm^2/s (0.0019 m^2/a)

THCL VALUE 5×10^{-7} cm^2/s

DIFFUSION COEFFICIENT

CHLORIDE: IN SOLUTION: D_o 12.7×10^{-6} cm^2/s

SUMMARY OF PARAMETERS (CONT'D)

TORTUOSITY	$\tau = D_m/D_o$	0.047
DIFFUSION COEFFICIENT BENZENE: IN SOLUTION	D_{oB}	7.5×10^{-6} cm^2/s
DIFFUSION COEFFICIENT BENZENE: IN SHALE (estimated) $=\tau . D_{oB} =$		3.6×10^{-7} cm^2/s (0.0011 m^2/a)
FRACTION OF ORGANIC CARBON	f_{oc}	0.18% (THCL, P. 100)

BENZENE

K_p (Based on Chiou & Schmedding, 1982)	0.19 cm^3/g
RETARDATION FACTOR R_m (BENZENE)	5.5-7
VALUE ADOPTED	5.5

INITIAL CONCENTRATIONS,

CHLORIDE	1500 mg/L	(THCL, P. 92)
BENZENE	150 µg/L	

MASS OF:

CHLORIDE	M_{TC}	
HALTON		7800 tonnes
BURLINGTON		2200 tonnes
BENZENE	M_{TC}	
HALTON		0.78 tonnes
BURLINGTON		0.22 tonnes

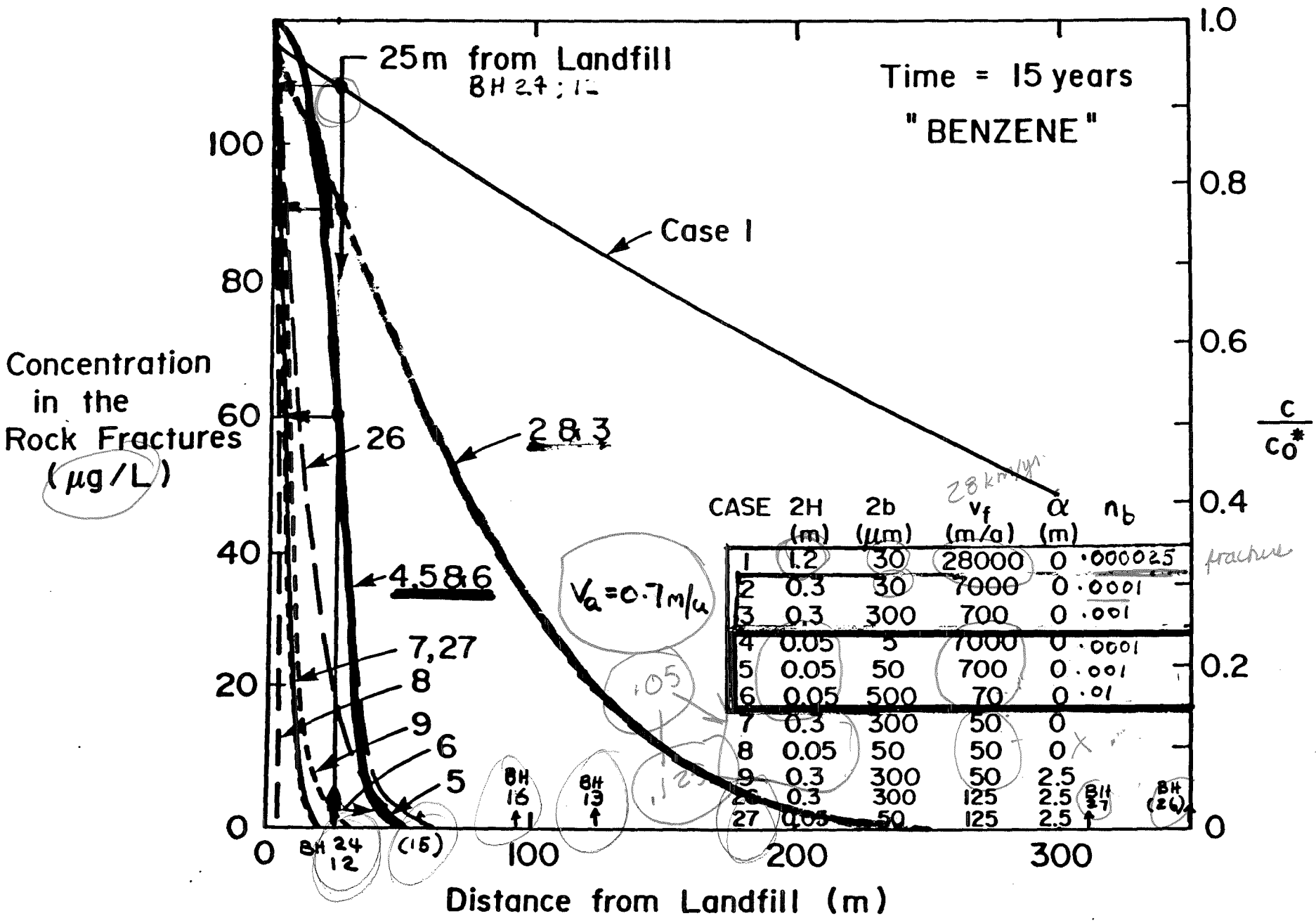


FIGURE 4 CALCULATED MIGRATION OF BENZENE FROM THE BURLINGTON LANDFILL AFTER 15 YEARS

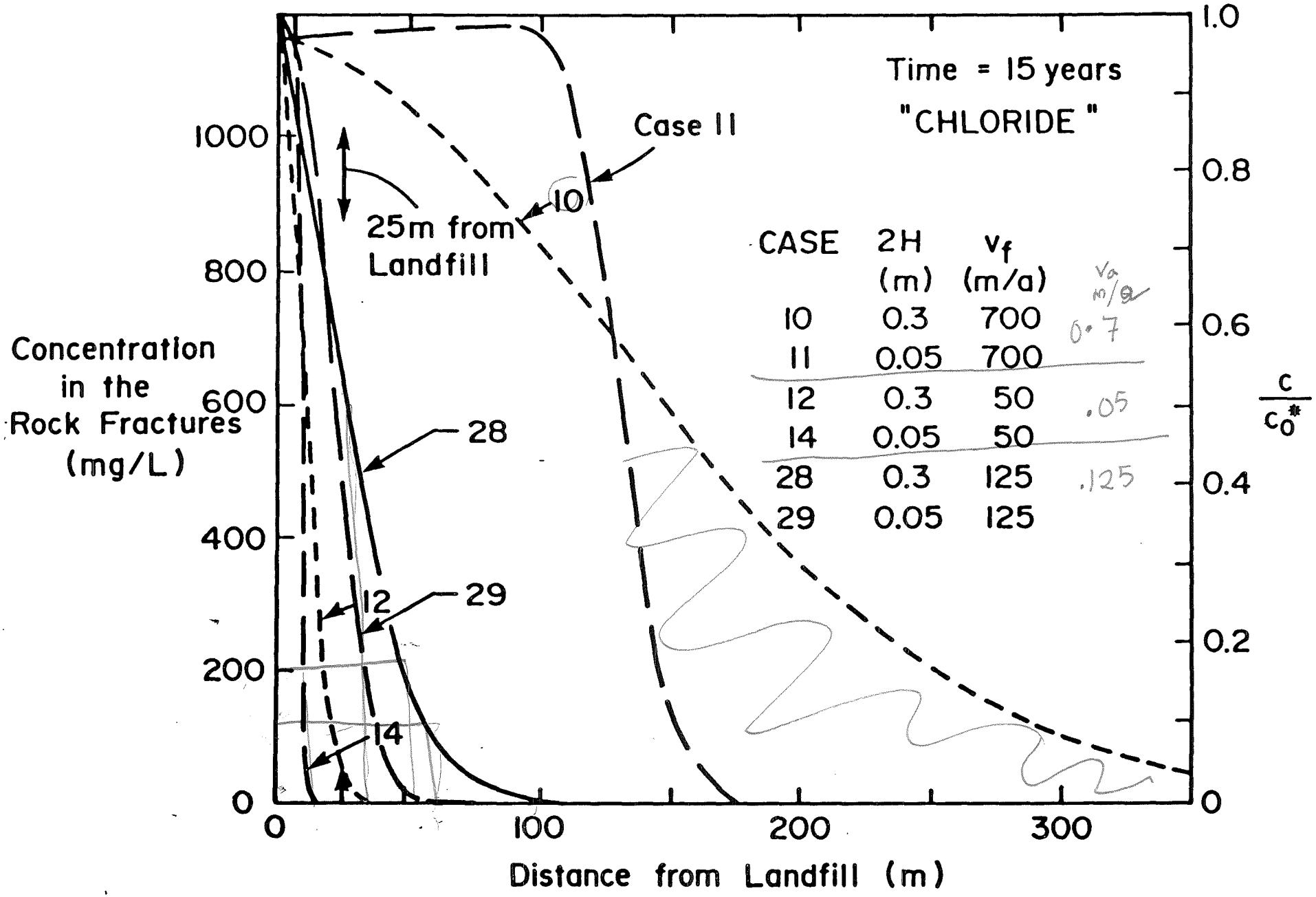


FIGURE 5 CALCULATED MIGRATION OF "CHLORIDE" FROM THE BURLINGTON LANDFILL AFTER 15 YEARS

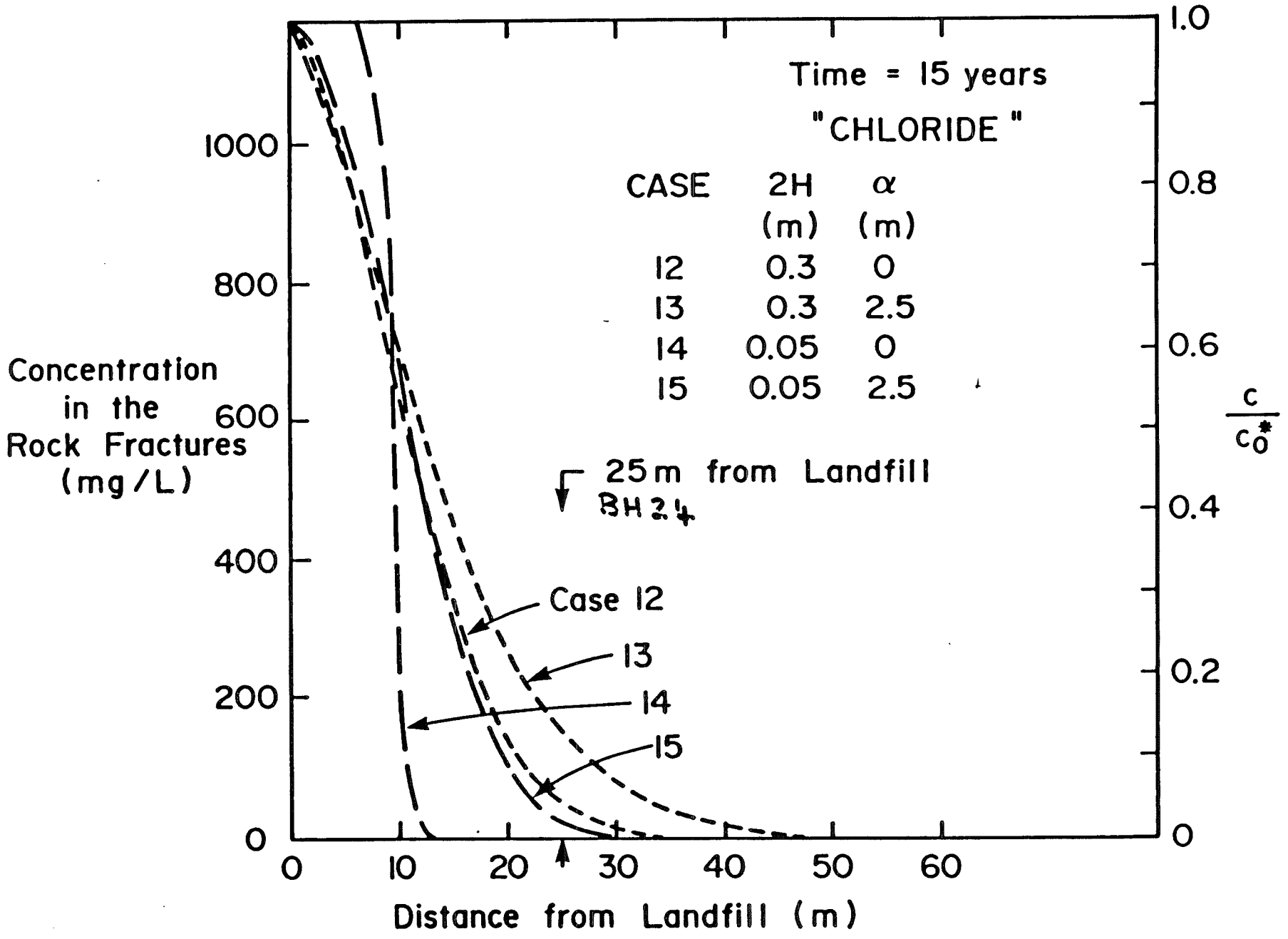


FIGURE 6 CALCULATED MIGRATION OF "CHLORIDE" FROM THE BURLINGTON LANDFILL AFTER 15 YEARS

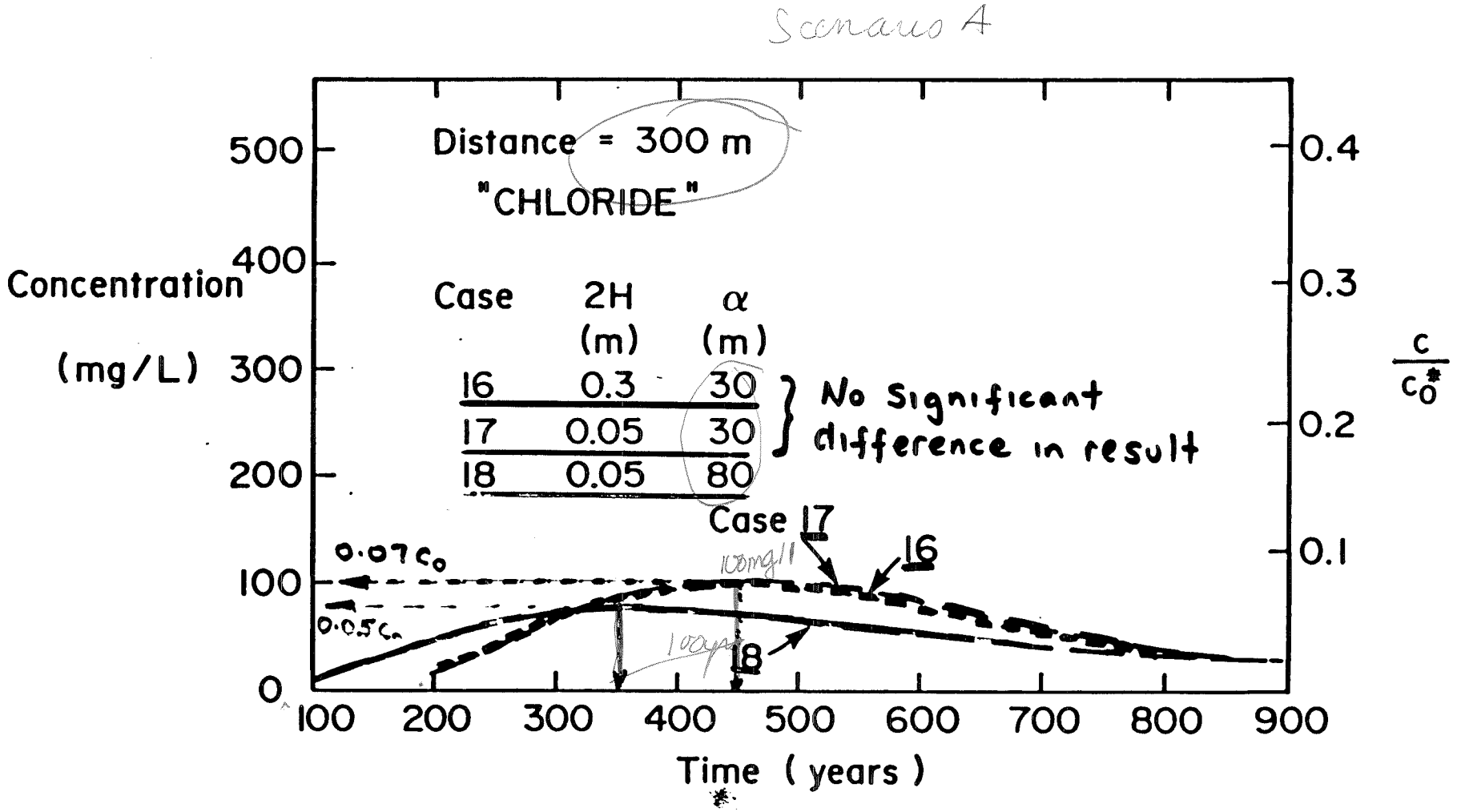


FIGURE 7 CALCULATED VARIATION IN "CHLORIDE" CONCENTRATION WITH TIME AT A POINT 300 m DOWNGRADIENT OF THE PROPOSED SITE F HALTON LANDFILL

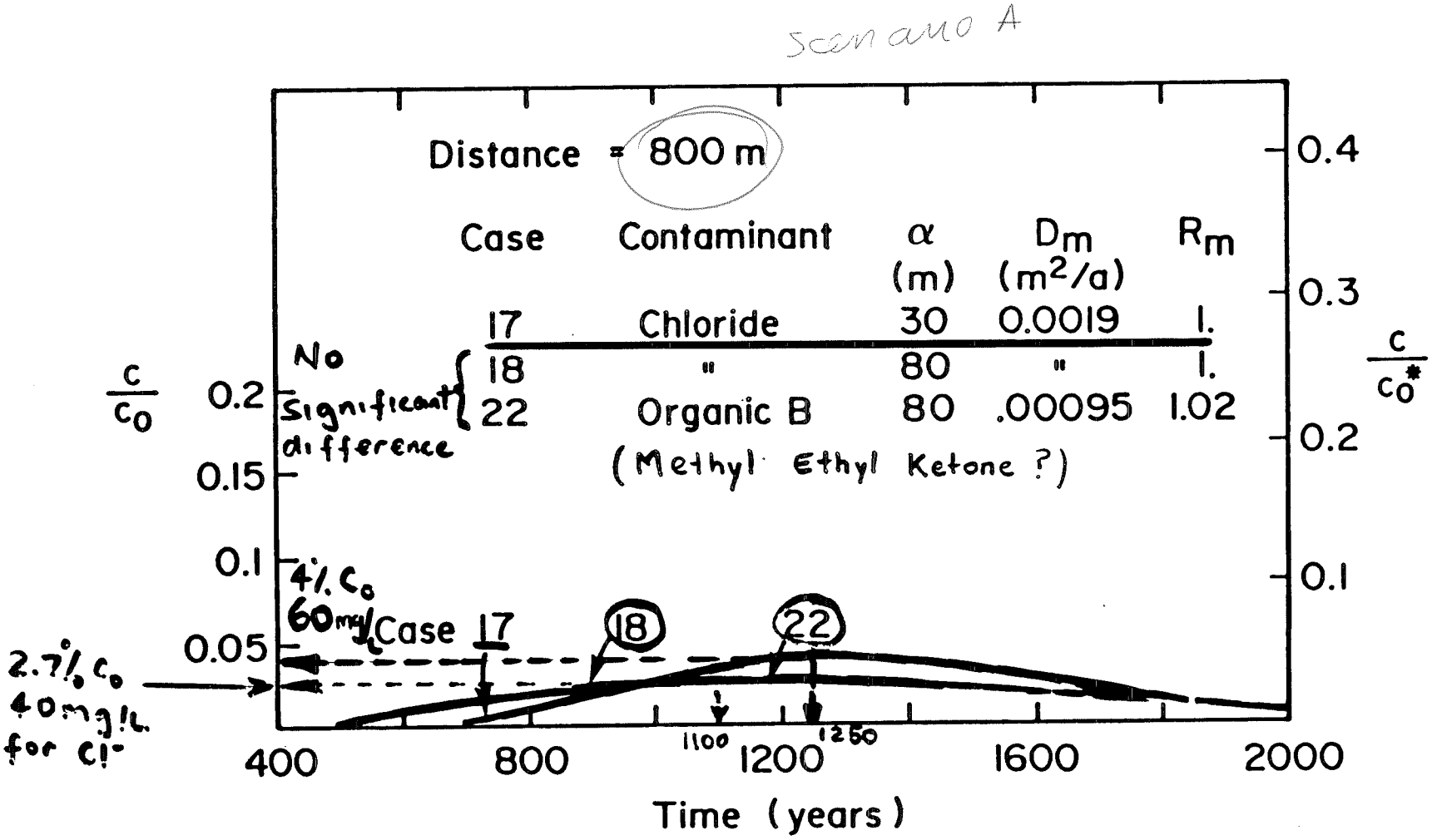


FIGURE 8 CALCULATED VARIATION IN CONTAMINANT CONCENTRATION WITH TIME AT A POINT 800 m DOWNGRADIENT OF THE PROPOSED HALTON LANDFILL (SITE F)

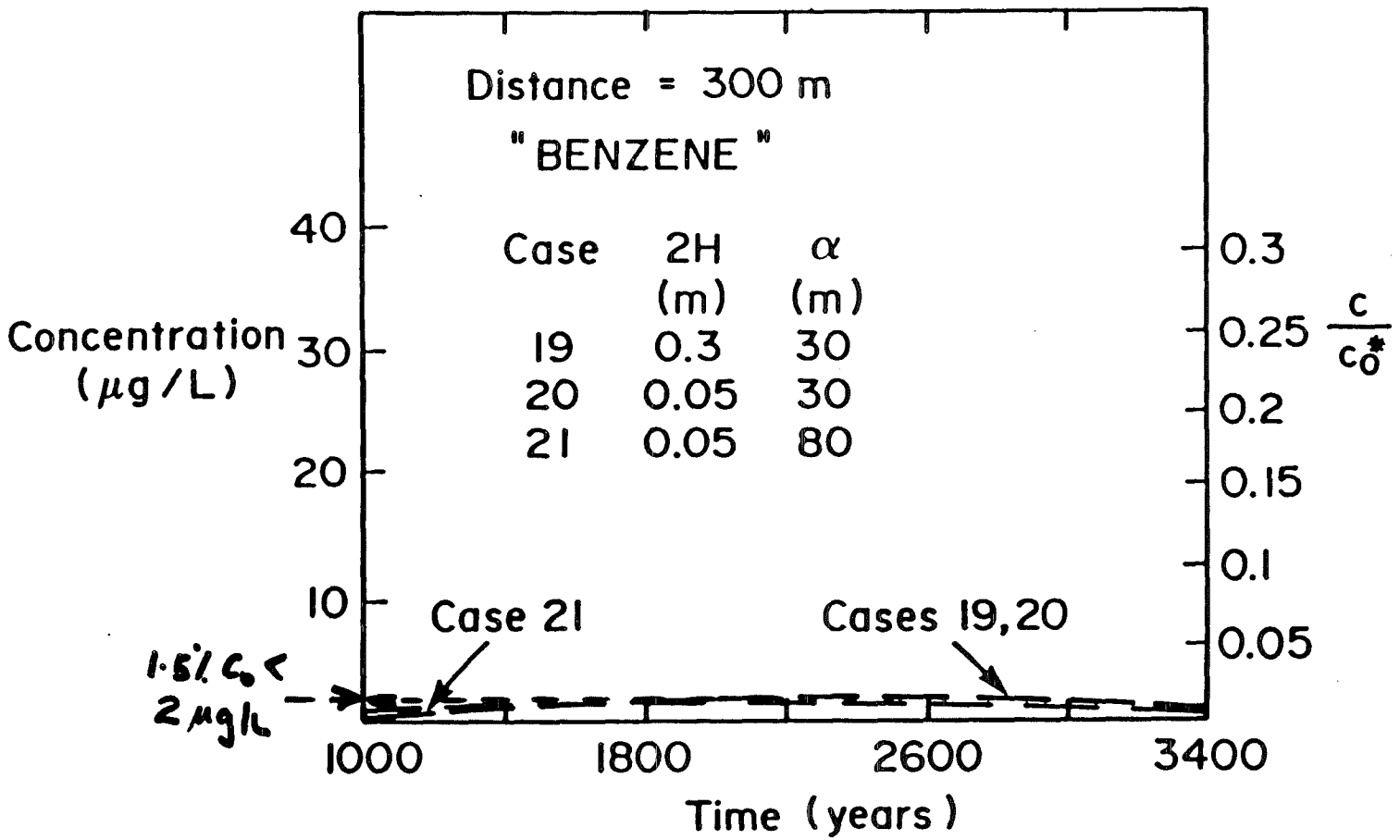


FIGURE 9 CALCULATED VARIATION IN "BENZENE" CONCENTRATION WITH TIME AT A POINT 300 m DOWNGRADIENT OF THE PROPOSED HALTON LANDFILL (SITE F)

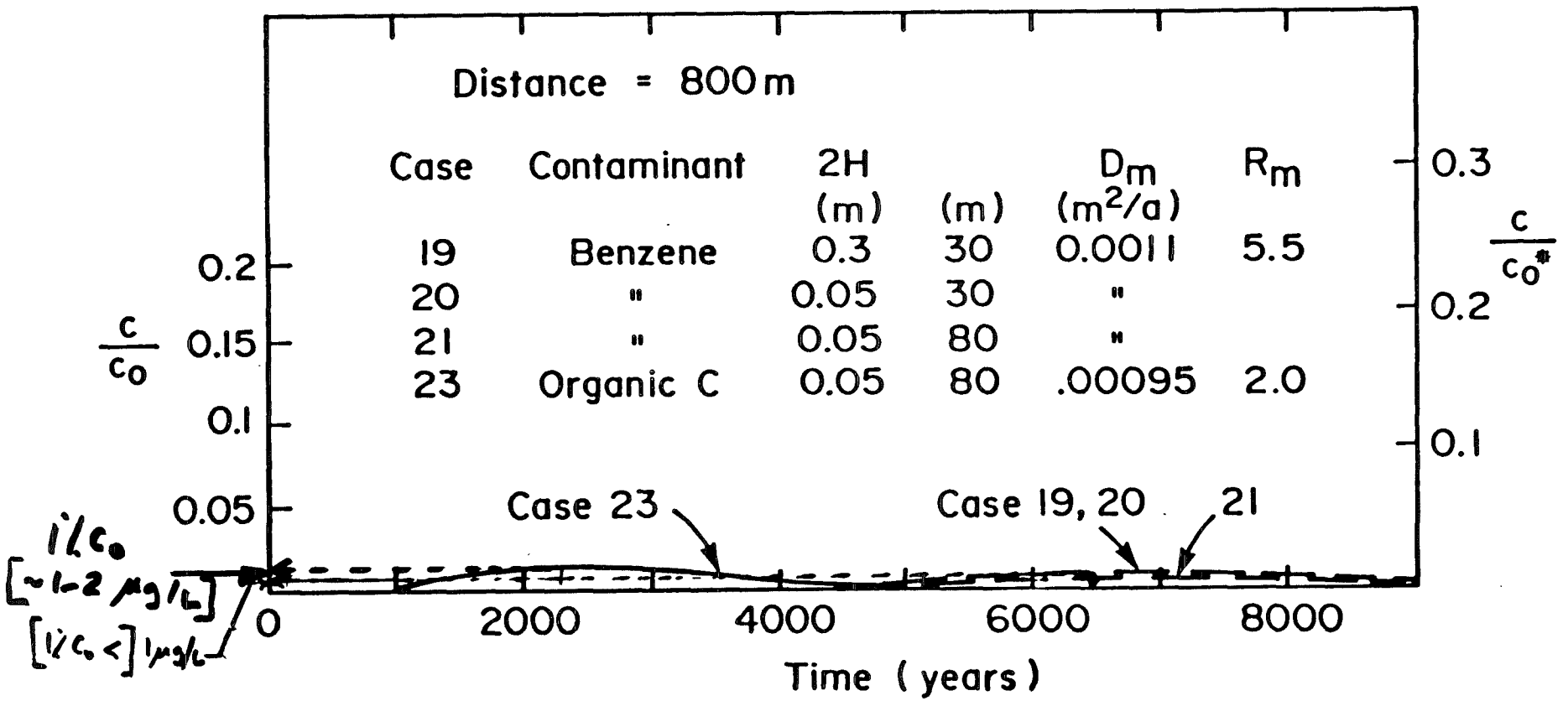


FIGURE 10 CALCULATED VARIATION IN ORGANIC CONTAMINANT CONCENTRATION WITH TIME AT A POINT 800 m DOWNGRADIENT OF THE PROPOSED HALTON LANDFILL (SITE F)

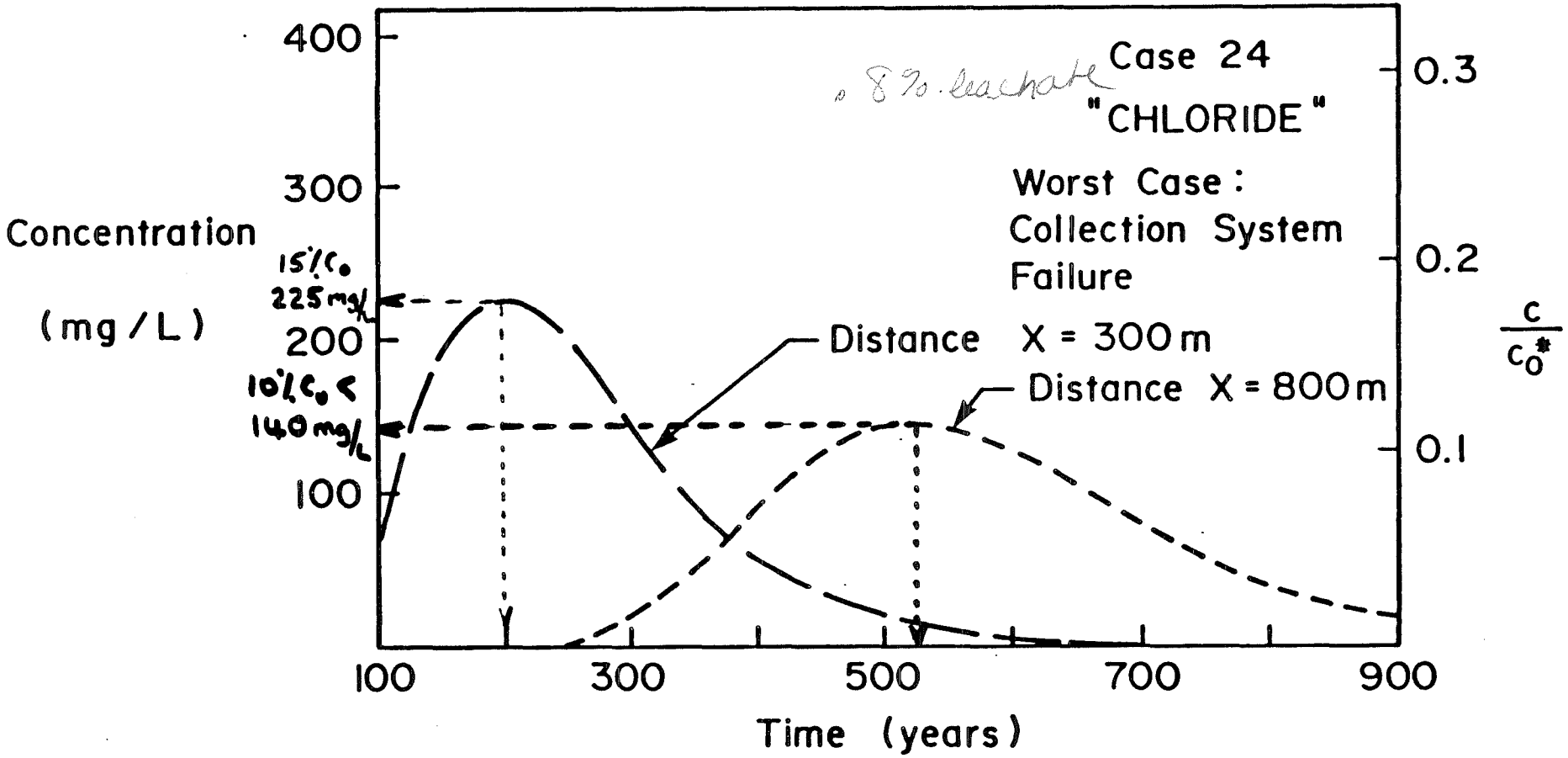


FIGURE 11 CALCULATED VARIATION IN "CHLORIDE" CONCENTRATION WITH TIME DOWNGRADIENT OF THE PROPOSED HALTON LANDFILL (SITE F) - "WORST" CASE SCENARIO

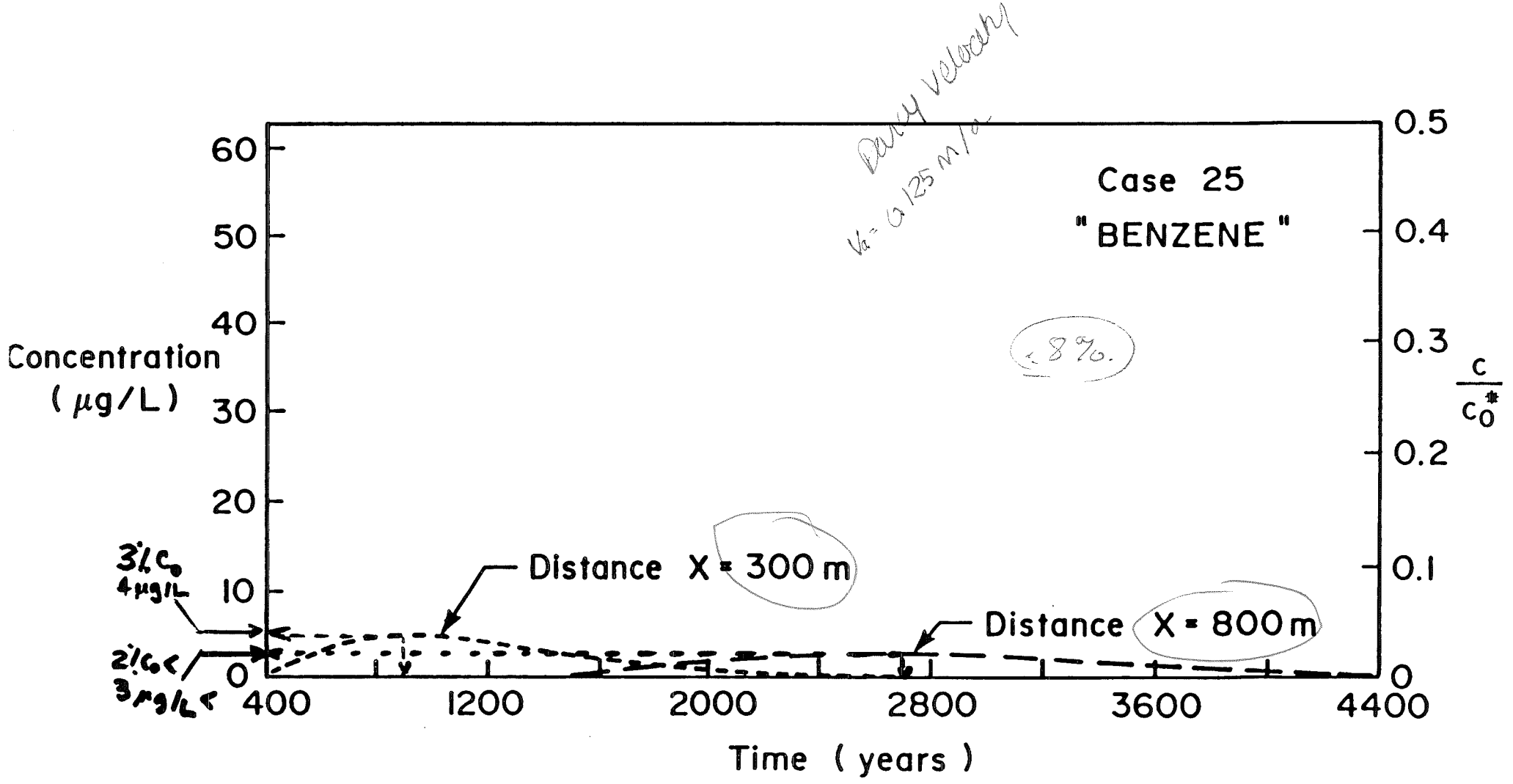


FIGURE 12 CALCULATED VARIATION IN "BENZENE" CONCENTRATION WITH TIME DOWNGRADIENT OF THE PROPOSED HALTON LANDFILL (SITE F) - "WORST" CASE SCENARIO

CONCLUSIONS: SITE F

Scenario A.

1. NO "WEAKLY FRACTURED SHALE UNIT" (CASE CONSIDERED IN REPORT):
   - OBSERVATIONS FOR BURLINGTON LANDFILL ARE CONSISTENT WITH A DARCY VELOCITY LESS THAN 0.125 m/a AND PROBABLY ABOUT 0.05 m/a
   - IMPACT OF PROPOSED LANDFILL ON GROUNDWATER QUALITY (SOUTH OF HIGHWAY 403) WOULD BE MINIMAL

     306m

     . CONSERVATIVE SPECIES: INCREASE IN CONCENTRATION LESS THAN
       15% FOR "WORST" CASE AFTER 200 YEARS
       7% FOR "EXPECTED" CASE AFTER 450 YEARS
     . ORGANICS SUCH AS BENZENE: INCREASE IN CONCENTRATION LESS THAN
       3% (4.5 µg/L) FOR "WORST" CASE AFTER 900 YEARS
       2% (3 µg/L) FOR "EXPECTED" CASE AFTER 2200 YEARS
   - IMPACT ON GROUNDWATER QUALITY SOUTH OF CN TRACKS WOULD BE EVEN SMALLER
     . CONSERVATIVE SPECIES: INCREASE LESS THAN
       10% FOR "WORST" CASE AFTER 525 YEARS
       4% FOR "EXPECTED" CASE AFTER 1250 YEARS
     . ORGANICS SUCH AS BENZENE: INCREASE LESS THAN
       2% (3 µg/L) FOR "WORST" CASE AFTER 2700 YEARS
       1.5% (2 µg/L) FOR "EXPECTED" CASE AFTER 6500 YEARS
   - CALCULATION NEGLECTS OTHER ATTENUATION MECHANISMS SUCH AS
     . DILUTION DUE TO RECHARGE
     . BIODEGRADATION AND MICROBIAL TRANSFORMATION OF ORGANICS
     . ATTENUATION IF IT DISCHARGES THROUGH THE TILL SOUTH OF HIGHWAY 403

CONCLUSIONS: SITE F

2. ACCEPTING THCL'S "BETTER FIT FLOWS" AND CONSIDERING MIGRATION DOWNWARD THROUGH THE "WEAKLY FRACTURED SHALE UNIT" AND THEN ALONG A 5 m THICK HIGHLY FRACTURED ZONE:

   - AFTER ONLY 15 YEARS MIGRATION, ONE WOULD NOT EXPECT ANY CONTAMINANTS TO HAVE REACHED 25 m SOUTH OF BURLINGTON LANDFILL BY THIS MECHANISM
   - MAXIMUM "CHLORIDE" CONCENTRATION BENEATH DOWNGRADIENT EDGE OF LANDFILL IS LESS THAN 5% OF THE INITIAL CONCENTRATION (I.E. 85 mg/L)
   - IMPACT IS ALREADY LESS THAN THAT DEDUCED FOR CASE 1 ABOVE EVEN WITHOUT CONSIDERING MATRIX DIFFUSION
   - ATTENUATION WOULD OCCUR WITH DISTANCE DOWNGRADIENT
   - EXPECT PEAK CHLORIDE CONCENTRATION DOWNGRADIENT (E.G. NEAR CNR TRACKS) TO BE LESS THAN 4% c_o (60 mg/L) AND PEAK WOULD NOT BE REACHED UNTIL AFTER AT LEAST 775 YEARS

STEPS FOLLOWED IN THE STUDY OF SITE D

1. SELECT A THEORETICAL MODEL.

2. SELECT THE CONTAMINANT SPECIES TO BE CONSIDERED (CHLORIDE).

3. ESTIMATE THE MASS OF CHLORIDE AVAILABLE FOR TRANSPORT INTO THE SOIL.

4. CONSIDER LEACHATE MOUNDING AND POSSIBLE MODES OF FAILURE OF THE "HYDRAULIC TRAP".

5. ADOPT AN IDEALIZATION OF THE THICKNESS OF THE HYDROSTRATIGRAPHIC UNITS AND SELECT HYDRAULIC CONDUCTIVITIES FOR EACH UNIT.

6. SELECT BOUNDARY HEADS.

7. CALCULATE THE FLOWS IN THE VARIOUS HYDROSTRATIGRAPHIC UNITS.

8. SELECT THE DIFFUSION COEFFICIENT AND DISPERSIVITIES.

9. PERFORM ADVECTIVE-DISPERSIVE TRANSPORT STUDIES.

FEATURES OF MODEL USED IN U.W.O. STUDY

* Considers Two-Dimensional (2D) contaminant migration

* Semi-analytic in Nature

  The technique used to obtain a solution involved

  . Taking the Laplace and Fourier Transform of the governing equations and boundary conditions

  . Finding an exact (closed form) solution for the concentrations and fluxes within each layer

  . Enforcing continuity of concentration and flux at layer boundaries

  . Numerically invert the Laplace and Fourier transforms to get the required concentrations

* Allows one to easily and accurately calculate concentration at any specific points in a layered system at any specific times without determining the solution at other points in space and time

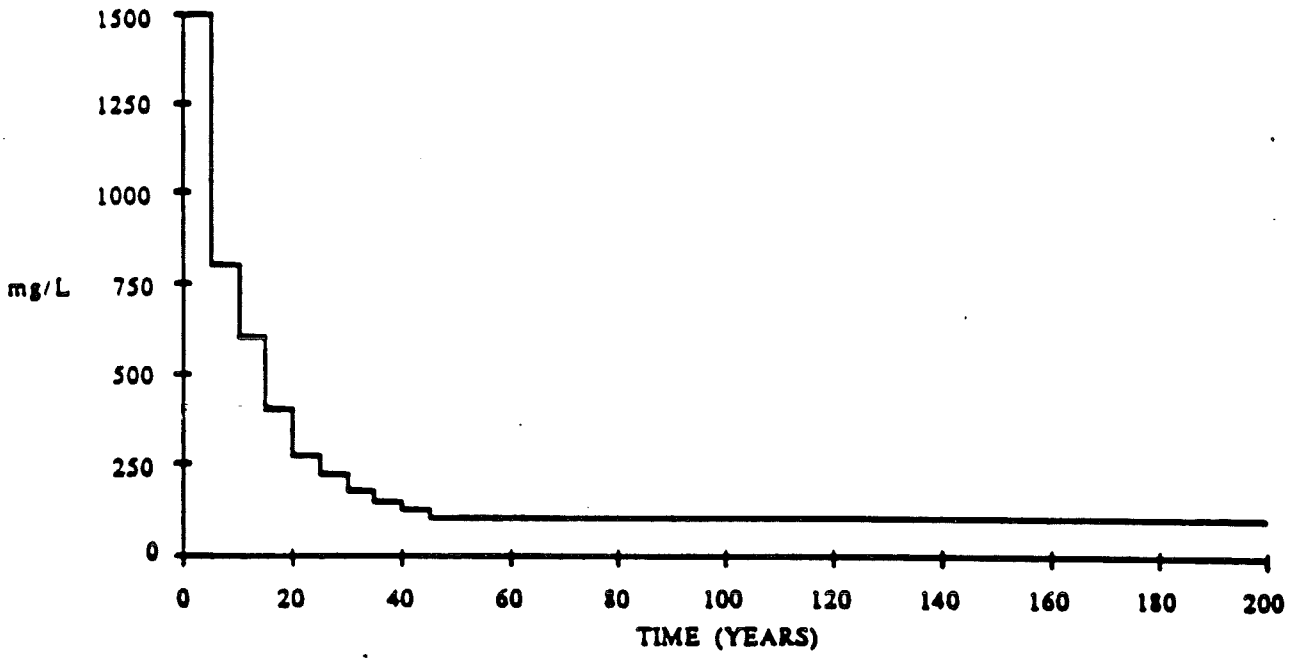
* Automatically satisfies conservation of mass

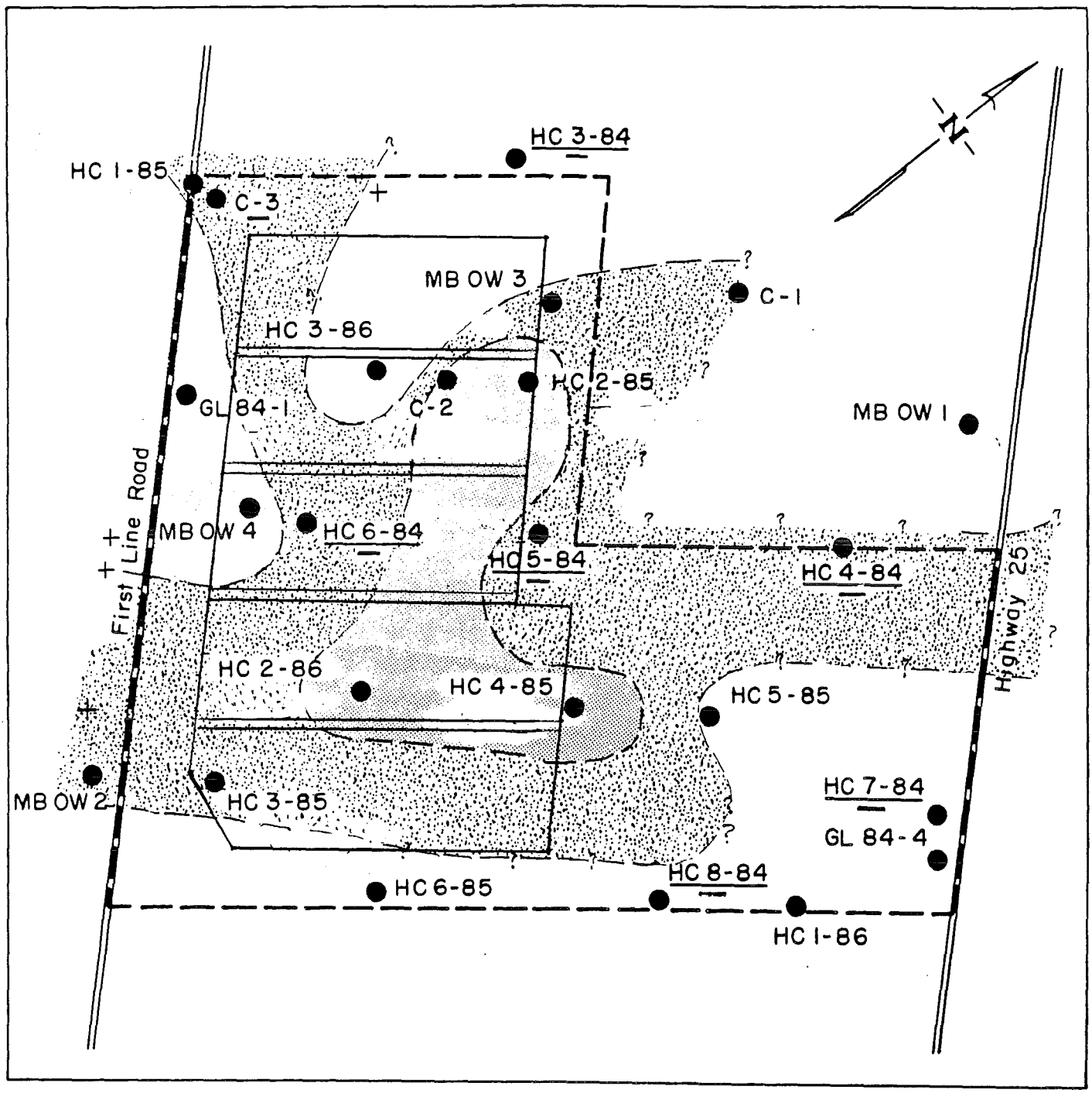
* Can automatically model the decrease in leachate concentration with time as contaminant moves into the soil

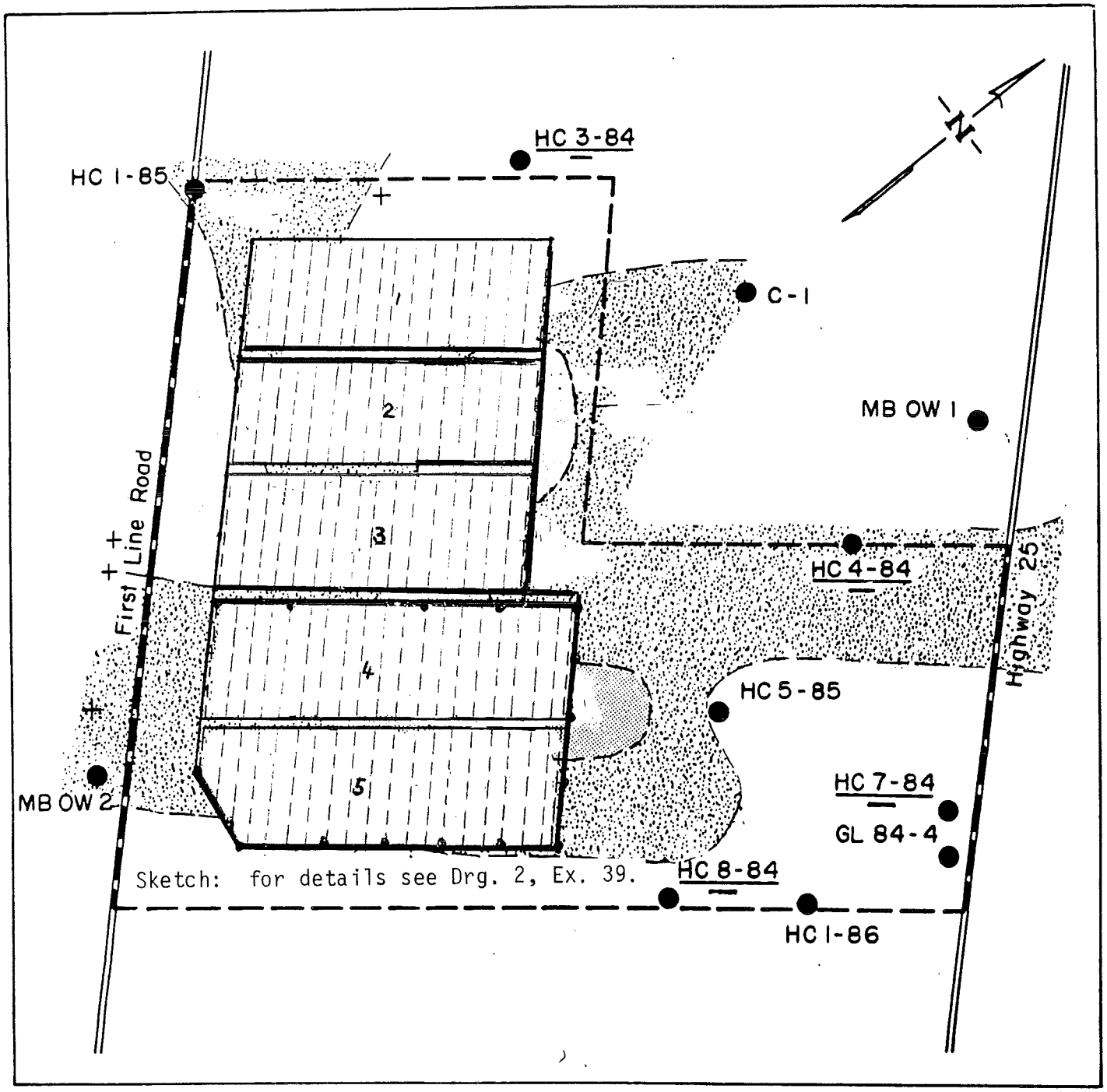
* Automatically considers "matrix diffusion"

FIGURE 3 (FROM THCL, Site D Report, Vol. 1, Figure 15, p. 110)

PROJECTED DECLINE IN CHLORIDE CONCENTRATIONS IN LANDFILL LEACHATE




HC 3-84
HC 1-85
C-3
MB OW 3
C-1
HC 3-86
HC 2-85
GL 84-1
C-2
MB OW 1
First Line Road
MB OW 4
HC 6-84
HC 5-84
HC 4-84
Highway 25
HC 2-86
HC 4-85
HC 5-85
MB OW 2
HC 3-85
HC 7-84
GL 84-4
HC 8-84
HC 6-85
HC 1-86
N


HC 3-84
HC 1-85
C-1
MB OW 1
2
3
4
5
First Line Road
HC 4-84
Highway 25
HC 5-85
HC 7-84
GL 84-4
MB OW 2
Sketch: for details see Drg. 2, Ex. 39.
HC 8-84
HC 1-86
N

CLOGGING OF DRAINS

- IMPORTANT IN A WIDE RANGE OF GEOTECHNICAL APPLICATIONS
- DRAINS AND FILTERS FOR A LANDFILL ARE ONE OF THE MOST DIFFICULT DESIGN ENVIRONMENTS
- VERY LIMITED RESEARCH HAS BEEN CONDUCTED INTO LONG TERM PERFORMANCE OF LANDFILL DRAINAGE SYSTEMS
- NEED TO CONSIDER THE POTENTIAL FOR FAILURE AND DEVELOP A DESIGN, CONSTRUCTION AND MAINTENANCE PROGRAM WHICH WILL MINIMIZE THE POTENTIAL FOR FAILURE AND WHICH WILL MINIMIZE THE IMPACT OF A FAILURE
- CLOGGING MAY OCCUR (I) IN THE DRAINAGE PIPE ITSELF AND/OR
  (II) IN THE SOIL AROUND THE PIPE
- CLOGGING OF THE FILTER AROUND THE PIPE MAY OCCUR DUE TO
  (I) THE MIGRATION OF FINES FROM THE WASTE
  (II) PRECIPITATION (BOTH CHEMICAL AND BIOCHEMICAL)
  (III) BIOLOGICAL GROWTH

  IT IS VERY DIFFICULT TO "CLEAN" THE FILTER SOIL AROUND THE PIPE
- CLOGGING OF THE PIPES MAY OCCUR DUE TO:
  (I) STRUCTURAL FAILURE
  (II) SEDIMENTATION
  (III) PRECIPITATION (BOTH CHEMICAL AND BIOCHEMICAL)
  (IV) BIOLOGICAL GROWTH

. DRAINS HAVE BEEN OBSERVED TO CLOG (DUE TO IRON CLOGGING) WITHIN ONE MONTH OF BEING LAID (EG PUIG ET AL., 1986)

. THE HYDRAULIC CONDUCTIVITY OF FILTERS MAY BE SUBSTANTIALLY REDUCED DUE TO MIGRATION OF FINES FROM THE WASTE. (GEOTEXTILE FILTERS HAVE HAD REDUCTIONS IN HYDRAULIC CONDUCTIVITY OF FIVE ORDERS OF MAGNITUDE WHEN TESTED WITH LEACHATE PASSING THROUGH - CONCELLI & CAZZUFFI, 1987)

. GOOD ENGINEERING PRACTICE WOULD BE TO

(I) CAREFULLY DESIGN THE FILTER USING TEST DATA OBTAINED USING LEACHATE WHICH CONTAINS FINES FROM THE PROPOSED DAILY COVER MATERIAL AS WELL AS PARTICULATE MATTER FROM LEACHATE ITSELF

(II) PREFERABLY USING A WATER DISPOSAL SYSTEM WHICH IS GRAVITY CONTROLLED

(III) PIPES SHOULD BE LAID IN STRAIGHT LINES WITH OPENINGS EVERY 30 TO 50 M THROUGH WHICH A FIRE HOSE CAN BE INSERTED TO FLUSH THE PIPE

(IV) MANHOLES SHOULD BE PROVIDED AT CHANGES IN DIRECTION AND AT INTERVALS OF 100 TO 150 M ALONG STRAIGHT SECTIONS

(V) HAVE AS HIGH A GRADIENT AS POSSIBLE WITHIN THE DRAIN (2% OR GREATER IS PREFERRED)

(VI) DESIGN THE SYSTEM SUCH THAT IF SOME OR ALL OF THE DRAINS FAIL, THE IMPACT WILL BE AS SMALL AS POSSIBLE

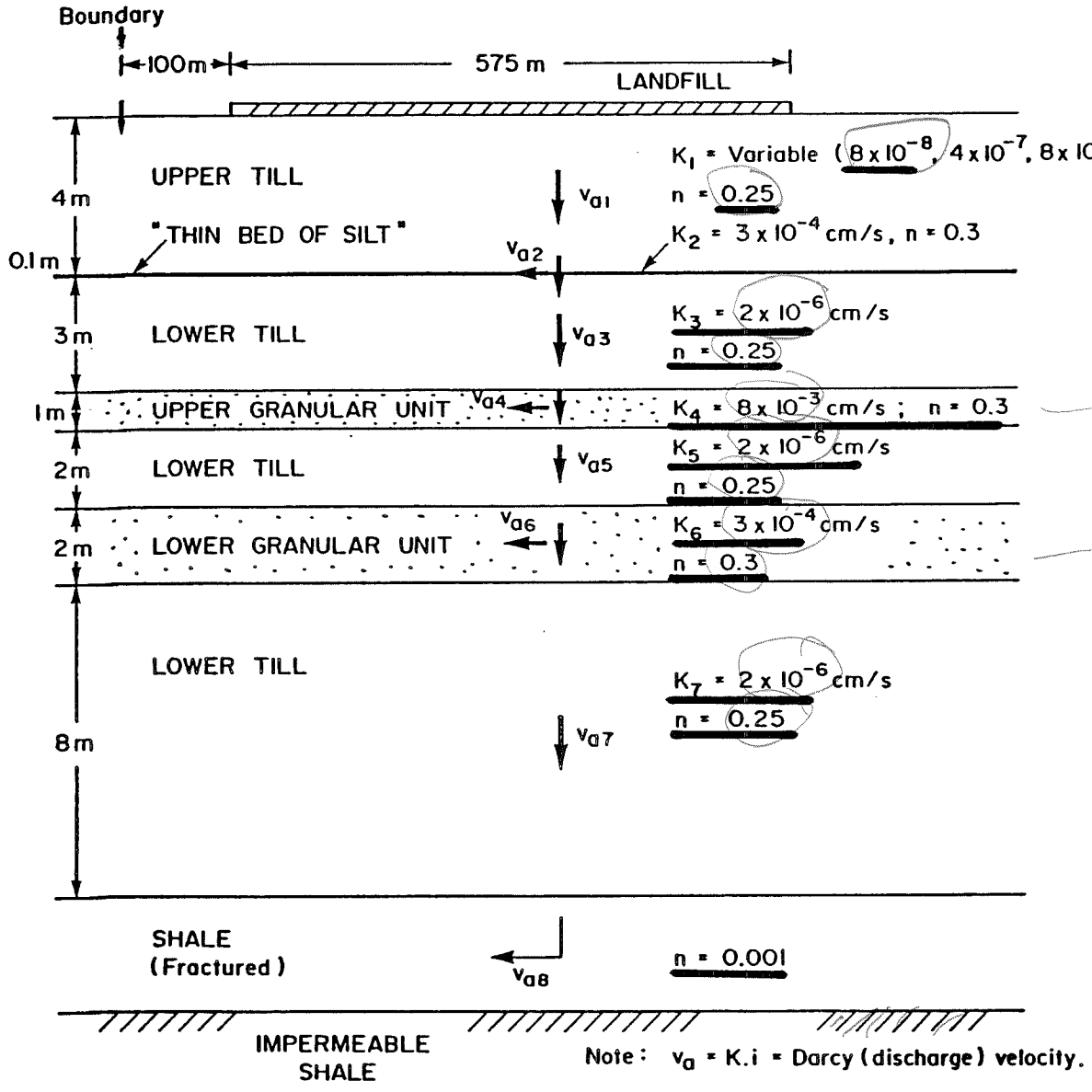


FIGURE 2 ASSUMED HYDROSTRATIGRAPHY AND FLOW PATHS

Values given by THCL [Ex 39]

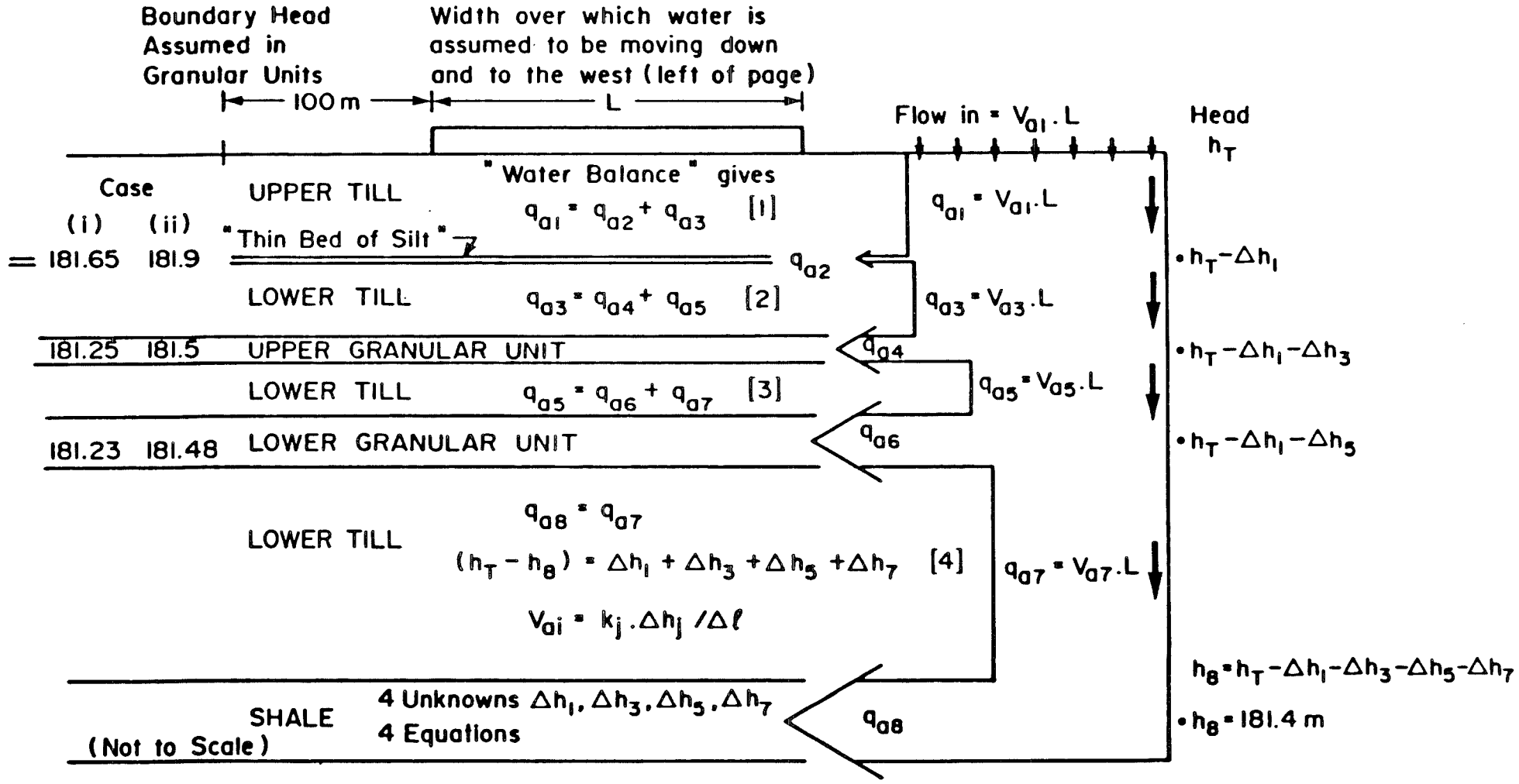
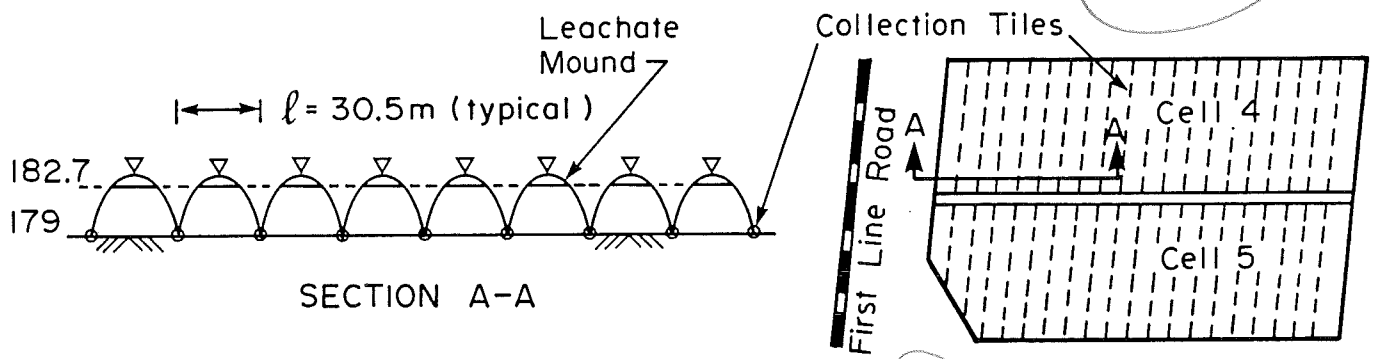


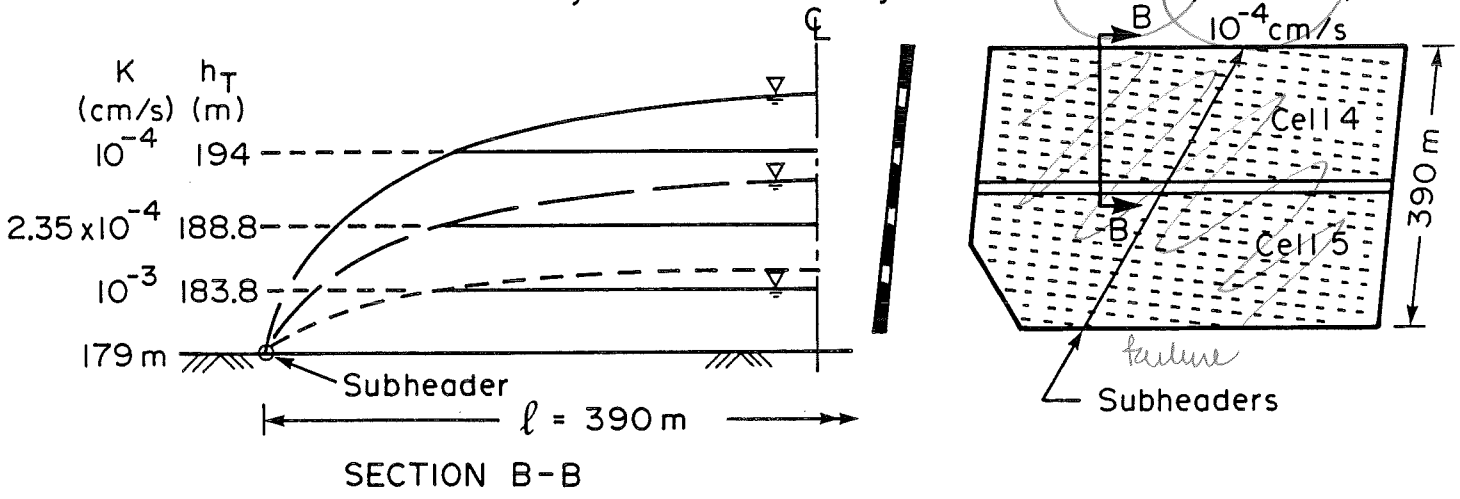
FIGURE 4 SCHEMATIC INDICATING CONSIDERATION OF "WATER BALANCE"
NOTE: THE THICKNESS OF HORIZONTAL ARROWS SHOWN HERE IS NOT INTENDED TO INDICATE THE RELATIVE PROPORTION OF THE FLOW IN THE VARIOUS UNITS.

FAILURE SCENARIOS CONSIDERED

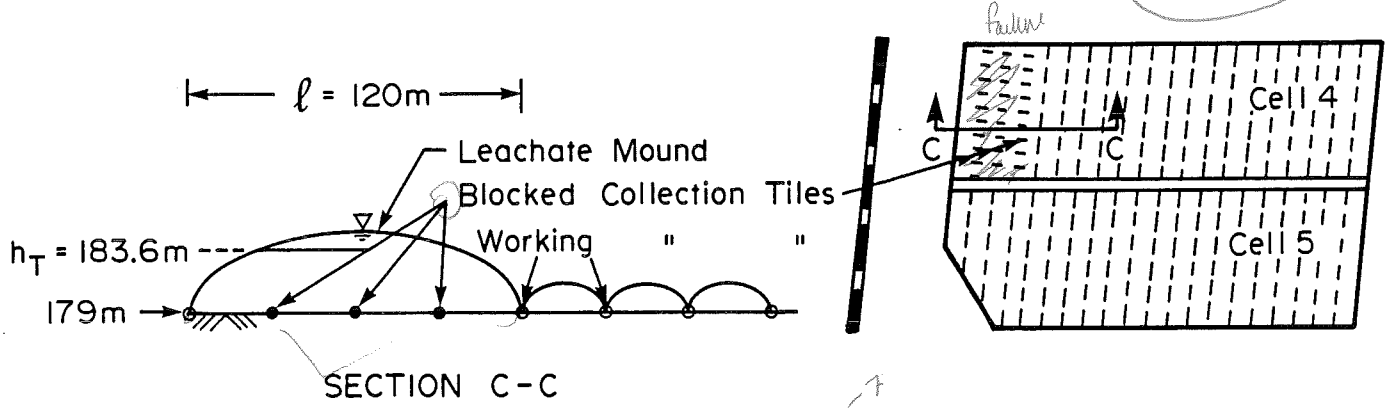
SCENARIO (a)
- All Collection Tiles Working
- Hydraulic Conductivity of Waste $K = 10^{-5}$ cm/s



SCENARIO (b)
- All Collection Tiles Failed
- Collection Subheaders Working
- Hydraulic Conductivity of Waste $K = 10^{-3}; 2.3 \times 10^{-4}; 10^{-4}$ cm/s

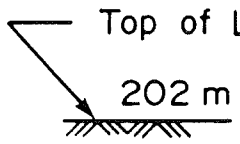


SCENARIO (c)
- Three Collection Tiles Fail – Remainder Working
- Hydraulic Conductivity of Waste $K = 10^{-4}$ cm/s

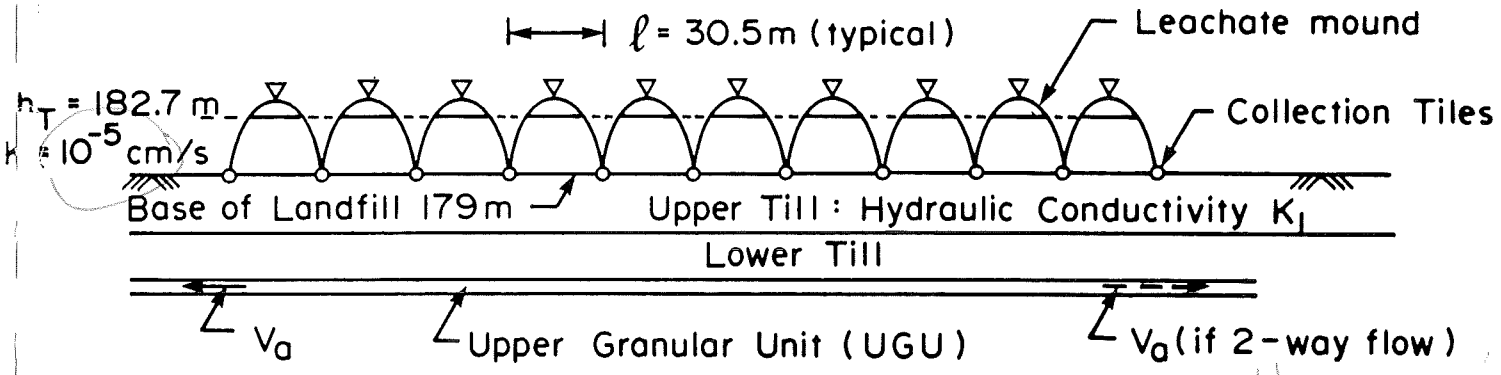


8×10^{-7}
4×10^{-7}

COLLECTION SCENARIO (A)



Scale (Approx.)
Horiz. 1:2500
Vert. 1:500



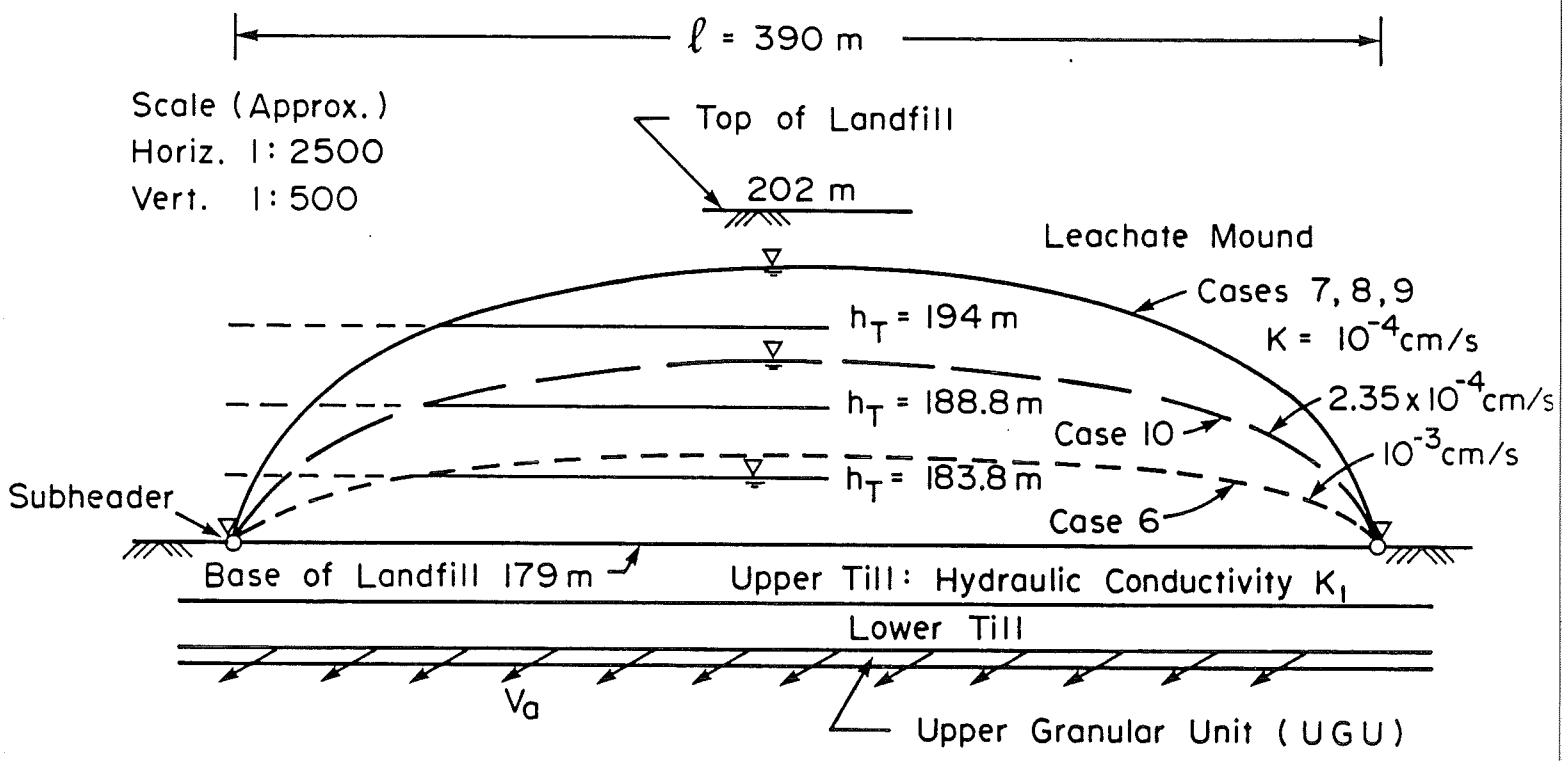
25mg/l.

Case	Flow	K_1 cm/s	Darcy "Velocity" in UGU: V_{a4} (m/a)	Leachate Collected %	Flow to Shale %	Time to Reach Boundary (years)	Peak Chloride Concentration (mg/L)	
2	1-way	4×10^{-7}	9.1	90	44	46	>530	80
3	"	8×10^{-7}	12.8	85	48	32	730	56
4a	"	8×10^{-7}	8.3	86	64	36	570	82
4b	"	"	"	"	"	"	490	6
4c	"	"	"	"	"	"	760	66
5	2-way	8×10^{-7}	6.4	85	48	35	590	58

NOTES: Except for cases 4b and 4c, all other analyses were performed for a mass of chloride of 4400 tonnes. Case 4b was performed for a mass of 3360 tonnes of chloride. Case 4c was performed for a mass of 7700 tonnes of chloride.

Summary

COLLECTION SCENARIO (B)



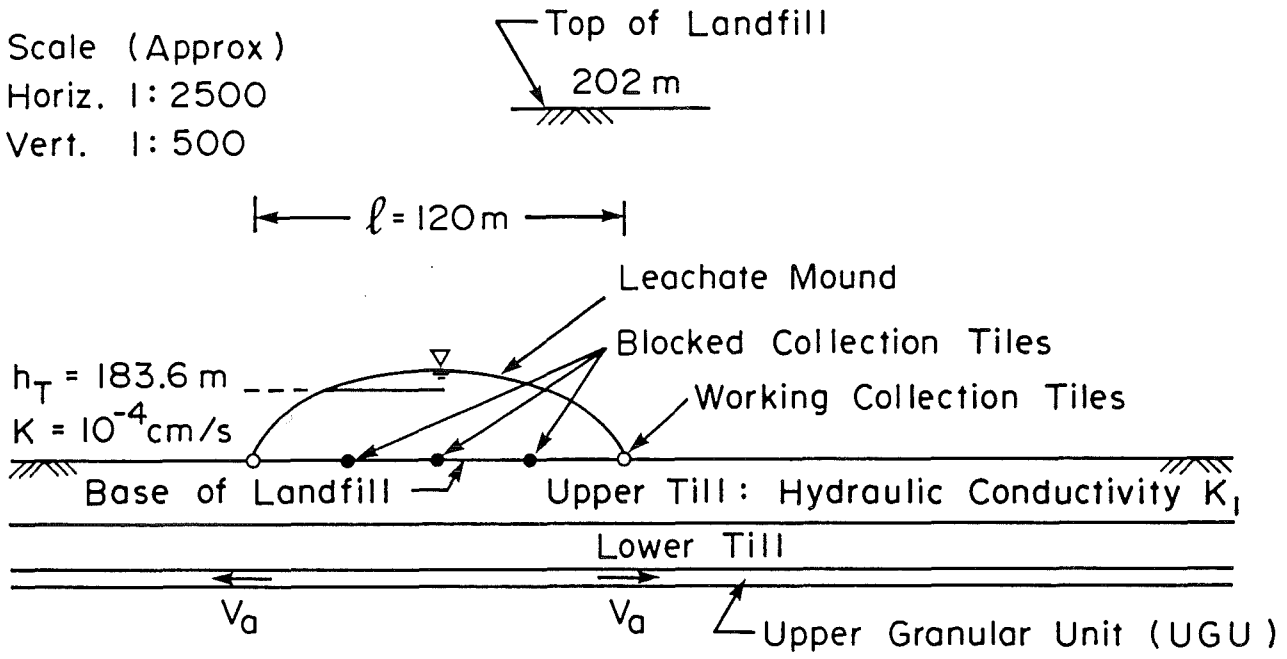
Case	Flow	K_1 cm/s	Darcy "Velocity" in UGU: V_{a4} (m/a)	Leachate Collected %	Flow to Shale %	Time to Reach Boundary (years)	Peak Chloride Concentration mg/L
6	1-way	8×10^{-7}	16.8	74	61	21	870
10	2-way	4×10^{-7}	26.8	44	42	10	1100
7	1-way	8×10^{-8}	15.5	76	61	21	840
8	1-way	4×10^{-7}	62.2	10	58	6	1260
9	2-way	8×10^{-8}	11.0	76	45	20	810

NOTE: Cases 7 & 9 use THCL's values for hydraulic conductivity of waste ($K = 10^{-4}$ cm/s) and upper till ($K_1 = 8\times10^{-8}$ cm/s).

All cases have hydraulic conductivity of waste greater than or equal to THCL's value of 10^{-4} cm/s

COLLECTION SCENARIO (c)

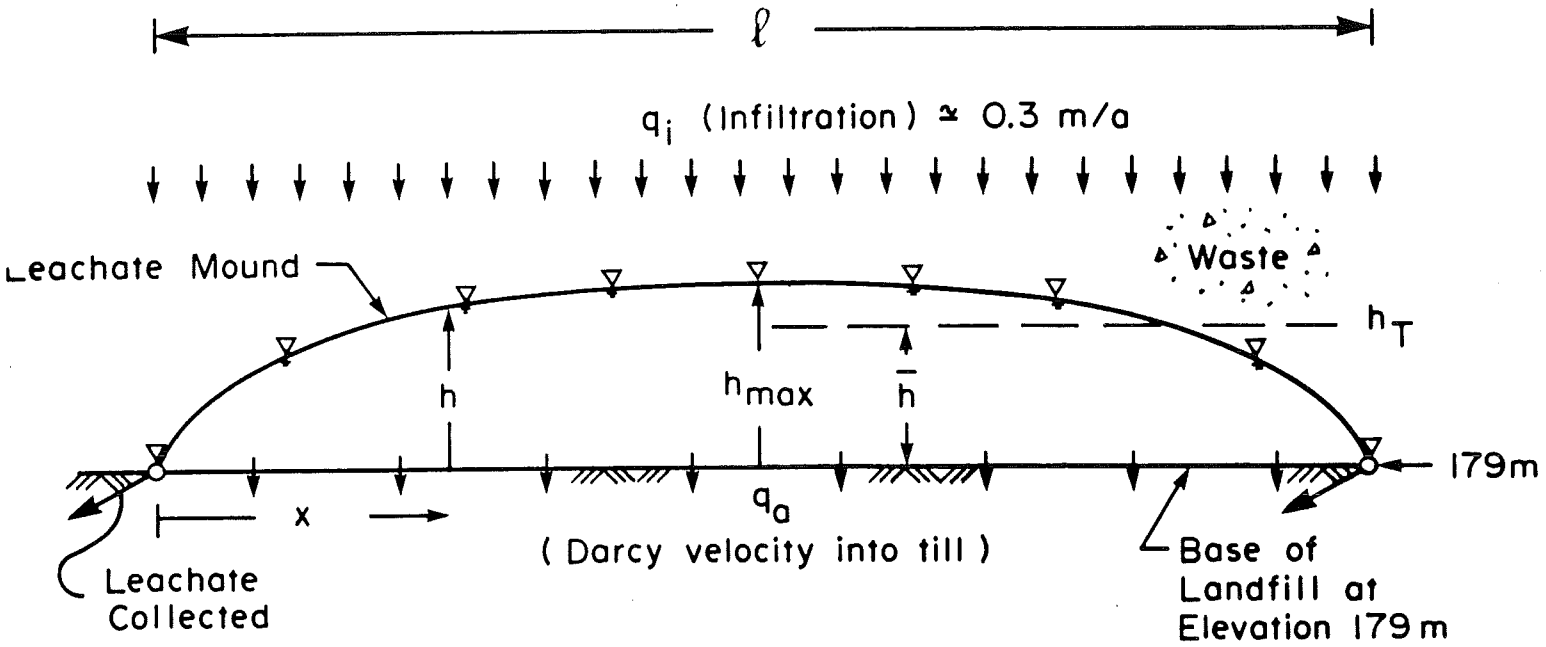
Scale (Approx)
Horiz. 1:2500
Vert. 1:500



Case	Flow	K_1 (cm/s)	Darcy "Velocity" in UGU: V_{a4} (m/a)	Leachate Collected %	Flow to Shale %	Time to Reach Boundary (years)	Peak Chloride Concentration (mg/L)
11	2-way	4×10^{-7}	2.5	82	25	36	390
12	2-way	8×10^{-7}	4.5	69	17	20	590

NOTE: Calculations assume THCL's value for hydraulic conductivity of the waste.

CALCULATION OF LEACHATE MOUNDING



Average height of leachate mound above base of landfill: $\bar{h}$

$$\bar{h} = 0.393\ell\sqrt{q/k} \qquad [6]$$

"Total head for leachate": h_T

$$h_T = 179 + \bar{h} \text{ (m)}$$

where equation [6] was derived assuming all infiltration was collected, thus

$$\frac{q}{k} = \frac{\text{Infiltration}}{\text{Nominal Hydraulic Conductivity of Waste}} = \frac{q_i}{K}$$

however, if there is a flow q_a into the till, then the leachate collected (per unit area) is

$$q = (q_i - q_a)$$

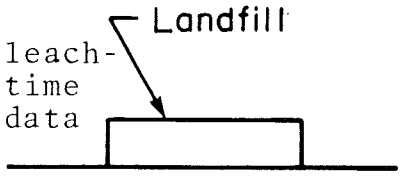
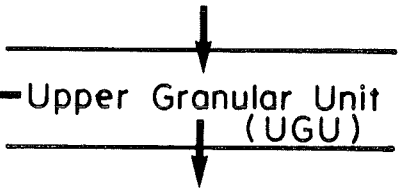
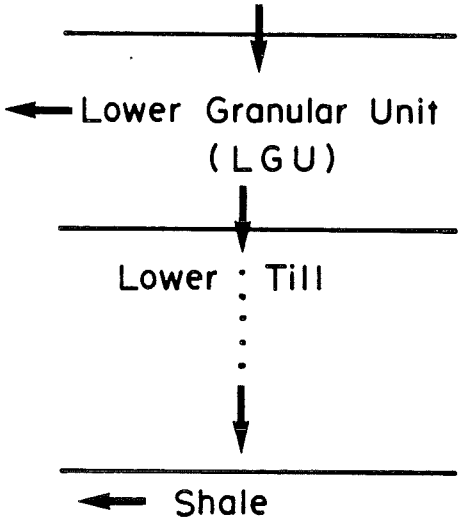
and so to achieve the average mound height calculated for $q/k = q_i/K$ the hydraulic conductivity of the waste K* may be given by

$$\frac{q_i}{K} = \frac{q_i - q_a}{K^*} \qquad \text{i.e. } K^* = K(q_i - q_a)/q_i$$

Thus the head h_T corresponds to a hydraulic conductivity in the range

$$K^* \leq k \leq K$$

DIFFERENCES IN ANALYSES CONDUCTED FOR SITE D

THCL Study		UWO Study
Assumes a variation in leachate concentration with time based on an envelop of data from other landfills		Adopts a mass of contaminant based on THCL's concentration curve but does not assume a priori how the leachate concentration will vary with time
Used the Ogata-Banks solution to the 1D advection-dispersion equation based on an infinitely deep deposit.	Upper Till	Used a semi-analytic solution to the 2D advection-dispersion equation for a layered deposit.
Only enforces continuity of concentration between layers.	Lower Till	Enforced continuity of concentration and flux (i.e. mass passing between layers per unit time).
Did not consider conservation of mass or continuity of flux in assessing contaminant transport in the UGU or underlying lower till.	 Lower Till	Considered conservation of mass when determining the contaminant moving laterally in the UGU and the contaminant moving down into the underlying lower till.
Did not consider possible migration along LGU (eg. see Fig. 3-7, Exhibit H86)		As for UGU

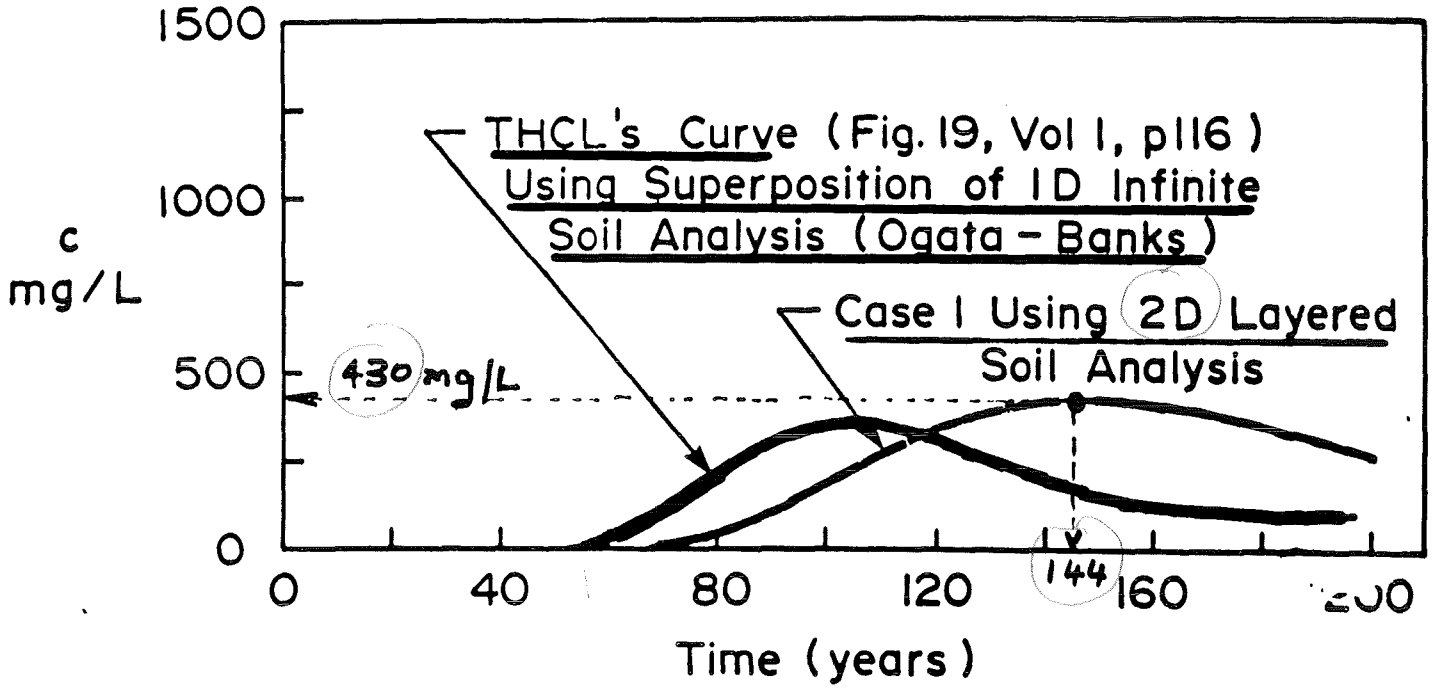


FIGURE 5 CHLORIDE CONCENTRATION AT THE SITE BOUNDARY IN THE GRANULAR UNITS: ANALYSIS BASED ON PARAMETERS GIVEN FOR THE THCL'S "NO LEACHATE COLLECTION SYSTEM IN OPERATION" CASE IN APPENDIX 15

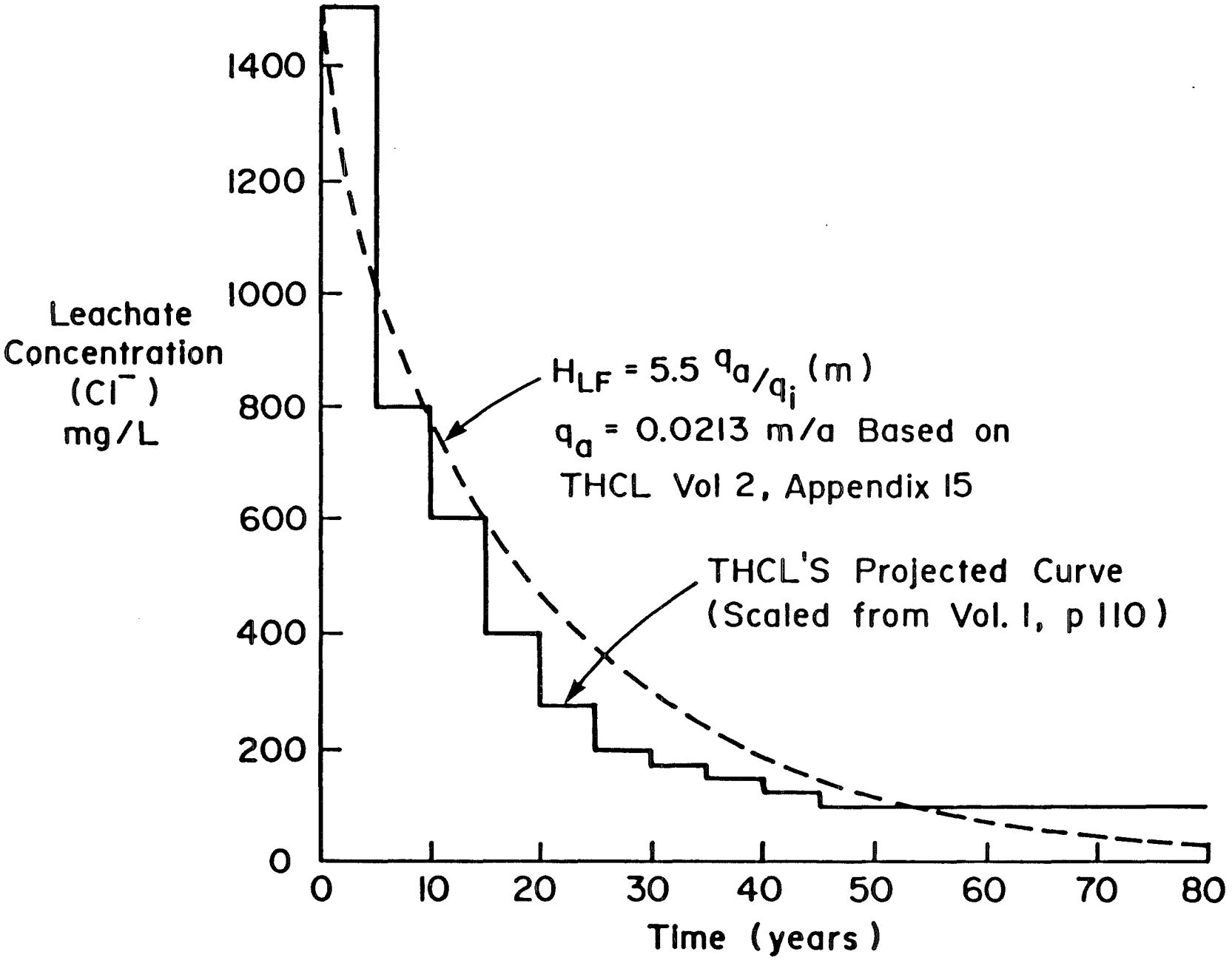


FIGURE 6 VARIATION IN CONCENTRATION IN THE LEACHATE WITH TIME

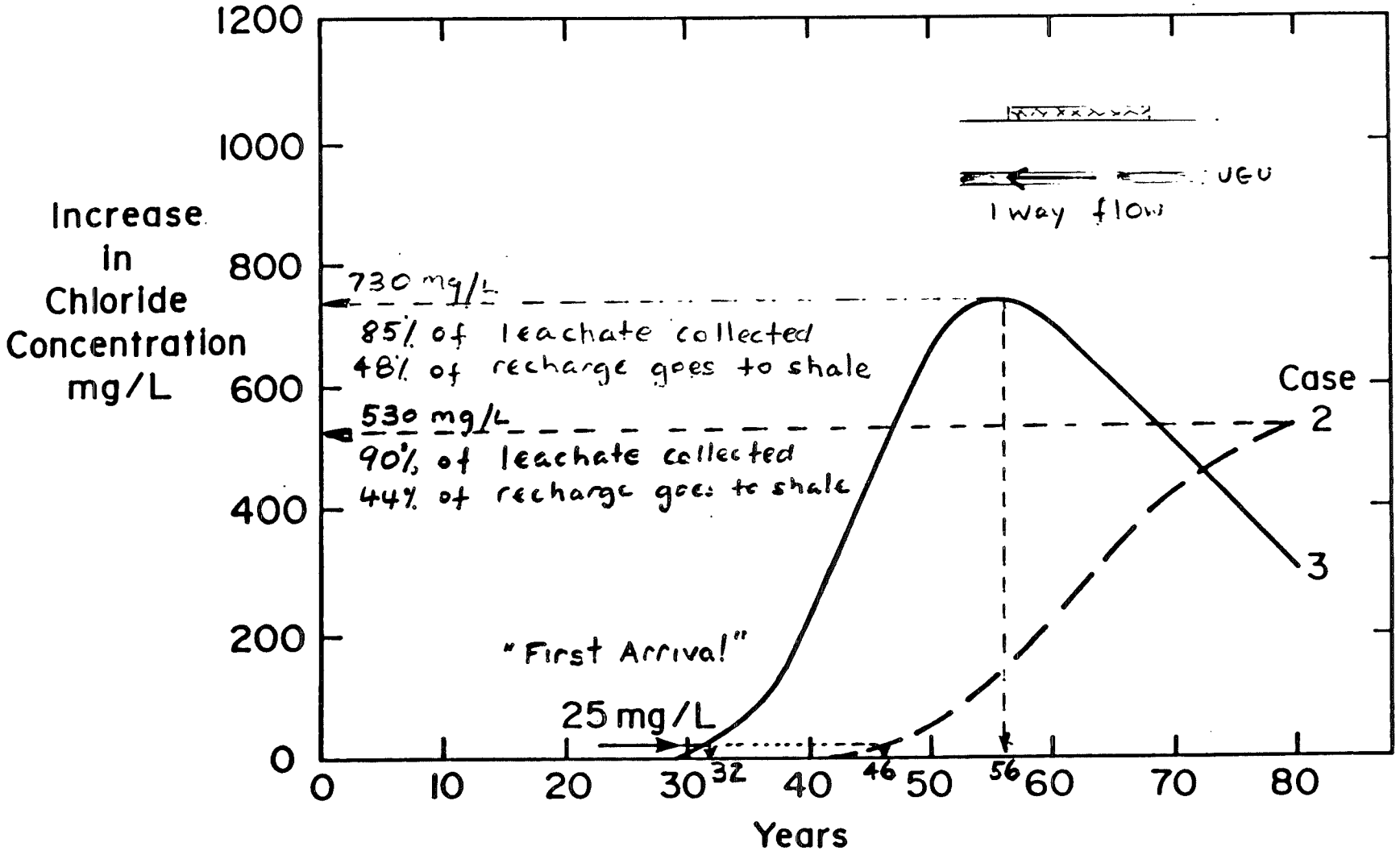


FIGURE 7 VARIATION IN CONCENTRATION WITH TIME IN THE UPPER GRANULAR UNIT AT THE SITE BOUNDARY, ASSUMING THE LEACHATE COLLECTION TILES ARE WORKING (CASES 2 & 3)

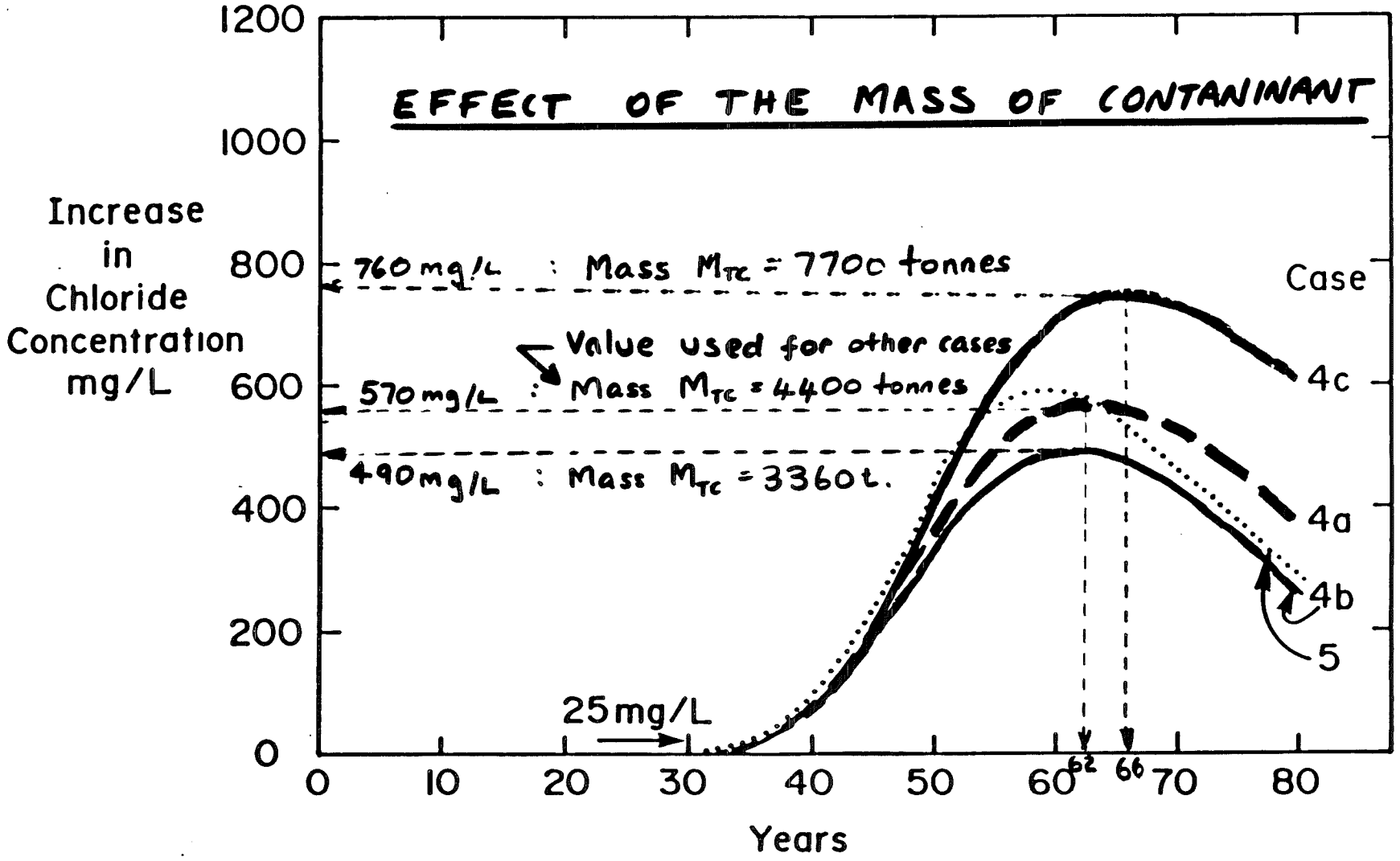


FIGURE 8 VARIATION IN CONCENTRATION WITH TIME IN THE UPPER GRANULAR UNIT AT THE SITE BOUNDARY, ASSUMING THE LEACHATE COLLECTION TILES ARE WORKING (CASES 4 & 5)

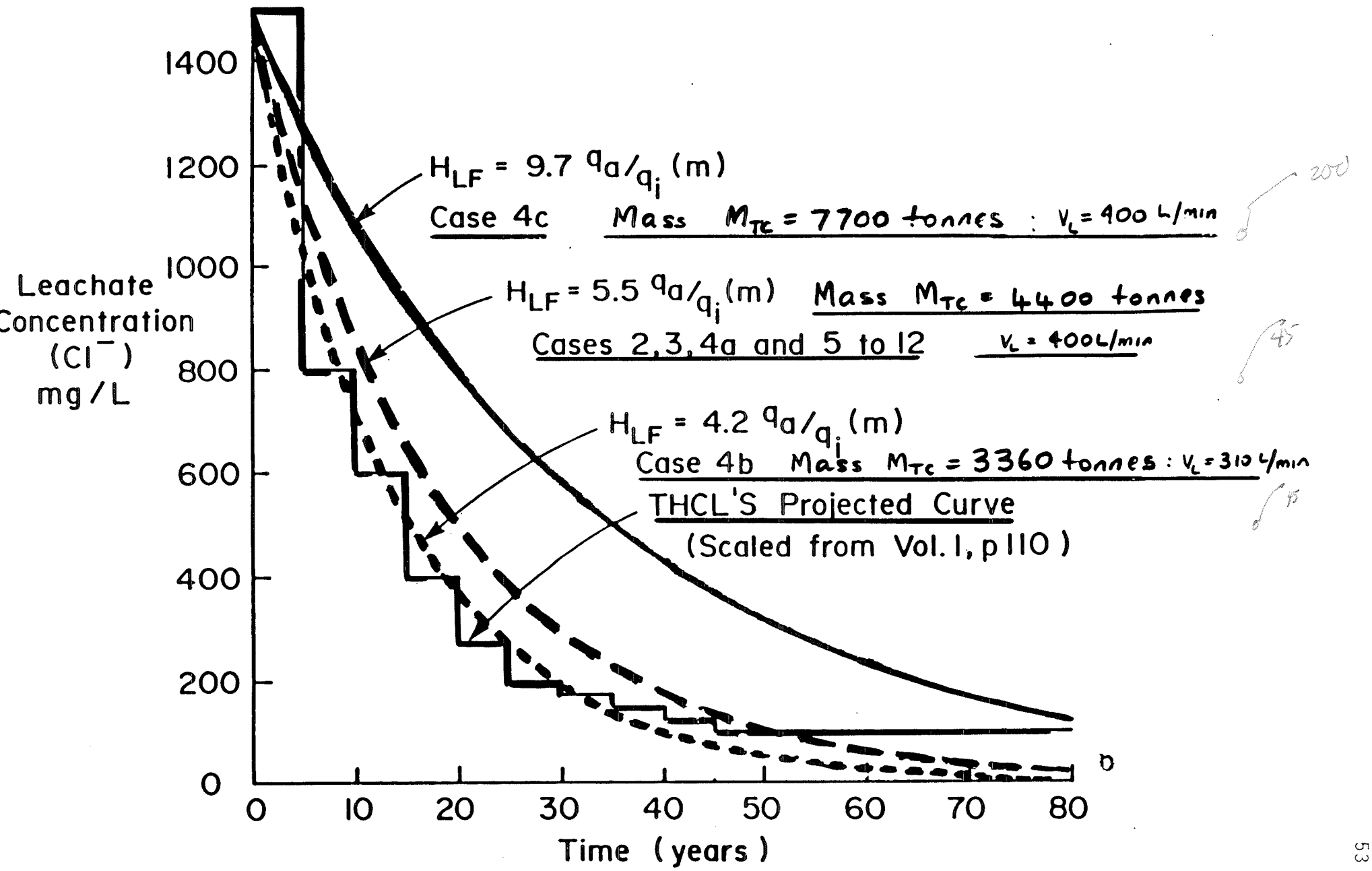


FIGURE 9 VARIATION IN CONCENTRATION IN THE LEACHATE WITH TIME

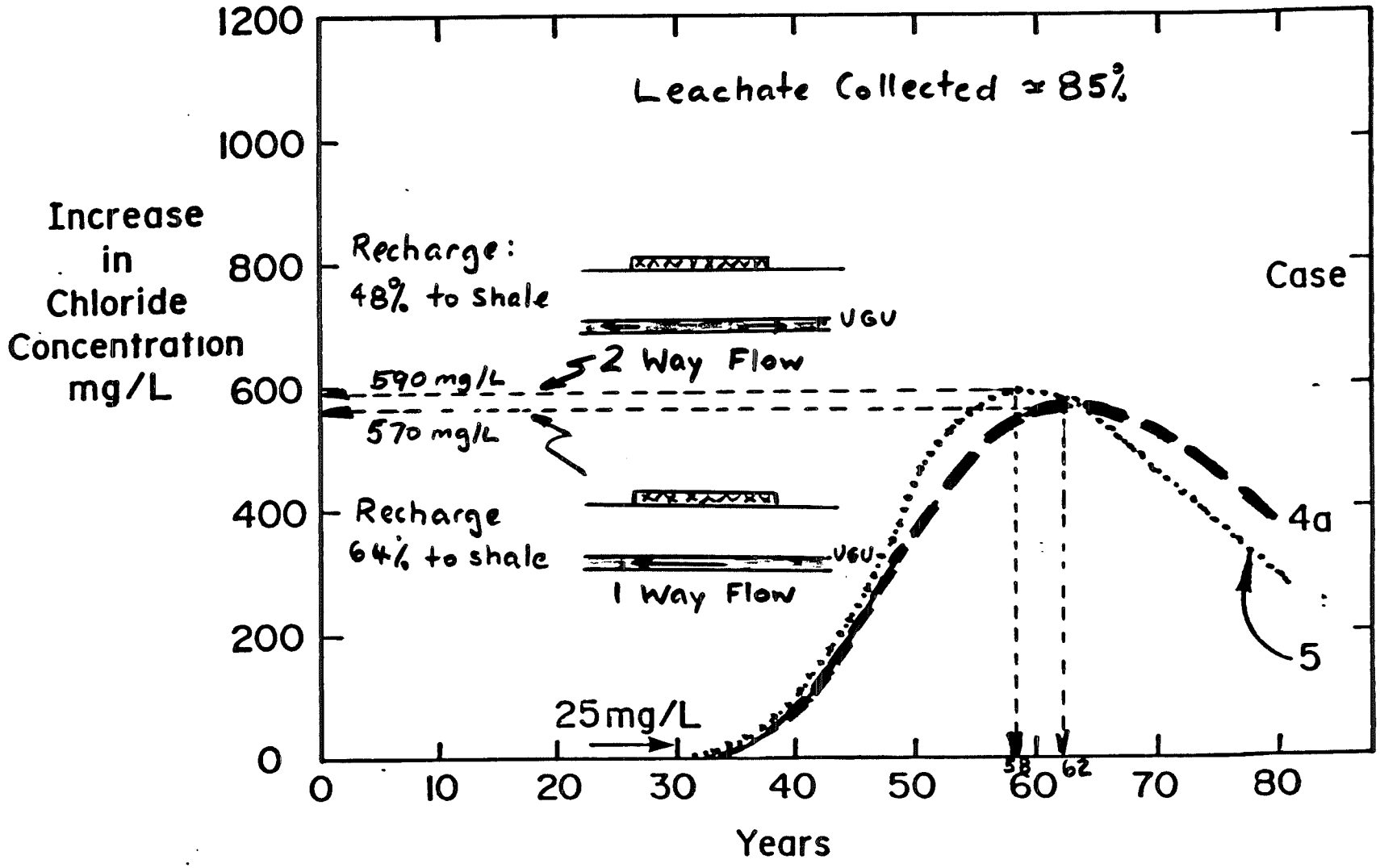


FIGURE 8 VARIATION IN CONCENTRATION WITH TIME IN THE UPPER GRANULAR UNIT AT THE SITE BOUNDARY, ASSUMING THE LEACHATE COLLECTION TILES ARE WORKING (CASES 4 & 5)

not working way designed

LEACHATE COLLECTION SYSTEM WORKING

Scenario (a)

CASE	Time to Develop Mound (1) (Years)	Time to Travel to Boundary (2) (Years)	Total Time Until Impact (3) (Years)	Time for Contaminant to move 100m once it reaches UGU (4) (Years)
2	3	46	49	3
3	3	32	35	2
4	3	36	39	4
5	3	35	38	5

NOTES:

(1) Assumes mound is developed at rate of 1 m/a (rounded to nearest year).

(2) Based on Table 3, p. 41 of Site D report.

(3) Represents the time after construction of a cell when contaminant is expected to arrive offsite assuming no contaminant transport until the leachate mound is fully developed.

(4) Most of the travel time is through the upper and lower till. Once contaminant reaches the Upper Granular Unit it moves at velocities of between 21 and 43 m/a [i.e. travel time in the UGU is 2-5 years].

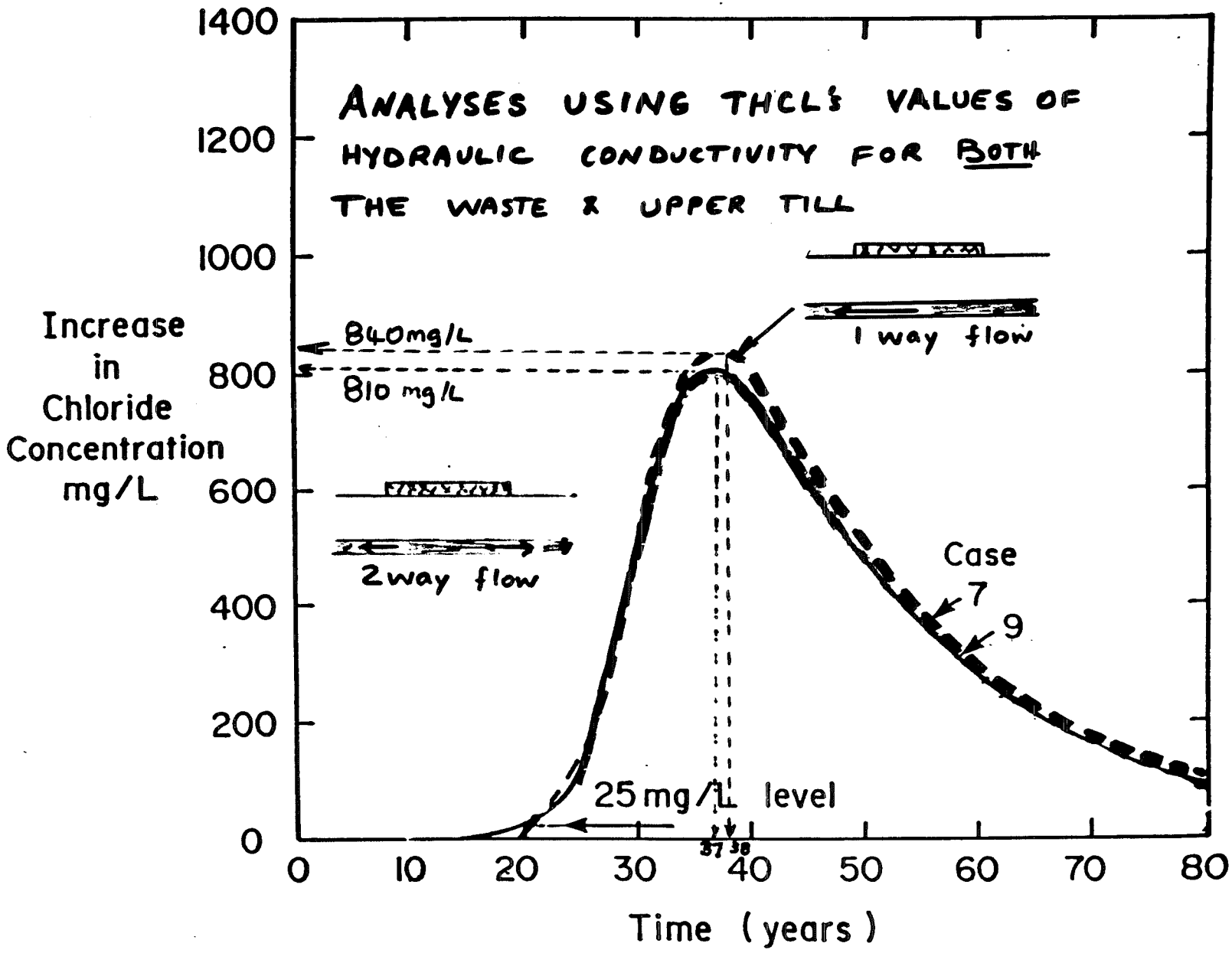


FIGURE 10 VARIATION IN CONCENTRATION WITH TIME (AFTER MOUNDING DEVELOPS) IN THE UPPER GRANULAR UNIT AT THE SITE BOUNDARY ASSUMING COLLECTION TILES FAIL (CASES 6 TO 10)

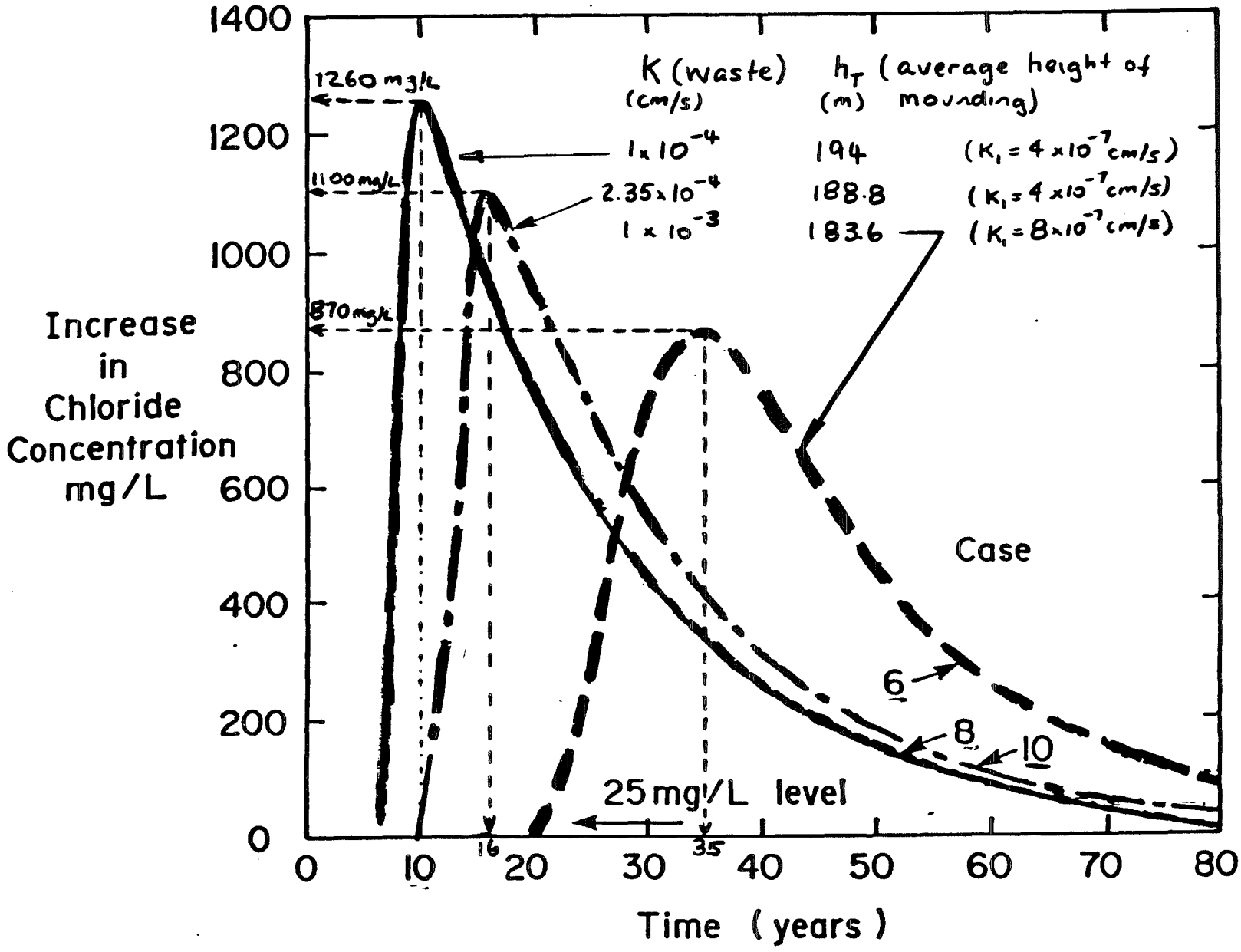


FIGURE 10 VARIATION IN CONCENTRATION WITH TIME (AFTER MOUNDING DEVELOPS) IN THE UPPER GRANULAR UNIT AT THE SITE BOUNDARY ASSUMING COLLECTION TILES FAIL (CASES 6 TO 10)

LEACHATE COLLECTION TILES FAIL IN CELLS 4 & 5

Scenario (b)

CASE	Time to Develop Mound (1) (Years)	Time to Travel to Boundary (2) (Years)	Total Time to Impact (3) (Years)	Total Time to Impact (4) (Years)	Time for Contaminant to Move 100 m Once It Reaches UGU (5) (Years)
6	4	21	25	25	1.8
7	14	21	35	32	1.9
8	14	6	20	14	0.5
9	14	20	34	33	2.7
10	9	10	19	16	1.1

NOTES:

(1) Assumes leachate mound is developed at a rate of 1 m/a starting from an initial value of the mound that THCL assume would exist (i.e. 0.785 x 1.5 = 1.2 m) if the hydraulic trap is working (rounded to nearest year).

(2) Based on Table 3, p. 41 of Site D report.

(3) Represents the time after failure of the collection tiles when contaminant is expected to arrive offsite assuming that there is NO contaminant transport until the leachate mound is FULLY developed.

(4) Represents the time after failure of the collection tiles when contaminant is expected to arrive offsite allowing for contaminant transport during development of the leachate mound.

(5) Once contaminant reaches the upper granular unit it moves at velocities of between 37 and 207 m/a [i.e. travel time in UGU is 0.5-3 years].

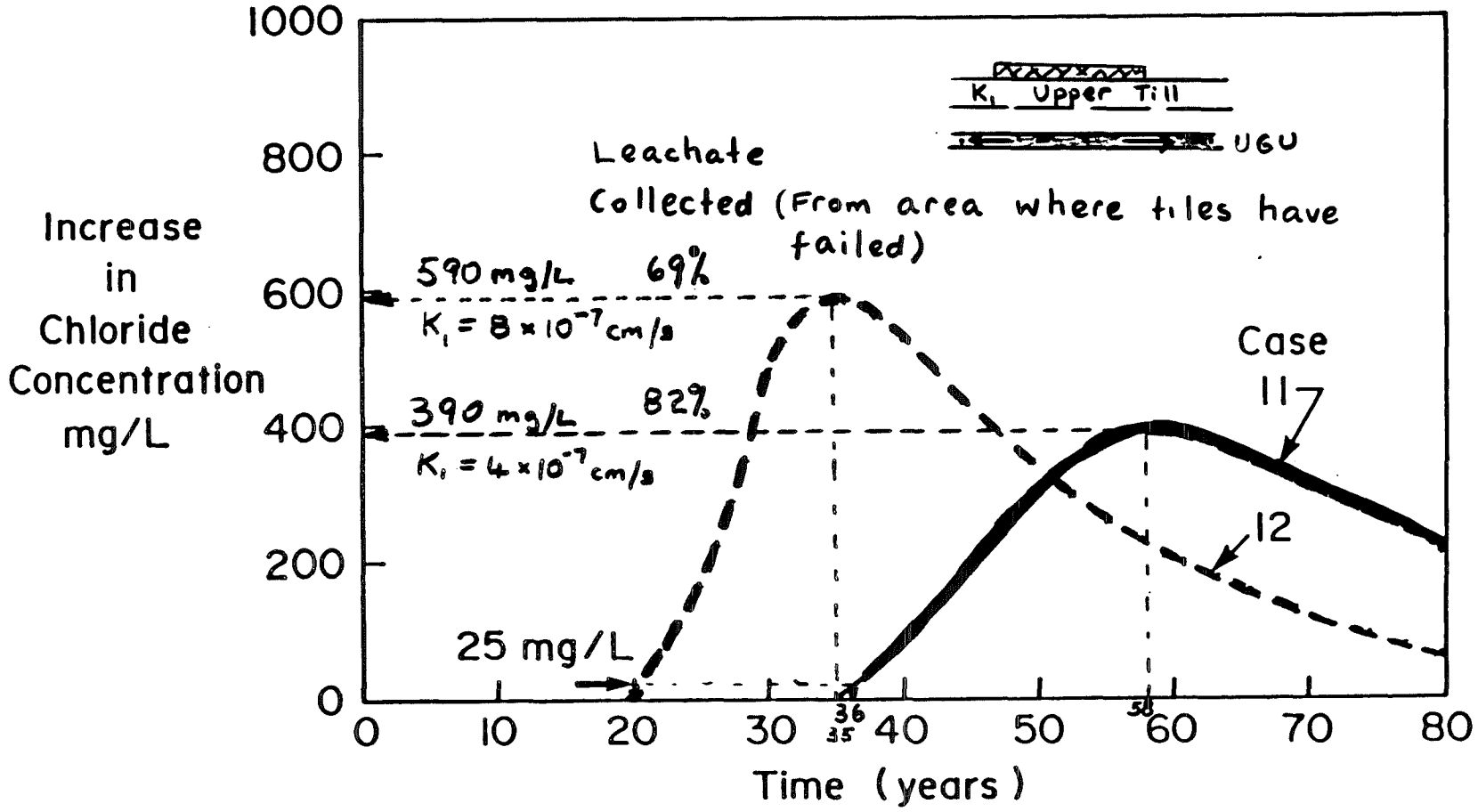


FIGURE 11 VARIATION IN CONCENTRATION WITH TIME (AFTER MOUNDING) IN THE UPPER GRANULAR UNIT AT THE SITE BOUNDARY ASSUMING 3 COLLECTION TILES FAIL (CASES 11 AND 12)

LENGTH OF TIME THE LEACHATE COLLECTION SYSTEM MUST FUNCTION

	Hydraulic Conductivity of Waste (cm/s)	Hydraulic Conductivity of Upper Till (cm/s)	Case	Proportion of Leachate Collected Locally* (%)	Proportion of all Leachate Generated**(%)	Initial Source Concentration C_o (mg/l)	Time to Reduce Boundary Concentration to 25 mg/l (years)	10 mg/l (years)
Complete Failure of Tiles	10^{-4}	8×10^{-8}	9	76%	80%	1500	65	>80
						3000	75-80	>80
	2.35×10^{-4}	4×10^{-7}	10	44%	70%	1500	70-75	>80
						3000	>80	>80
	10^{-4}	4×10^{-7}	8	10%	58%	1500	75-80	>80
						3000	>80	>80
Failure of only 3 tiles	10^{-4}	4×10^{-7}	11	82%	98%	1500	50-55	65-70
						3000	60-65	>80
	10^{-4}	8×10^{-7}	12	69%	98%	1500	60-65	75-80
						3000	75-80	>80

* This is the proportion of leachate collected in the area where the failure occured

** This is the proportion of leachate collected for the site as compared with what was theoretically predicted by Trow Hydrology Consultants Limited

PARTIAL FAILURE OF TILES IN CELL 4

Scenario (c)

CASE	Time to Develop Mound (1) (Years)	Time to Travel to Boundary (2) (Years)	Total Time Until Impact (3) (Years)	Time for Contaminant to Move 100 m Once It Reaches UGU (4) (Years)
11	3	36	39	12
12	3	25	28	7

NOTES

(1) Assumes leachate mound is developed at a rate of 1 m/a starting from an initial value of the mound that THCL assume would exist (i.e. 0.785 x 1.5 = 1.2 m) if the hydraulic trap is working (rounded to nearest year)

(2) Based on Table 3, p. 41 of Site D report.

(3) Represents the time after failure of the collection tiles when contaminant is expected to arrive offsite assuming NO contaminant transport until the leachate mound is fully developed.

(4) Once contaminant reaches the upper granular unit it moves at velocities of between 8 and 15 m/a [i.e. travel time in UGU is between 7 and 12 years].

CONCLUSIONS: SITE D

1. IF THE HYDRAULIC TRAP FUNCTIONS AS INTENDED AND IF THE HYDRAULIC CONDUCTIVITY OF THE WASTE IS LESS THAN 10^{-4} CM/S AND IF THE COLLECTION TILES CONTINUE TO WORK FOR MORE THAN 80 YEARS WITHOUT SIGNIFICANT BLOCKING OR CLOGGING THEN GIVEN THE DATA PROVIDED BY THCL AND THE CURRENT STATE OF KNOWLEDGE, ONE WOULD NOT EXPECT THE LANDFILL TO IMPACT ON OFFSITE GROUNDWATER QUALITY.

2. IF THE HYDRAULIC CONDUCTIVITY OF THE WASTE IS 10^{-5} CM/S THEN SIGNIFICANT OFFSITE CONTAMINATION OF GROUNDWATER IS TO BE EXPECTED (530-760 MG/L FOR CHLORIDE ASSUMING AN INITIAL CONCENTRATION OF 1500 MG/L).*

3. IF THE LEACHATE COLLECTION TILES WHICH CANNOT BE CLEARED (I.E. BETWEEN MANHOLES) BLOCK/CLOG (EG IN CELL 4) THEN ONE WOULD EXPECT SIGNIFICANT CONTAMINATION OF OFFSITE DRINKING WATER (390-590 MG/L FOR CHLORIDE ASSUMING AN INITIAL CONCENTRATION OF 1500 MG/L).*

4. IF ALL THE COLLECTION TILES FAIL (BUT SUBHEADERS CONTINUE TO FUNCTION IN CELLS 4 AND 5) ONE WOULD EXPECT SIGNIFICANT CONTAMINATION OF OFFSITE GROUNDWATER (810-1260 MG/L FOR CHLORIDE ASSUMING AN INITIAL CONCENTRATION OF 1500 MG/L).*

4 monitor · log Mon

5. IF THE UPPER GRANULAR UNIT IS NOT CONTINUOUS THEN CONTAMINANT COULD MOVE OFFSITE WITHOUT BEING DETECTED.

6. THERE IS A REASONABLE LIKELIHOOD THAT GROUNDWATER CONTAMINATION WILL OCCUR OFFSITE DUE TO THE PROPOSED LANDFILL AT SITE D.*

* ASSUMING THAT CONTAMINANT HAS NOT BEEN DETECTED IN THE UPPER GRANULAR UNIT AND THAT EFFECTIVE CONTINGENCY MEASURES HAVE NOT BEEN IMPLEMENTED.

COMPARISON OF TWO SITES BASED ON THCL'S DATA AND DESIGN

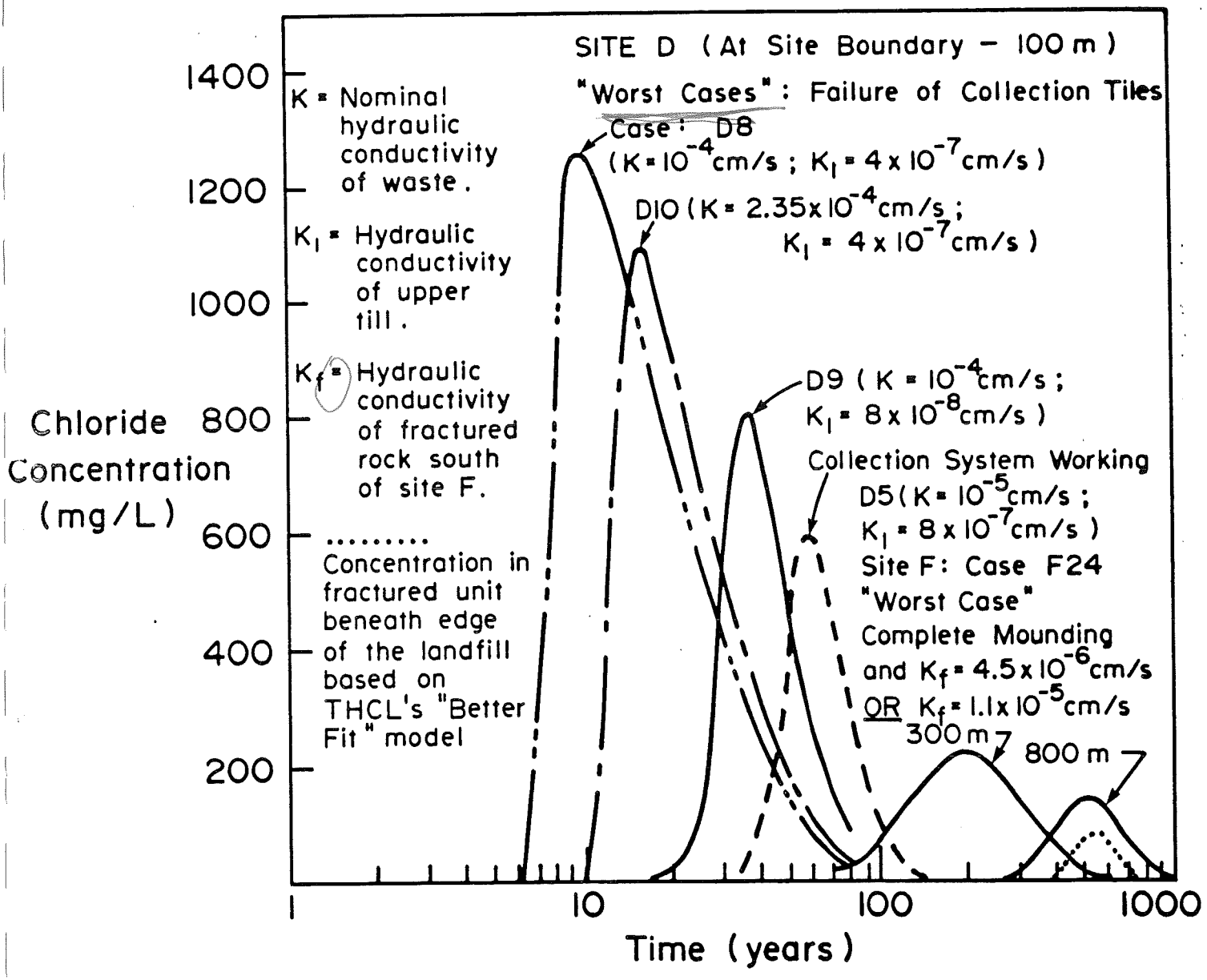
	SITE F	SITE D
THICKNESS OF "PRIMARY BARRIER"	15-20 M	7 M.
HYDRAULIC CONDUCTIVITY OF BARRIER	$K_H=10^{-7}$ CM/S, $K_V=10^{-8}$ CM/S	$K_H=K_V = 1.4 \times 10^{-7}$ CM/S*
LATERAL DARCY VELOCITY	0.2 M/A	2.5 M/A (NO HYDRAULIC TRAP)
SLOPE OF COLLECTION TILES	2%	0.25%
LENGTH OF LEACHATE COLLECTION SYSTEM PIPES	12000 M**	20000 M**
CAPACITY TO CLEAN PIPES	GOOD	FAIR
NUMBER OF COLLECTION SYSTEMS	2	1
DILUTION OF CONTAMINANT	50%	NONE
POTENTIAL FOR ATTENUATION DUE TO MATRIX DIFFUSION	EXCELLENT	POOR
POTENTIAL ATTENUATION ZONE	650-1400 M	100 M

* THCL Appendix 16 - considers both the upper and lower till
**Transcript, Vol. 48, p. 10935

* Based on water for model.

M221

COMPARISON OF IMPACTS CALCULATED AT BOTH SITE D AND SITE F



NOTE: Potential impact at Site F is minimal even with conservative assumptions. Potential impact at Site D is significant if there is any failure of the hydraulic trap.

Mass of contaminant adopted in the analysis:
For Site D: 4400 tonnes of Chloride
For Site F: 7800 tonnes of Chloride

SITE D: THCL DATA FOR BOREHOLES IN UPPER TILL ON AND PROXIMATE TO AREA OF LANDFILL

BH	Depth	% Clay	% Silt	% Sand	% Gravel	Description by KR and DC	Description by THCL
HCl-85	5.64-5.94	17	40	18	25	Gravelly silt till, some sand, some clay	Clayey silt till,some sand
2-85	4.27-4.57	17	41	25	17	Sandy silt till, some clay, some gravel	Sandy clayey silt till
3-85	4.94-5.24	21	40	25	14	Clayey sandy silt till, some gravel	Sandy clayey silt till
4-85	4.11-4.42	21	39	30	10	Clayey sandy silt till, trace gravel	Sandy silt till, some clay
5-85	5.18-5.49	23	42	18	17	Clayey silt till, some sand, some gravel	Clayey silt till,some sand
6-85	5.03-5.33	19	42	22	17	Sandy silt till, some clay, some gravel	Clayey silt till,some sand
Average Value		20	40	23	17		

Drainage and Filters

6A/3

Third International Conference on Geotextiles, 1986, Vienna, Austria

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FERRIC CLOGGING OF DRAINS
DIE EISEN-VEROCKERUNG VON DRÄNS
LE COLMATAGE FERRIQUE DES DRAINS

Typical cases of iron clogging of drains have been investigated on pyrenean border. One originated with the intermediary of an autotroph bacteria, Gallionella ferruginea, from a water bed rich in ferrous iron but poor in organic matter. The others resulted from the conjunction of mechanical (silicates) chemical (iron oxides) and biological cloggings. The later is initiated by Sphaerotilus natans or Siderococcus sp..

An application to the forecasting of iron clogging risks sketched.

INTRODUCTION

Iron clogging of wells, canalizations and drains is a phenomenon known since antiquity; the leading role of any bacterium in the process was only described 150 years ago. Early works did not make a distinction between biological iron oxidation and its chemical precipitation, consequently, ferrous bacteria were assimilated to autotrophs, taking their energy from ferrous iron oxidation.

The experiments carried out after that by MOLISCH (1892) enabling actually to differenciate two groups of bacteria : those which oxidize ferrous iron (a few species) and those which precipitate it by deteriorating the organic ligand (numerous soil and water bacteria).

Despite bacteria action in the phenomenon, purely chemical oxidation is not to be disregarded, and in the field (1,2,3) make a major contribution to the analysis of dissolved iron behaviour and to the evolution of ferruginous water.

Since we know fairly well the development process today, it is quite impossible to estimate accurately the iron clogging appearance and intensity in a given place. Research work carried out by the CEMAGREF (4,5) shows that it is not an easy problem and that any contribution to its solution will be welcome. Our own work concerns typical cases observed on the territory of CAPVERN (Hautes-Pyrénées) and LESCAR (Pyrénées Atlantiques).

1 - THE SITE OF CAPVERN DISTRICT

1.1. Clogging by Gallionella ferruginea

The drain laid along the railway line following the Pontian-Pliocene horizon (red sandy clays with shingle beds) has major ochre gel deposits in its terminal portion and in the evacuation ditch. It captures water from sloping grounds including moors, meadows and a few cultivated plots. Its flow is constant all year round and varies between 4.5 and 6 litres per minute.

Investigation on clogging risks, in direct connexion with iron compounds precipitation, was performed by means of biological and chemical analyses carried out on water samples spread out between october 1982 and june 1983.

Chemical analysis of the water

The results of the analysis are stated in Table 1. The water coming out of the drain is odourless and is not coloured but yet it con-tains ferrous iron whose average content is 15 mg/l in one year.

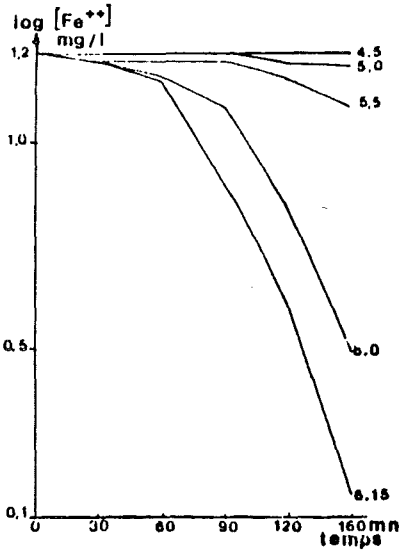

Figure 1
Evolution of the iron oxidation speed in natural water according to pH.

Table 1
Analysis of the water from the CAPVERN railway drain (from september 1982 to june 1983). Concentrations in mg/l.

	29.9.82	24.10.82	14.11.82	11.12.82	15.1.83	19.2.83	20.3.83	24.4.83	15.5.83	12.6.83	26.6.83
T°	14,5	13,8	13,1	13,0	12,5	12,0	11,5	11,0	11,4	11,8	12,0
pH	6,20	6,30	6,10	5,90	6,20	6,10	6,15	6,30	6,15	6,30	6,20
Eh	+ 65	+ 68	+ 64	+ 65	+ 70	+ 72	+ 70	+ 68	+ 70	+ 60	+ 90
Fe^{2-}	11,0	9,60	11,70	12,0	12,20	19,60	19,20	18,60	13,60	14,60	11,40
Fe^{tot}	11,60	10,60	12,30	12,50	12,90	22,0	21,0	19,60	15,60	15,10	12,20
Mn	0,55	0,59	0,58	0,63	0,58	0,46	0,52	0,48	0,50	0,50	0,52
Ca^{++}	20,50		23,20		27,2	27	24,5	25,0	21,5	18,2	15,20
Mg^{++}	1,58		1,70		1,95	1,75	1,70	2,0	1,45	1,50	1,30
Na^{+}	4,0		5,10		6,50	4,90	4,10	4,0	3,65	3,40	3,30
K^{+}	0,52		0,50		0,72	0,36	0,34	0,38	0,50	0,34	0,54
NH_4^{+}	0,33		1,60		2,0	2,40	2,50	1,50	3,50	3,80	2,40
NO_3^{-}	0,71		0,45		0,35	0,24	0,15	0,10	0,08	0,05	0,09
HCO_3^{-}	64,40		92,50		102,60	107,60	105,5	102,5	97,57	95,35	74,74
SO_4^{-}	10,40		15,0		17,0	18,80	10,5	6,25	3,75	1,73	7,76
Cl^{-}	17,80		16,0		15,0	15,24	15,7	15,95	12,40	8,85	7,10

The pH, Eh and the O_2 content are the three elements which affect mostly iron solubility; with the latter relatively low (0.5 mg/l), it is difficult to assess the pH or Eh action by using the theoretical curve calculated by (1). In order to value the pH action on iron resis-tance when dissolved, we acidified various water samples; then, we observed the iron oxidation (the water was aerated by means of magnetic agitation). Figure 1 shows the results of this experiment (the pH ground average is 4.9).

For a pH of 5.5, the oxidation is low during the first hours, and becomes even nil for a pH of 4.5. For a pH close to neutrality, a difference of 0.1 pH unit leads to very different oxidation speeds. In the experienced system, the pH is mainly controlled by the CO_2 HCO_3 equilibrium; so, it is easy to see the microorganism action (which produces CO_2 by breathing) or the water rippling causing a CO_2 release. However, the physico-chemical factor and mineral compounds are not the only ones which act upon iron solubility. Organic compounds also have a direct (complexing) and indirect action (bacterial development) leading to the rise or limitation of iron solubility.

The investigation on water oxidability ($KMnO_4$) and carbon dosage (2.4 mg/l) showed in water a relatively poor organic matter. However, organic compounds can be qualitatively interes-ting. The analyses which are carried out in various steps concern four groups of compounds : sugar, amino acids and phenolic compounds :
- Sugar is not detected in samples, which indi-cates that it is quickly assimilated at ground level.

The amino acid analysis shows, at drain's outlet, a quantitatively and qualitatively depleted mixture compared with the draining water from soil A-horizon.

As far as organic acids are concerned, the only detected compound has a phenolic characteristic and corresponds to gallic acid, which is not surprising as this last one originates from lignins decomposition.

The analysis of draining water organic com-pounds reveals mainly as follows :
- the organic matter which can be used to feed heterotrophic microorganisms is chiefly composed of amino acids;
- if we take into account the quantity, this organic matter cannot play a major role in the transport of iron in solution.
However, if we consider the nature of compounds which have been observed (sulphurous amino acids, gallic acids), we may think that those solutions can participate to the iron reduc-tion.

Chemical analysis of ochre

The sediments which were taken from the drain's final part (Table 2) have a low silica content and a very high iron one, which is due to the presence of Gallionella (6). This value is close to the maximal ones stated by different authors, that is 65.9 % (7). On the other hand we note the absence of Calcium and Magnesium.

Finally, the low C/N confirms the idea in which the medium is poor in compounds resulting from the plants deterioration (high C/N)

this relation is close to that of bacterial cells composition.

Table 2

Chemical analysis of CAPVERN drain deposit

SiO_2	16,5 %	K_2O	0,5 %
Fe_2O_3	59,6 %	Na_2O	0,2 %
Al_2O_3	3,4 %	OH_2	15 %
FeO	-	C	1,98%
TiO_2	0,2 %	N	0,17%
CaO	-	C/N	11,6
MgO	-		

Microbiological analysis of ochre

* Heterotrophic bacteria

They were isolated from sediments of drain's final part. Two mediums were used for their culture :
- an ammoniacal iron citrate medium (8);
- a peptone glucose medium used during the culture of Sphaerotilus natans.

Among the investigated microbial population, we note a high number of negative-gram chromogenic bacteria and the absence of positive-gram sporulated bacteria.

By observing the isolated elements, we attempted to assess the influence they might have on the iron deposit. (In the drain, they only play a minor role if we take into account their concentration and the fact that the iron circulates in the ionic form). For this, the iron oxidizing process was observed by comparison with an element having no bacterium; when bacteria are present, it has been noted that the oxidizing process rate is low (the endogenic respiration consumes oxygen), but at the end of the experiment, that is four hours later, the iron precipitation was more complete. The supernatant solution is clear, which means that there is coprecipitation of iron and bacteria. It has also been observed that there is no difference between the tested bacterial germs. This may cause an indirect action : Gallionella develops more easily when Pseudomonas is present (9).

* Autotrophic bacteria

If we consider the physico-chemical characteristics of the medium, we may say that the precipitated and oxidized iron comes greatly from the iron's autotrophic bacteria. The evidence of these bacteria was obtained by comparing a sample of normal water and a sample of water added with Thiomersal with a concentration of 0.01 %. Results are specified in Figure 2. Then, we searched for the three main species : Thiobacillus ferroxidans, Metallogenium symbioticum and Gallionella ferruginea.

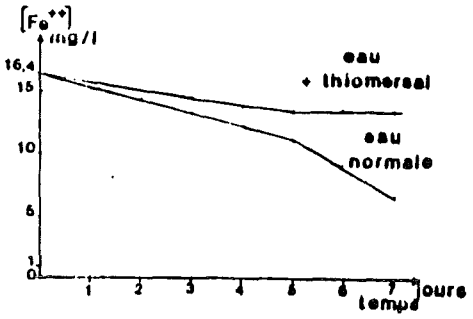

Figure 2
Evidence of iron's biological oxidation in natural water.

The first two bacteria were not detected in the drain water. On the contrary, the presence of Gallionella was directly observed by microscopic examination; actually, the twisted filaments are sufficiently typical to enable the identification of the bacterium in the deposits (Photo 1).

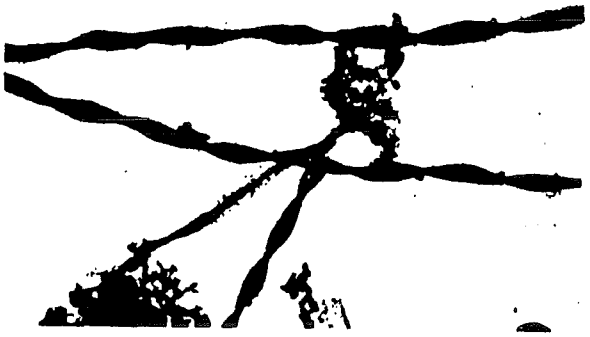

Photo 1
Gallionella ferruginea : typical twisted aspect (X 2400 optical microscopy).

We also obtained the growth of the bacterium on the medium proposed by (10); the isolated microorganism corresponds to the Gallionella ferruginea one. We enumerate the bacterium by extinction, using its property to reduce nitrates to nitrites (11) by means of dilution and seeding in the KUCERA and WOLFE (10) medium + nitrates (50 mg/l). The number of $4.5 \cdot 10^4$ germs/ml is important, especially considering the fact that there is a fixed bacterium.

1.2. Iron clogging of a geotextile by Siderococcus.

This is about a clogging observed on a temporary drain carried out in geotextile configuration alongside a motorway construction. The dissolution of the ochre which is deposited on fibers by oxalic acid enabled us to observe heterotrophic bacteria belonging to the Siderococcus species (Photo 2). These bacteria do not directly precipitate iron on their cells : the group becomes impregnated with ferric oxides, then with clay and calcite (lime treatment) which forms a sleeve around the fiber whose diameter doubles or triples. We have to note that the investigated drain portion was located in the open air. Average manganese and iron concentrations of water reaching the drain were respectively 1.22 mg/l and 0.20 mg/l.

Photo 2
Geotextile fiber treated with oxalic acid, showing bacterial sheath.

2 - THE LESCAR 'S DISTRICT SITE : Clogging by Sphaerotilus natans.

The investigation concerning this clogging which was carried out posteriorly, as we could only visit the site after the draining system was restored (installation of a new drain and lime treatment of soils). However, as the old drain remained in position, the following observations were made possible :

The drained grounds correspond to a peaty bog area; development was favoured by a trough-shaped topographic site.

The drain's feed water runs in the MINDEL's alluvial deposits when it picks up iron; at the peaty area level, as it becomes acid, the water solubilizes organic matter, stimulating in that way the microorganisms proliferation.

The drain (in ringed plastic, 100 mm diameter, laid at a depth of between 80 to 100 cm) is clogged (it was inoperative one month after being laid). Outside, it has no ferric deposit; inside, it is covered by a whitish bacterial gel. In its final part, the gel becomes rusty on the surface (under the rusty layer, a whitish one remains) (photo 3). The drain

Photo 3
Opened portion in the LESCAR's drain showing different kind of deposits.

pores reveal small ferric oxide protuberances (these protuberances surrounded pores but did not block them). The drain final part and and the discharge ditch have major ochre deposits. As the drain is almost no longer flowing only a partial water analysis was carried out; it concerned its pH : 6.05;its Redox potential : + 70 mv, and its Fe2 + content is 6.2 mg/l. In the adjacent grounds, in order to reach the bed, a bore enabled us to measure its ferrous iron content, which is 18 mg/l. Some parts of water-logged beds have pools where we can see a high development of rusty bacterial gels composed of fibrous bacteria.

The analysis of the deposit which was taken from the inner part of the clogged drain (table 3) shows the importance of silica (50 % compared with the 16.5 % in the CAPVERN site). The C/N relation (21.5 compared with the 11.4 in CAPVERN) is high; a major part of the organic matter comes, at this point, from peat.

Table 3
Chemical analysis of LESCAR's drain deposit

SiO_2	50 %	K_2O	1,0 %
Fe_2O_3	19,2%	NO_2O	0,6 %
Al_2O_3	8,8%	OH_2	10,4 %
FeO	-	C	4,7%
TiO_2	0,6%	N	0,22%
CaO	0,6%	C/N	21,5 %
MgO	0,3%		

In this case, three types of clogging are observed : a biological clogging (bacterial gels), a mechanical clogging (silica) and a chemical one (iron oxides).

As to the role of microorganisms, in this area rich in organic matter, the heterotrophs, which act indirectly, must be incriminated : they deteriorate the organic matter, which releases the linked iron; the latter becomes chemically oxidized and precipitates on bacterial gels. The microbiological analysis showed the existence of several bacteria such as the Pseudomonas ones and especially wrapped bacteria (photo 4) which is part of the Sphaerotilus type (these ones are responsible for the whitish gels inside the drain).

If we consider these analyses, the following clogging sequence can be stated : the development of fibrous bacteria wrapped in their case forms the bacterial gel; the deteriorated organic matter and microorganisms release chelated iron; this one becomes chemically oxidized and precipitates on gel surface. In addition, drifted mineral particles get trapped in the bacterial matrix. Within the deposits, we consequently reach an oxidized surface layer (rusty) and at a greater depth (in contact with the drain) a thin whitish layer, rich in mineral particles.

Photo 4
Sphaerotilus natans : sheathed fibrous bacteria (X 5550 electron microscopy).

3 - CONTRIBUTION OF OUR INVESTIGATION TO IRON CLOGGING DIAGNOSIS.

Before laying properly a drainage system, various investigations and analyses, aiming at determining whether there is any risk of iron clogging, are recommended Initially, a geologic map examination enables us to learn the formation process of soils. The parent rock is a factor which, in many cases, may supplant the role played by soil and vegetation. So, close to the CAPVERN site, in contact between the Pontian-Pliocene formation (sandy clay alluvial deposits) and LANNEMEZAN upper formation (quaternary alluvial deposits rich in clay),the disappearance of dissolved ferrous iron in the water of Pontian-Pliocene formation was assessed. We note that the difference existing in water composition is not due either to soils (brown acid soils in both cases), vegetation(which is an acidophilic one), topography (slope does not change when we switch from a formation to another) or to pHs (which are inferior to 6 in both cases).

However, vegetation and soil influence in iron clogging should not be minimized (autochthonous iron), consequently and secondly, a classical pedologic analysis will provide interesting information as to soil clay wealth, its acidity, its hydromorphy and its iron and organic matter content.

Natural vegetation is also of interest if we consider the role played by organic compounds in iron transport and reduction, and by aliphatic organic acids (citric acid) in heterotrophic nutrition; in addition, its examination enables us to know immediatly the soil hydromorphy degree (typical vegetation of hydromorphic mediums).

The first data can be optimized by examinations aiming at determining in ditches or pools rusty products with a gelatinous aspect and intensive irisations due to bacterial veils, which is a sign of important activity (12). Finally, the analyses of the bed water (Fe^{2+}, organic matter, pH, Eh content)enables, using previous results, to set up a diagnosis on the iron clogging risk.

Following the study carried out at CAPVERN and LESCAR, we may say that, actually, there are three main situations :

- calcareous areas where water is free from any ferrous iron and where clogging risks are extremely low. However, all risks are not eliminated if we consider the possible heterogeneity of organic matter formation which may form a superficial reducing horizon (autochthonous iron);

- Non calcareous areas where water contains ferrous iron but little organic matter. There may be a clogging risk but it depends on the development of the autotrophic bacteria as the Gallionella one (i.e. the drain of CAPVERN. In this case, we may state the following clogging sequence :

When water penetrates into the drain,ferrous iron is partly chemically oxidized by atmospheric oxygen and biologically oxidized by autotrophic bacteria. Consequently, there is a formation of a ferric hydroxide soil, and with other ions and bacterial substances, and precipitation. As the ferric hydroxide has an important exchange capacity (13), it will combine with various water cations and anions. The formed gel traps, then, organic and mineral particles and, as a consequence, becomes a substrate favourable to the development of the heterotrophic bacteria; finally it will result in ochre : a rusty sediment with a gelatinous aspect. In this typical case, it is interesting to know the water iron content and the number of bacterium oxidizing ferrous iron in order to

evaluate the iron clogging amplitude and rate of incrustation.

- The non calcareous areas both rich in ferrous iron and organic matter (LESCAR), favour a direct development of heterotrophic bacteria (Sphaerotilus), as their adhesion is stimulated more by the ringed-shape drain.

Clogging is very rapid there, and in this case, we may wonder whether this type of drain should not be abandoned. The lime treatment of the ditch revealed that it was immediatly efficient, but duration of the effect is to be observed (in the long run). As for us, we noted on CAPVERN's motorway site that the use of lime did not prevent the growth of Siderococcus bacteria, which is responsible for clogging of part of geotextile filter surrounding the drain.

CONCLUSION

The two cases of iron clogging investigated have only the ferrous iron resource of drained water in common. This is a factor of the utmost importance as the allochthonous iron cannot be definitely eliminated by punctual methods, such as lime treament. The absence of energetic organic matter does not prevent clogging due to possibilities of rising settlement of the Gallionella ferruginea autotrophic bacterium. In addition, this one paves the way to heterotrophic biological or mechanical clogging. The latter has a higher scope and rate (the LESCAR drain no longer ran after a month). Simultaneous and abundant organic matter and ferrous iron in water, undoubtedly outlines the example when the building of drainage systems may be questioned (4).

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Permittivity of Geotexiles in Presence of Water and Pollutant Fluids

1. INTRODUCTION

The use of geotextiles has been increasing during the last 20 years in multifarious fields of civil engineering, and it's still increasing today. These materials can perform different, mechanical and hydraulic functions, depending on the specific application area.

When used in sanitary landfills, geotextiles generally perform hydraulic functions (filtration and drainage), and also of mechanical protection for the synthetic liner (1). In most cases, geotextiles are not supposed to come into contact with leachate, and interactions can be occasionally originated from putting-in-place defects, or from long-term deterioration of the geomembrane. On the other hand, there are some design solutions, in which a geotextile is used as filter and drainage for leachate collection systems (see sketches in fig. 1 a-b); thus, geotextiles come directly into contact with leachate since the beginning of sanitary landfilling operations, and they should be properly selected and designed for such working conditions.

Interactions between leachate and geotextiles can include both physical (hydraulic) and chemical phenomena. However, chemical aspects are not considered in this preliminary research work, and only the influence of leachate on permittivity of geotextiles is investigated.

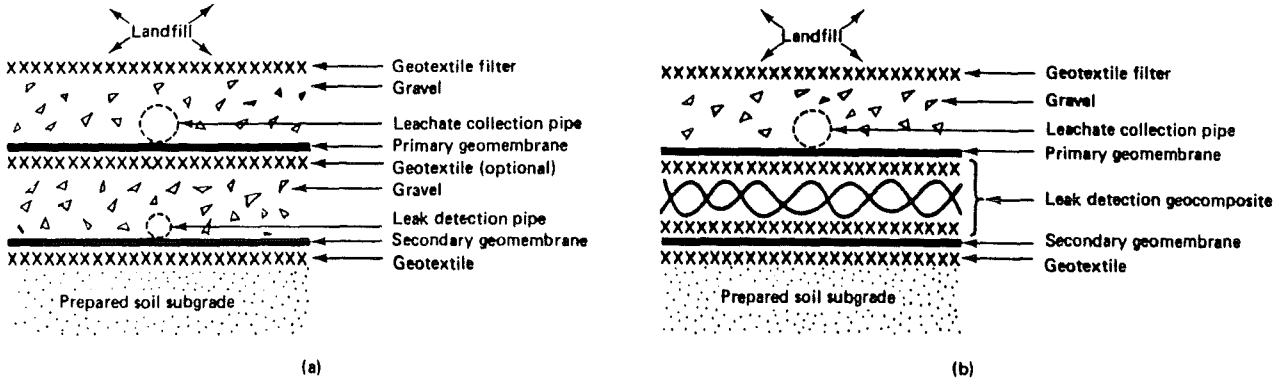


Fig. 1. Examples of technical solutions for double-lined landfills with leachate collection systems (from (1)).

2. EXPERIMENTAL WORK

The testing program included permittivity tests on several kinds of geotextiles, selected among the most commercially available in Italy. Two sets of tests were performed, accordingly to the type of fluid: demineralized water, and leachate.

2.1. The Testing Apparatus

All tests were carried out by means of a permeameter, that was purposely designed for geotextiles and realized at ENEL-CRIS laboratories in Milan (2) (3). Most parts are made with plexiglass: in this way, a direct observation of all phenomena rising during permeability tests is possible. The apparatus hydraulic scheme is shown in fig. 2; the basic elements are:
- the permeameter;
- the feeding group;
- the flow measuring group.

The permeameter is formed by 3 coaxial cylinders, having internal diameter of 284 , 124 and 51 mm, respectively. The internal cylinder houses, at its base, the geotextile specimen to be tested, and, at the same time, it is the feeding cylinder for the permeability test. The intermediate cylinder acts as a constant level cylinder; the water exceeding the spillway is sent, by an overflow pipe, to the feeding tank.

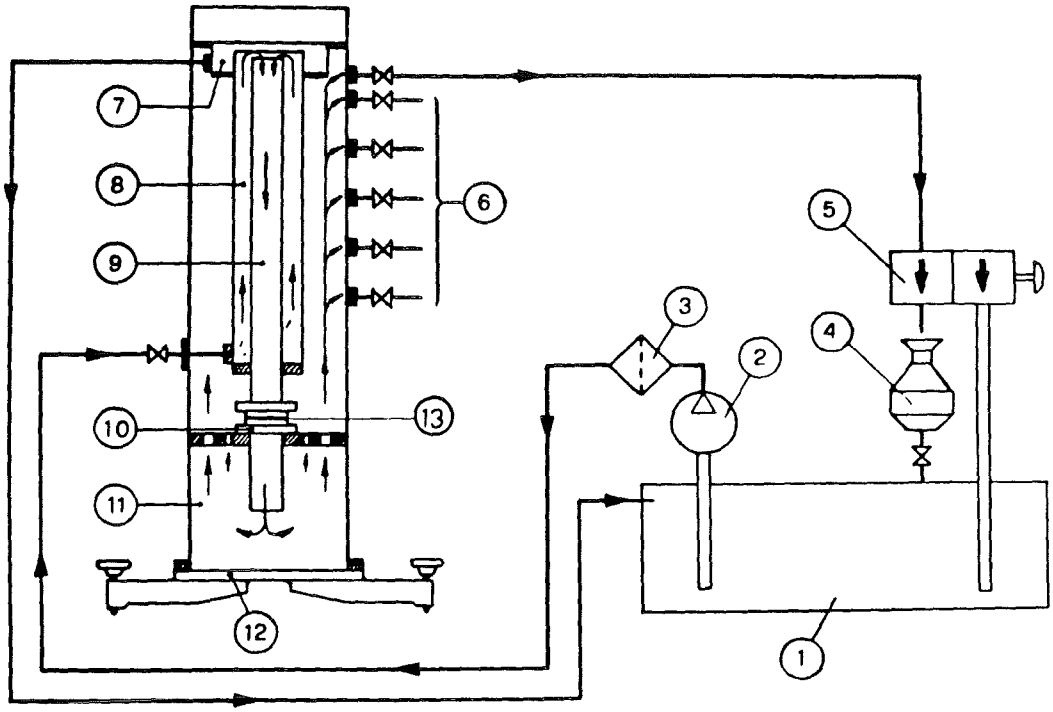


Fig. 2. Hydraulic scheme of the closed circuit, constant head permeameter for permittivity tests: (1) backwater tank; (2) feed pump; (3) filter; (4) calibrated container; (5) two-way valve; (6) outlets (6 positions for 6 different levels); (7) overflow system; (8) constant level cylinder; (9) feed cylinder; (10) specimen bearing; (11) external cylinder; (12) base; (13) geotextile specimen (from (2), (6)).

The external cylinder is the support of the entire apparatus; it has 6 openings, at 6 different levels, so that 6 different values of the hydrostatic load h can be applied (h is given by the difference between the spillway level and the selected outlet, and is maintained constant during a single test). Each opening has a cross section of 100 mm by 12 mm.

The feeding apparatus is formed by a pump, sucking water from a feeding tank; this tank is also the backwater tank at the end of the test circuit. The flow measuring group is essentially formed by a calibrated reservoir, that can be feeded, or excluded, by a two-way valve.

This constant head permeameter has a closed hydraulic circuit, so that the same permeability test can be performed by using the same, small quantity of demineralized, disaerated water (or, alternatively, leachate).

When performing normal permeability tests on geotextiles, a particular problem is given by the validity limits of Darcy-Ritter's law. In this view, an upper limit value was computed for the apparent water velocity, resulting $v_{max} = 0.035$ m/s (3)(5). This means that a minimum value of the specimen thickness H, in turn function of the hydrostatic load h, should be kept. For this research work, the minimum value of H was obtained by superimposing different layers of the same geotextile; the number of layers was ranging from 3 to 6, depending on the thickness T_g of the geotextile to be tested (3).

The coefficient of normal permeability k_n and the permittivity Ψ of the geotextile are computed by the formulas

$$k_n = \frac{V \cdot H}{A \cdot h \cdot t} = \frac{V \cdot x \cdot T_g}{A \cdot h \cdot t} \qquad (1)$$

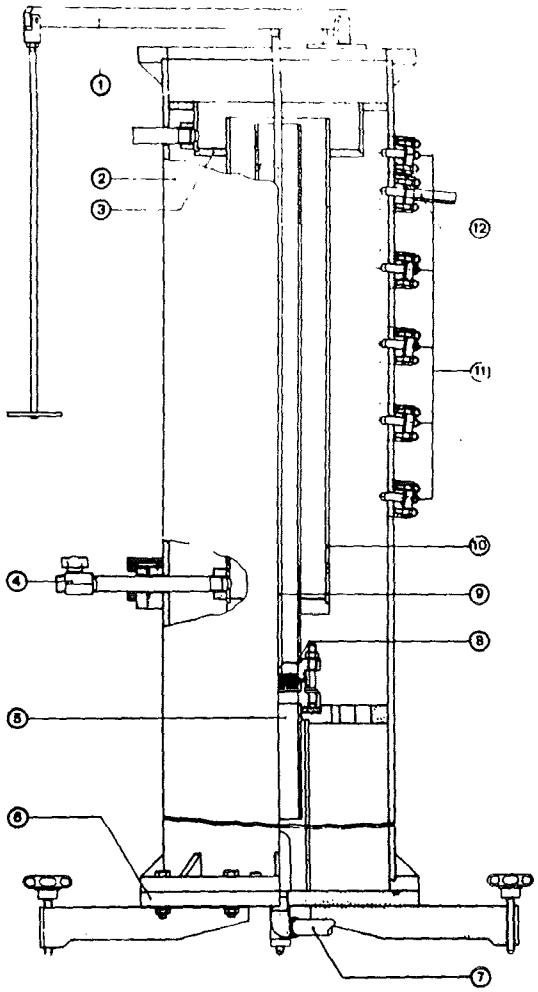
and

$$\Psi = \frac{V \cdot x}{A \cdot h \cdot t} \qquad (2)$$

where: V = calibrated volume = $1450 \cdot 10^{-6}$ m^3;
A = specimen surface = $2043 \cdot 10^{-6}$ m^2;
H = specimen thickness (m);
x = number of geotextile layers forming a specimen;
h = hydrostatic load (m);
t = required time to fill calibrated volume V during a single test (s).

The permeameter, as described above, allows for tests in the absence of normal stresses on the specimen surface. In order to perform permittivity tests under compressive normal stresses, the permeameter was modified as shown in fig. 3.

A synthetic porous stone (formed by monogranular, siliceous sand bonded by small quantities of polyester resin, and having permeability higher than tested geotextiles) is placed on the specimen surface, and overlaid by a PVC perforated element (bearing a spherical seat for the loading steel rod). The static load is applied by a lever system; values of the average normal stress σ up to 0.2 MPa were adopted in this testing program. The loading device is completed by a dial gauge, allowing to measure the variations of T_g versus the applied normal stress σ.



LEGEND

1. Loading system
2. External cylinder
3. Overflow system
4. Feed valve
5. Specimen support system
6. Base flange
7. Outflow valve
8. Geotextile specimen
9. Loading rod
10. Constant level cylinder
11. Outlets
12. To the flowrate measurement

Fig. 3. Scheme of the permeameter with the loading device, for permittivity tests in the presence of normal stresses (from (2)).

2.2. Characteristics of Tested Geotextiles

The complete list of tested geotextiles is reported in table 1, together with the most significant physical properties. For some materials, different thicknesses and masses per unit area were available, so that for most of them water and/or leachate was used as fluid for permittivity tests (see numbers or letters in table 1).

The list includes only nonwoven geotextiles: mostly monofilament, both needled and thermobonded, and some staple-type Italian products. The mass per unit area is ranging from 240 to 370 g/m^2, and the nominal thickness (as measured under σ = 2 kPa) from 2.1 to 9.2 mm.

Where possible, data about the opening size O_{95} were inserted into the table, as drawn from literature (4): these data are referred to the hydrodynamic filtration procedure, that is commonly adopted in Italy, France, and Belgium.

Table 1 - General characteristics and type of permittivity test developed on the considered geotextiles.

Commercial name and manufacturer	Polymer and process	Type	μ_2 (g/m^2)	Tg (mm)	O_{95} (μm)	Permittivity test	
						Water	Leachate
BIDIM - Rhône-Poulenc(F)	PES NP CF	U 34	290	2.9	120	2	A
TREVIRA - Hoechst (D)	PES NP CF	11/300	300	2.8	120	1	
DRENOTEX - Politex (I)	PES NP SF	TFP	320	1.7	82	4	
SINTER - Ovatex (I)	PES NP SF	300	300	3.0	52	3	
DREFON - Man.Fontana (I)	PES NP SF	S-45	200	2.2	48		B
TECNOFELT - Tecnofibra (I)	PES NP SF	FAG	300	3.4	-		C
GEODREN - Edilfloor (I)	PES NP SF	PE/S	300	2.8	-		D
SODOSPUN - Sodoca (F)	PP NP CF	AS 420	320	3.1	-	5	
POLYFELT - Chemie Linz (A)	PP NP CF	TS 750	370	3.0	125	6	E
DREFON - Man.Fontana (I)	PP NP SF	SIA-200	200	2.9	-		F
		SIA-400	400	5.2	-		I
STRATUM - Viganò Pav. (I)	PP NF SF		450	5.4	-		L
TYPAR - Du Pont (LUX)	PP TB CF	3807	280	0.7	37	8	H
TERRAM - I.C.I. (UK)	PP-PE TB CF	1000	140	0.8	100		G
		2000	240	1.1	80	7	

PES : polyester PP : polypropylene PE : polyethilene
NP : needle-punched TB : thermo-bonded
CF : continuous-filament SF : staple-fibre

2.3. Testing with Water

Permittivity tests on geotextiles have been concerned by several investigations, at least for $\sigma = 0$, and tentative standard methods were proposed in the most recent years (see e.g. (5)). Therefore, without entering in details of testing procedure, the following main points were kept present in this research work:

a) all tests were carried out by using water that had been previously demineralized (by treatment with cationic and anionic resins), and disaerated by a vacuum pump; special care was used to eliminate air bubbles from the whole hydraulic circuit;

b) for $\sigma = 0$ tests, in order to fulfill the aforementioned requirements for the validity of Darcy's formula ($v_{max} = 0.035$ m/s), the specimen thickness was obtained by superimposition of 3 geotextile layers;

c) for tests under normal stresses $\sigma \neq 0$, the same criterion required to double the number of layers, so to neglect the head loss due to the presence of porous stones in adjacency to the specimen;

d) the water temperature was maintained approximately constant (about 20°C) for all tests;

e) for $\sigma = 0$ tests, the time required to attain a steady state flow through the specimen was only 10 minutes;

f) for $\sigma \neq 0$ tests, a complete stabilization was needed both for water flow and for the specimen compression under the datum load increment; therefore, times of 30 minutes about were necessary, and some tests were pursued for 24 hours or more (the program of loading steps was 0.002 - 0.026 - 0.05 - 0.075 - 0.100 - 0.125 - 0.150 - 0.175 - 0.200 MPa).

The results of tests on 8 different geotextiles are shown in fig. 4, where both the coefficient of normal permeability k_n and the permittivity Ψ are reported versus the normal stress σ. For each testing condition, the values of k_n and Ψ were obtained as mean values of 3 successive measurements. Concerning these results, the following observations can be made:

a) in the absence of normal stresses ($\sigma = 0$), the values of k_n are ranging from $1.4 \cdot 10^{-4}$m/s to $6.28 \cdot 10^{-3}$m/s; accordingly, the values of Ψ vary from 0.2 to 2.22 s^{-1}; these values are consistent with the range of values found by other researchers (1);

b) for all geotextiles, k_n and Ψ decrease with increasing σ, but the rate of decrease is different from a geotextile to another, depending on nature of fibers and bonding processes;

c) among monofilament, needled products, and for comparable values of mass per unit area μ and thickness T_g, PP geotextiles show values of k_n lower than PES ones; this fact is explained by the lower density of PP filaments and, consequently, by the lower porosity of PP geotextiles; anyway, the difference tends to be reduced with increasing σ;

d) as it's well known, thermobonded geotextiles show lower values of k_n and Ψ, with only a slight decrease with increasing σ, as a consequence of lower porosity and higher cohesion;

e) staple - fibre products show intermediate values of k_n and Ψ between monofilament needled and thermobonded ones; they also show a more rapid decrease of k_n with increasing σ.

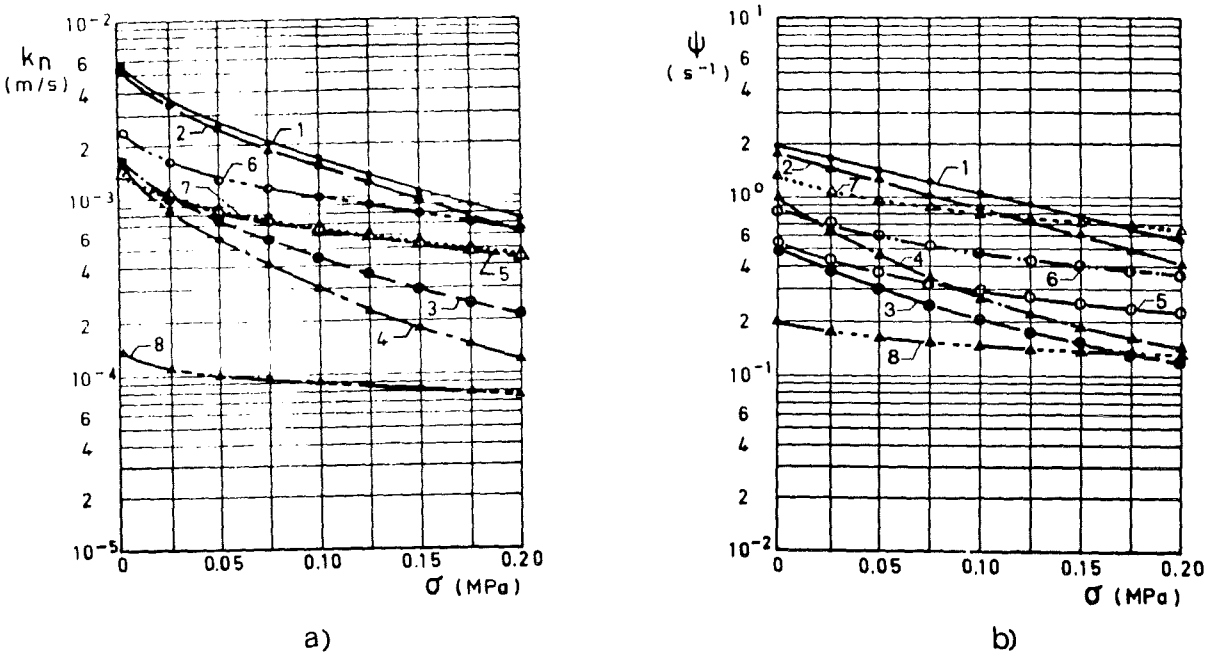


Fig. 4. Results of permittivity tests to water: a) k_n vs. σ; b) Ψ vs. σ (the numbers refer to geotextiles listed in table 1).

2.4. Testing with Leachate

In order to investigate the permittivity of geotextiles to contaminant fluids, a second series of tests was programmed and carried out. In general, the geotextiles were not the same of the former set of permittivity tests to water, and a real comparison was possible only for BIDIM U34, POLYFELT TS/750 and TYPAR 3807 (see table 1).

The contaminant fluid was sampled from the outlet of leachate collecting system of a sanitary landfill in Guanzate (Province of Como, Northern Italy). As permittivity tests were carried out in 2 groups at 2 different times, 2 separate samples had to be taken: sample A was used for testing BIDIM and POLYFELT, while sample B was used for all the other geotextiles (see table 1). The chemico-physical characterization of the two leachate samples is reported in table 2.

In defect of previous experiences with permittivity tests to leachate, the testing procedures were modified in the course of the research work. Owing to the (feared) high clogging capacity of leachate, testing specimens formed by a single geotextile layer were adopted (anyway, the major obstacle to seepage was proved to be due not to the clogging of pores into the geotextile, but to the deposition of suspended material on the geotextile surface).

As told above, the testing apparatus has a closed hydraulic circuit, and the same pure water could be utilized indefinitely for different materials (at least in theory). Of course, this procedure is not possible with leachate, because of deposition of suspended solid material on geotextile specimens, and every geotextile should be tested with "fresh" fluid; on the other hand, the fluid volume that is needed to

Table 2. Composition of leachates used for tests (analyses by Institute of Sanitary Engineering, Politecnico di Milano).

Parameter (°)	Leachate 'A'	Leachate 'B'
pH	6.3	6.0
Chemical oxygen demand (COD)	28060	38520
Biological oxygen demand (BOD)	10400	3000
Alkalinity ($CaCO_3$)	4250	5125
Volatile acids	435	1575
Organic nitrogen (N NH_4)	554	60
Ammonia (NH_3)	1203	1293
Chlorine (Cl)	1868	2231
Sulfate (SO_4)	1860	1600
Calcium (Ca)	995	175
Magnesium (Mg)	827	1469
Iron (Fe)	330	47
Manganese (Mn)	27	42
Zinc (Zn)	5	7
Sodium (Na)	1300	1400
Potassium (K)	1200	1200

(°) All values in milligrams per liter, except pH

fill the whole permeameter is about 100 liters, and only limited quantities of leachate were available for this research. Therefore, a compromise solution was adopted, consisting in changing the fluid every two geotextiles, so that only five materials could be tested with fresh leachate, while the remaining ones were tested with filtered leachate.

As a consequence:
- the results obtained for the former group (BIDIM U34, GEODREN PE/S, TECNOFELT FAG, DREFON S1A-200, and TERRAM 1000) can be considered valid and comparable each to other;
- for the latter group of geotextiles (POLYFELT TS/750, STRATUM, DREFON S1A-400, DREFON S45-200, TYPAR 3807), the measured values are only indicative, and should be considered with extreme caution (it has to be remembered that, in field applications, geotextiles do not come into contact with pre-filtered leachate).

Permittivity tests to leachate were performed only for $\sigma = 0$ conditions. For most materials, tests were interrupted after 50 ÷ 100 minutes; in two cases (staple-fibre geotextiles, having high thickness and mass per unit area, indicated with letters I and L in table 1), the duration of the test was prolonged to about 3 hours. The results of all tests, expressed by k_n vs. time and Ψ vs. time plots, are reported in figs. 5 and 6.

On the basis of all results, the following considerations can be made:

a) all tested materials show a marked decrease of the coefficient of permeability k_n (and, accordingly, of the permittivity Ψ) with the time t elapsed from start-

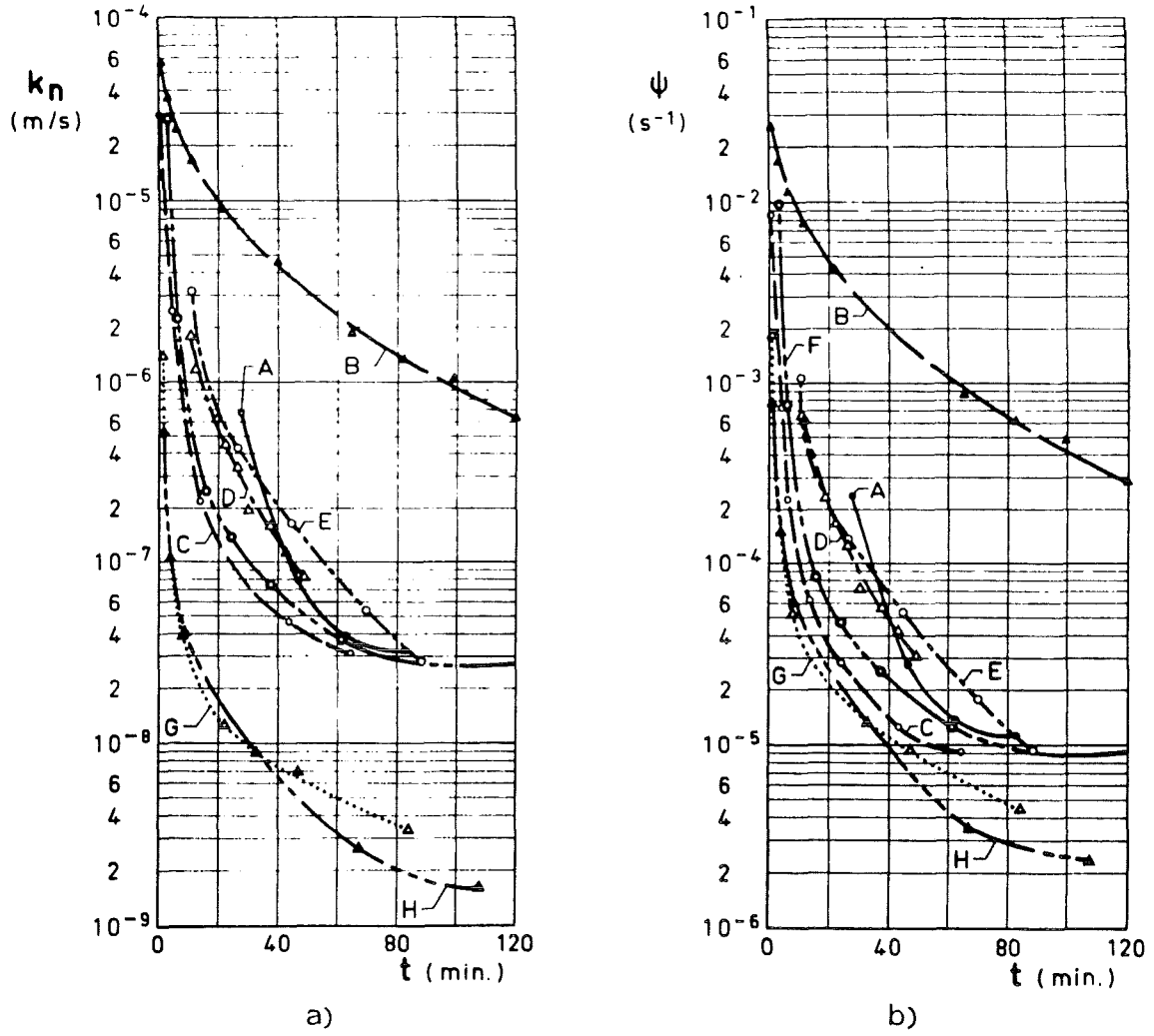


Fig. 5. Results of permittivity tests to leachate on 8 materials: a) k_n vs. time; b) Ψ vs. time (the letters refer to geotextiles listed in table 1).

ing the test (figs. 5 and 6); the ratio between initial and final permeability is about 100 times; 1 ÷ 2 hours are generally required for a rough stabilization of the flowrate;

b) likely, the decrease has to be ascribed essentially to the deposition of solid substances, that were suspended into leachates, prevalently on the geotextile surface, and partly also into the pores of the geotextile itself;

c) if only the three materials tested both to water and to leachate (i.e., BIDIM U34, POLYFELT TS/750, and TYPAR 3807) are considered, with reference to the values measured for σ = 0 (fig. 4) and for t = 80 ÷ 120 minutes (fig. 5), it can be seen that the effect of leachate is to reduce normal permeability to less than 1 / 100,000 of the original value; anyway, the rating between different classes

Obviously, a correct design and a proper application of the geomembrane system has, in any case, a preminent importance.

Further developments of researches about leachate / geotextile interactions should include:
- analysis of retained material;
- chemical effects on the geotextile permittivity;
- study of the behaviour of the whole geotextile filter - geomembrane system, in th presence of leachate.

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