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by

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ABSTRACT

The past performance and usage of various seepage measures beneath earth dams founded on deep pervious foundations are considered important precedents. This is especially true when selecting the most proper seepage measure in the preliminary design stages of a new dam.

In this thesis the author compiles and correlates performance data on various measures used to reduce and/or control seepage from over 100 dams throughout the world situated on pervious soil foundations. A comprehensive discussion on the applicability and performance record of slurry trench cutoffs, concrete diaphragm walls, upstream impervious blankets, grout curtains and relief wells is summarized. The information contained within is presented as an aid to the dam designer in that specifics of each seepage measure are highlighted.

Also included is an evaluation of acceptable and unacceptable seepage measurements throughout the world. From these measurements, guidelines for dam safety review are suggested.

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1.0 INTRODUCTION

In the design of any water retaining structure, foremost consideration must be given to methods that effectively reduce and control the quantity of seepage. Forces and pressures exerted by such seepage can pose serious threats to the safe performance of the structure. The selection of the most suitable measure, however, is an exceedingly difficult task particularly when the dam is to be founded on deep alluvium. This material is extremely pervious, heterogeneous and susceptible to failure by piping.

As evident by the number of references at the rear of this text, there is an abundance of information available pertaining to both seepage reduction and control measures. The majority of these publications relate to either individual case histories or theoretical considerations. However, there is little guidance for the designer on which to base his initial selection of a seepage measure for a given dam site.

It was therefore the intention of this thesis to compile and correlate data on various measures used to reduce and control seepage beneath earth and rockfill dams situated on pervious soil foundations.

It was elected to compile data from case histories over performing a parametric study using analytical techniques. It was thought that using past experiences would provide better insight into the applicability of each measure and relevant parameters would become evident.

The seepage reduction measures discussed include the following;

i) Slurry Trench Cutoffs

ii) Concrete Diaphragm Walls

iii) Upstream Impervious Blankets, and

iv) Grout Curtains

Relief wells are the only seepage control measure discussed in detail.

1.1 Objectives

The major objectives of this thesis are as follows:

1) Assess acceptable and unacceptable performance of dams on pervious foundations,

2) Compile available worldwide data on the location and insitu conditions where various seepage reduction measures and relief wells have been used,

3) Delineate the applicability of each seepage measure,

4) Evaluate the performance record of each measure studied, and

5) Determine the most suitable site conditions to use each of the respective measures

It is not the intention of this thesis to review at length various construction methods, specifications or stability considerations.

1.2 Scope of Work

Included in Chapter 2.0 are, an evaluation of acceptable and unacceptable seepage measurements and a review of available state-of-the-art publications pertaining to seepage reduction measures and relief wells.

Chapter 3 presents the case histories studied and provides a description of the various seepage measures investigated.

Results of the data analyzed from Chapters 2.0 and 3.0 are reported in Chapter 4.0.

Conclusions drawn from this study are presented in Chapter 5.0 followed by a Bibliography, Figures and two Appendices, respectively.

Appendix A contains a listing of all case histories used for the determination of acceptable and unacceptable seepage quantities.

Data from each case history which involved slurry trench cutoffs, intersecting pile walls, panel walls, overlapped pile walls, upstream blankets, grout curtains or relief wells are summarized in tabular form in Appendix B.

2.0 BACKGROUND

2.1 Acceptable and Unacceptable Seepage

The permissible or acceptable values of seepage beneath a dam are dependent on the function of the reservoir, the cost attributed to the stored water, the form of land use downstream and the volume of inflow into the reservoir. Mitchell (1983) states that leakage from storage dams typically are in the order of 0.1% of the stream flow. On the other hand for flood control dams, large leakage limits can be tolerated as there are no economic consequences. However, in either case, leakage is only acceptable provided that it is controlled and does not result in any adverse conditions such as piping or other instabilities.

Hoff (1970) states that in Norway, it is considered unacceptable when the quantity of seepage beneath a dam is greater than 0.1 cubic metres per second (cumecs). He recommends that inspection of the dam be increased to every other week rather than twice a year.

2.2 Seepage Reduction Measures

Seepage reduction measures are used to effectively reduce the quantity of flow beneath a dam. This objective is achieved either by increasing the flow path or by providing a vertical impermeable barrier beneath the dam.

The efficiency of a reduction measure can be defined in two different ways:

i) As the ratio between head loss due to the presence of the reduction measure and the overall hydraulic head across the dam.

ii) As the ratio between the quantity of seepage with a reduction measure in place and the quantity of seepage which would occur if the seepage measure was not present.

Ambraseys (1963) presents a more detailed evaluation of efficiencies. However, for purposes of this thesis the above definitions have been adopted since these are the more common published values.

The majority of the information presented in this section was extracted from the following state-of-the-art publications, Cambefort (1967), Casagrande (1961), Cedergren (1977), Marsal and Resendiz (1971), Sherard et al. (1963) and Wilson and Squier (1969). Many of the thoughts presented are consistent from one author to another; therefore, additional references are only included for authors not mentioned above.

The four seepage reduction measures will be discussed below in ascending order of their depth limitations. The four measures are as follows:

a) Slurry Trench Cutoffs,

b) Concrete Diaphragm Walls, including

  (i) Intersecting Pile Walls

  (ii) Panel Walls

  (iii) Overlapped Pile Walls

c) Upstream Impervious Blankets, and

d) Grou ains

2.2.1 Slurry Trench Cutoffs

A slurry trench cutoff as defined by Jones (1967) and for the purposes of this thesis, is a cutoff trench excavated in the wet, supported using a bentonite slurry and later backfilled with a blended soil while the slurry is still in the trench. The typical construction sequence is shown on Figure 1. Excavation of the trench can be carried out using either a dragline, clam shells, backhoe or trenching equipment.

Typically trenches are 1 to 3 metres (m) wide and can be excavated to a depth of 30 m. The limiting factors to the depth are generally economics and the capabilities of available construction equipment. The width of the trench is governed by the anticipated hydraulic gradient and the gradation of the foundation material. However, as Jones (1967) suggests the final width of the cutoff is determined by the width of the excavation bucket.

Slurry trench cutoffs are used where the depth and presence of the water table preclude the excavation and placement of a standard earth backfilled cutoff.

Mitchell (1983) states that slurry trench cutoffs are best suited for an easily excavated material such as an alluvium or coarse grained soil. Preferably the excavated material should be predominantly of gravel sizes due to the difficulties involved with separating the sand sizes from the slurry during construction.

Since the trench is later backfilled with a blended soil, sources of adequate borrow should be considered in the preliminary design stages. Typical published gradations of such soil mixtures are shown on Figure 2.

Placement of the cutoff may either be beneath the centreline of the dam or in the vicinity of the upstream toe as illustrated by Figure 3. Jones (1967) indicates that the exact location is dependent on the dam's profile, foundation conditions, and the construction sequence and schedule. However, the matter of location is subject to two schools of thought. The centreline location gives better protection, along the contact between the core and the cutoff, against high water pressures. Jones (1967) also considers the cutoff to be least expensive if excavated at this location. However, Jones (1967) and U.S. Department of the Interior, Bureau of Reclamation (Burec, 1977) state that the upstream location has the advantages of possible future maintenance, increased stability and provides the capability of staged construction.

Slurry walls must withstand high levels of deformation without cracking or failing. Millet and Perez (1981) reported no cracking problems would be anticipated for a soil-bentonite backfilled trench provided the material was not coarsely graded and that the slump of the backfill material was between 100 to 150 millimetres (mm). Differential settlement can also be compensated for by providing a transition zone at the top of the trench.

The performance of slurry trench cutoffs has been good. They have the advantage of being flexible, compressible, inexpensive and are able to withstand high hydraulic gradients. Jones (1967) reported that lab tests carried out for the Wanapun project resulted in gradients of 35 before piping occurred into open gravels.

However, segregation of the slurry is possible primarily due to the presence of large boulders at the base of the excavation or poor quality control. Segregation can lead to settlements and/or zones of high permeability which will subsequently lower the efficiency of the cutoff. Other disadvantages of slurry trench cutoffs include loss of slurry material into open gravels, poor efficiency in the presence of salt and no inspection is possible during or after construction.

2.2.2 Concrete Diaphragm Walls

For purposes of this thesis concrete diaphragm walls have been separated into the following three categories:

(i) Concrete Intersecting Pile Walls,

(ii) Concrete Panel Walls, and

(iii) Concrete Overlapped Pile Walls.

The common factor to each is that the excavation or borehole is supported with a bentonite slurry during construction and later backfilled with reinforcement and a cement-bentonite mixture. The main differences between each wall are their construction techniques and subsequent plan views.

Figures 4, 5 and 6 demonstrate the typical construction sequence for an intersecting pile, panel and overlapped pile wall, respectively.

Intersecting piles typically are 0.6 m in diameter and are extended to depths of approximately 40 m. Whereas panel walls and overlapped pile walls may be extended to depths of 50 m and greater than 100 m, respectively. The diameter of the overlapped piles is also in the order of 0.6 m.

Concrete panels are generally 4 to 6 m long and about 0.6 m wide. The length of panel, as reported by Millet and Perez (1981), is governed by both stability considerations and the fact that one tremi pipe is required for every 4.6 m of panel length. In three cases throughout the world; Manicouagan 3, Obra and Tenughat Dams, double panel walls have been constructed. The panel walls were spaced 3 m apart and between the walls, the foundation soil was grouted. The effectiveness of this procedure will be discussed in Chapter 4.0.

Concrete diaphragm walls are best suited for granular soils comprised of silts, sands and fine gravels. The Edison Group et al. (1961), suggests no difficulties are even expected with cobble size material of 100 to 150 mm diameter.

The placement of the cutoff either at the center or upstream location is subject to the same arguments as discussed for slurry trench walls. However, the major difference between a concrete wall with its cement-bentonite backfill and a slurry cutoff is that the former is very rigid in comparison with the foundation material and core of the dam. For compressible soils, negative skin friction can result whereby increasing the compressive load and the shortening of the wall. If the foundation material settles more than the wall tension cracks may also develop within the core material. Millet and Perez (1981) indicate the higher the cement-water ratio the stronger and more rigid the wall. However, by increasing the bentonite-water ratio the wall could be more flexible at the expense of strength.

The performance of concrete diaphragm walls has been very good in ensuring a positive cutoff to depths in excess of 100 m. However, they are not without problems that must be considered in design. Such problems include:

1) High cost of the measure,

2) Excavation through boulders,

3) Loss of bentonite slurry into open gravels,

4) Rigidity of the structure,

5) Misalignment with depth, and

6) Increased compressive stresses in the concrete due to negative skin friction.

2.2.3 Upstream Impervious Blankets

Upstream blankets provide an alternative to cutoffs and in the past have typically been used when the depth of foundation material has been considered too great for the construction of a perfect cutoff. However, blankets can only be considered if sufficient amounts of local borrow material is available for the construction of the blanket.

Blankets, because of their poor efficiency, must be used in conjunction with adequate seepage control measures in order to reduce uplift pressures beneath the base of the dam.

An upstream blanket decreases both seepage quantities and exit gradients because the flow path over which head loss can occur is increased by an amount proportional to both the length and thickness of the blanket.

Therefore, the thickness and length of an upstream blanket is governed by the available head.

Burec (1977) recommends that the minimum thickness of an upstream blanket should be 1 m and then increased in thickness 1 m for every 10 to 25 m of head in order to maintain a constant gradient through the blanket.

Emmelin and Welinder (1967) and Khan and Naqvi (1970) recomended that the length of the blanket should provide a 1/15 overall gradient in the foundation. It is also recommended for maximum efficiency that the blanket cover any outcrops of pervious material within the floor of the reservoir.

For the most part, blankets are placed and compacted in the dry. However, work by Golder and Bazett (1967) has shown that material can be adequately placed by dumping through open water.

Ideally, for the use of an upstream blanket, a natural impervious blanket should exist between the constructed blanket and the underlying pervious material. The permeability of this material should preferably be less than 1E-5 metres per second (m/s) and not have a high degree of anisotropy. Lefebvre et al. (1981) showed by increasing the ratio of horizontal to vertical permeability of the foundation material, from 1 to 25, the effectiveness of an upstream blanket to reduce exit gradients decreased by 80%.

Londe (1970) states that the quantity of seepage beneath a dam can decrease from 50 to 75% in the first year of operation due to siltation behind a dam. This process may reduce seepage by any of the following manners;

i) Clog any voids that may be present in the less compacted blanket material;

ii) Compact the blanket material by downward seepage forces and/or

iii) Increase the thickness or upstream extension of the blanket, by deposition of fines.

Lane and Wohlt (1961) suggest that siltation can be enhanced if during first filling the reservoir is kept at low levels initially. The phenomenon of siltation will be discussed further in Chapter 4.0.

2.2.4 Grout Curtains

Since the development of the "tubes a manchettes" (tube with a sleeve) technique in France in the early 1960's, alluvial grouting has become common practice throughout the world. The main advantages of this injection technique are that zones can be grouted individually based on insitu conditions and any portion can be regrouted at a later date without redrilling.

Grout curtains have been commonly used to seal alluvial foundations to depths of 100 m. However, as the cutoff is advanced deeper, the chance for deviations of the drill holes increases proportionately. As a result, gaps or pervious zones may exist within the cutoff. The presence of such zones or defects has been shown by both Cedergren (1977) and Casagrande (1961) to drastically decrease the effectiveness of a grout curtain in reducing hydrostatic pressures and seepage quantities under an embankment. It is therefore of the utmost importance when installing a grout curtain to have experienced personnel on site.

Grout curtains are generally comprised of several rows of injection holes. Normally the outside holes are the shallowest and injected with a highly viscous grout. Whereas, the inner holes are the deepest and injected with a less viscous, more penetrating type of grout. The spacing between holes and rows is in the order of 2 to 3 m. The number of rows or final width of curtain is dependent on the required head loss across the curtain.

The actual grout type used is dependent on the grain size and subsequent permeability of the foundation material. If the permeability is in the order of 1E-5 m/s (sands and gravels), a cement and clay grout may be used. However, if the permeability is 1E-6 m/s (silts), chemical gel grouts would be necessary. A permeability of less than 1E-5 m/s is considered very difficult to grout.

Londe (1970) states that grout curtains have been found to successfully lower the permeability of alluvial material to permeabilities of 5E-6 to 5E-7 m/s.

Grout curtains have the advantage of reducing the compressibility of the alluvial foundation and are more flexible than a concrete diaphragm wall. However, as previously mentioned the possible loss of efficiency with depth has been questioned. The efficiency of a grout curtain with time due to scour by high hydraulic gradients has also been raised by many authors.

2.3 Relief Wells

Turnbull and Mansur (1954) state that the primary requirements of a relief well system for the control of excess pressures due to underseepage are as follows:

1. The wells should penetrate into the principal water bearing strata and be spaced sufficiently close together so as to intercept the seepage and reduce the pressure which otherwise would act beyond the wells.

2. The wells must offer little resistance to water flowing into and out of them; they must prevent infiltration of sand into the well after initial pumping; and they must resist the deteriorative action of the water and soil.

Relief wells are more suitable for deep stratified foundations, where excess pressures may exist, than other seepage control measues since they can penetrate to greater depths.

The design of relief wells was initiated by the United States Army Corps of Engineers (Corps) in conjunction with their work on dams and levees along the Mississippi River. Originally, relief wells were designed as remedial measures and were not incorporated into the design stage of a dam until 1940. The design philosophy of relief wells, even from the early work of Middlebrooks and Jervis (1947), has been that the system is very flexible and at any time additional wells could be added if required.

Middlebrooks and Jervis (1947) developed initial formulas for the design of fully penetrating relief wells based on seepage theory and field measurements. These formulas assumed artesian flow towards an infinite line of equispaced wells from an infinite line source parallel to them. The relief wells, located at the downstream toe of an impervious embankment, fully penetrate a thick pervious layer which is overlain by a thin impervious stratum. Both strata are assumed to be isotropic and homogeneous. Based on the same assumptions noted above, Middlebrooks and Jervis (1947) also developed empirical charts for the design of partly penetrating wells. These charts were based on results from hydraulic and electric analogue model studies.

The design methods discussed above for such generalized conditions are rarely applicable to field conditions. For example, head losses due to inflow and outflow from the well and the possibility of seepage through the upper impervious layer, either upstream or downstream of the dam, were not considered. However, through discussions of this initial work, the design methods were modified so as to account for conditions described above.

Turnbull and Mansur (1954) carried out a number of model studies which represented various foundation conditions, seepage entrances and seepage exists for various spacings and penetrations of relief wells. Bennett (1954) subsequently developed a set of empirical equations for the design of relief wells, based on the work of Turnbull and Mansur (1954).

Turnbull and Mansur (1961) describe the compilation of the above work and present the design procedure adopted by the Corps. An example of this design procedure is presented by Thorfinnson (1960). Subsequent modification to this procedure has been carried out to account for a finite line of wells.

Relief wells are generally spaced 10 to 30 m apart and discharge into either an open ditch or to a sump via header pipes. The diameter of a relief well is dependent on anticipated seepage quantities. However, Sherard et. al. (1963) and many other authors suggest a minimum diameter of 152 mm. Middlebrooks and Jervis (1947) suggest that the diameter and spacing of relief wells are not as effective on their performance as the degree of penetration.

Middlebrooks and Jervis (1947) states a minimum penetration into the principal water bearing strata should be 25%. Sherard et al. (1963) suggests if the permeability increases with depth, a partly penetrating well has almost no effect.

Two disadvantages of relief wells are that they increase the seepage beneath a dam by shortening the flow path and that they require periodic maintenance. Turnbull and Mansur (1961) states that seepage quantities may be approximately 20 to 40% higher than without the installation of relief wells. Therefore, it is necessary to provide adequate measures downstream of the dam to cope with increased flows. However, these flows have been found to decrease over the years due to silting of the filter surrounding the well and/or encrustation of the well due to bacterial growth or formation of precipitates. Cedergren (1977) reports a study carried out by the Corps in 1972 indicated seepage flows out of the wells decreased by 33% in 15 years. Therefore, in the design of a relief well it is necessary to ensure that they are accessible for future maintenance and that the filter and pipe can be surged at a later date.

3.0 CASE HISTORIES

During the course of this study a detailed review of published literature was carried out for purposes of developing a chronological account of items pertinent to design, construction, setting and performance of both seepage reduction and control measures. The main sources of data were reports published in the International Congress on Large Dams (ICOLD) and various soil mechanics conferences throughout the world.

Initially, all available case histories of earth and rockfill dams on pervious granular foundations were included in the study. At this stage no restriction was imposed on the type of seepage measure to be studied. This initial work formed the basis of a statistical evaluation of acceptable and unacceptable seepages below earth and rockfill dams.

The second part of the literature review was restricted to only case histories involving slurry trenches, concrete diaphragm walls, upstream blankets and grout curtains beneath earth and rockfll dams on pervious granular material. Some of these case histories were incorporated into the first stage of the literature review if quantities of seepage were documented within the respective articles.

A review of case histories incorporating relief wells in their design or as subsequent remedial measures is presented in the third portion of this chapter.

It is felt that all available major case histories are discussed herein and that the necessary pertinent data to form conclusions have been considered.

3.1 Acceptable and Unacceptable Seepage

Throughout the course of the literature review it became apparent that common values of seepage beneath dams were considered either acceptable or unacceptable. These values of course are dependent on the type of structure, the use of the reservoir, the extent of inhabitation downstream and the owners. However, since the majority of dams reviewed were part of hydroelectric schemes the economic value of the water from these reservoirs would be comparable. Therefore, it was believed that an average value of seepage could be obtained from the case histories which would represent acceptable performance of a hydroelectric earth or rockfill dam.

The definition of acceptable and unacceptable quantities of seepage beneath a dam is very difficult to address. However, for the purpose of the literature review the division was based on the opinion of the respective authors and were defined as follows:

Acceptable: A quantity of seepage flow which did not pose any threat to the embankment and thus no remedial measures were considered necessary. Inspection and monitoring programmes would be carried out on a routine basis.

Unacceptable: A quantity of seepage flow which raised concern for the safety of the embankment and/or ancillary structures. Inspection and monitoring programmes would be carried out on a non-routine basis.

Cases that were considered unacceptable either failed, were abandoned or substantial remedial measures were taken to reduce seepage flows.

The actual quantity of seepage beneath the dam was recorded in two manners. Firstly, the total quantity of seepage, Q, was tabulated for each dam. However, as one would expect the quantity of seepage beneath a long dam would generally be much greater than for a short dam. Therefore, in order to normalize the data, the total quantity of seepage was divided by the respective crest length of each dam. These numbers represent the total quantity of seepage per lineal metre of dam.

The above data are presented in Table A-1, in Appendix A.

A discussion and analysis of these data are presented in Section 4.1.

3.2 Seepage Reduction Measures

The following section is a compilation of data from case histories published in the technical literature. The objective of this section is to provide the reader with a brief review of the dams which incorporate the four seepage reduction measures mentioned earlier in text.

To ensure consistency throughout the course of this study, data from each case history were summarized in tabular form. Generally, the headings that appear on the tables are the same for each seepage reduction measure. It is believed all available pertinent data is presented in such a manner that comparisons between measures can easily be drawn.

The information in the tables are presented under the five following general headings:

1) Dam - Within this portion of the table pertinent data such as the name and location of the dam, date of construction, available hydraulic head, and nominal hydraulic gradient are presented.

2) Subsurface Conditions - This section provides a general description of the foundation material including depths, soil type and permeability.

3) Seepage Reduction Measure Data - Presented under this heading is a brief description of the respective seepage reduction measures. These data are presented in order that comparisons could be made with the data presented in Chapter 2.0.

4) Associate Measure - Any seepage control and/or reduction measure used in conjunction

with the seepage measure being discussed is included under this heading.

5) Efficiency and Performance - The performance record and the efficiency of the seepage measure, if available, is provided.

The case histories are tabulated in Appendix B. A legend for the tables is provided in Table B-8, also in Appendix B.

3.2.1 Slurry Trench Cutoffs

In all, 17 case histories were reviewed. Information from these case histories are presented in Table B 2 in Appendix B. For convenience, ranges and averages have been included in Table 1.

3.2.2 Concrete Diaphragm Walls

The three types of concrete diaphragm walls have been tabulated separately because of their interent differences as discussed in Chapter 2.0.

Of the 26 concrete diaphragm walls presented in Appendix B, 6 are intersecting pile walls, 14 are panel walls and

TABLE 1

SLURRY TRENCH CASE HISTORIES - SUMMARY*

	DAM			SUBSURFACE CONDITIONS		S.R.M. DATA				ASSOC. MEASURE	EFFICIENCY & PERFORMANCE
	DATE	HEAD (m)	GRADIENT	DEPTH (m)	PERM (m/s)	DEPTH (m)	WIDTH (m)	LOCATION	PERM (m/s)		
Range											
Low	1952	4.9	.07	7	1 E-7	4.5	1.3	7CL	1 E-9	1 none 3 G.C. 3 u/s B	60%
High	1980	72.4	.26	350	1 E-2	31	3.3	9 u/s	1 E-6	4 R.W.	89%
Number	17	17	11	12	12	17	17	16	5	11	4
Average	1967	22.2	.12	82	1 E-3	18.7	2.4	u/s	2E-7	-	73% H
Standard Deviation	6.7	16.6	.06	112	8E-3	9.0	.72	-	4E-7	-	13

TOTAL NUMBER OF DAMS

17

*For Description of Table, See Table B-8, in Appendix B.

6 are overlapped pile walls. The case histories are tabulated separately in Tables B-3, B-4 and B-5, respectively in Appendix B.

This information is also summarized in Tables 2, 3 and 4, respectively.

3.2.3 Upstream Impervious Blankets

Table 5 summarizes data obtained from 21 case histories involving upstream blankets on pervious granular material. Individual case histories are presented in Table B-6 in Appendix B.

3.2.4 Grout Curtains

The 18 case histories reviewed are summarized in Table 6. More detailed information on each case history can be found in Table B-7 in Appendix B.

3.3 Relief Wells

The following section is a compilation of data from case histories in the technical literature on relief wells.

As in the case of seepage reduction measures, all data were tabulated to ensure consistency and thoroughness.

TABLE 2

INTERSECTING PILE WALL CASE HISTORIES - SUMMARY*

	DAM			SUBSURFACE CONDITIONS		S.R.M. DATA				ASSOC. MEASURE	EFFICIENCY & PERFORMANCE
	DATE	HEAD (m)	GRADIENT	DEPTH (m)	PERM (m/s)	DEPTH (m)	WIDTH (m)	LOCATION	PERM (m/s)		
Range											
Low	1955	12	.08	10	1 E-6	22	.5	3 CL	1 E-7	1 u/s B	60%
High	1971	30	.27	40	1 E-2	41	.9	3 u/s	-	4 none	-
Number	6	6	6	6	3	6	6	6	1	5	1
Average	1962	20	.17	27.2	3E-3	31	.65	-	1 E-7		60% Q
Standard Deviation	6.0	6.4	.08	13	5E-3	8.7	.13	-	-	-	-

TOTAL NUMBER OF DAMS
6

* For Description of Table, See Table B-8, in Appendix B.

TABLE 3

PANEL WALL CASE HISTORIES - SUMMARY*

	DAM			SUBSURFACE CONDITIONS		S.R.M. DATA				ASSOC. MEASURE	EFFICIENCY & PERFORMANCE
	DATE	HEAD (m)	GRADIENT	DEPTH (m)	PERM (m/s)	DEPTH (m)	WIDTH (m)	LOCATION	PERM (m/s)		
Range										1 P.C. 1 G.C.	
Low	1959	20	.12	5	1 E-6	18	.4	6 u/s	1 E-8	3 R.W. 3 u/s B	60%
High	1980	92	.27	140	1 E-2	77	.76	8 CL	1 E-8	6 none	100%
Number	14	13	13	14	10	14	12	14	2	12	6
Average	1968	38	.19	53	1.1E-3	46	.63	CL	1 E-8		87% Q 85% H
Standard Deviation	6.3	23	.05	37	2.7E-3	19	.12	-	0		15 14

TOTAL NUMBER OF DAMS
14

* For Description of the Table, See Table B-8, in Appendix B.

TABLE 4

OVERLAPPED PILE WALL CASE HISTORIES - SUMMARY*

	DAM			SUBSURFACE CONDITIONS		S.R.M. DATA				ASSOC. MEASURE	EFFICIENCY & PERFORMANCE
	DATE	HEAD (m)	GRADIENT	DEPTH (m)	PERM (m/s)	DEPTH (m)	WIDTH (m)	LOCATION	PERM (m/s)		
Range											
Low	1959	24	.12	27	1 E-5	27	.5	2 u/s		1 p.C. 1 drain	82
High	1973	92	.34	122	1 E-2	122	.66	4 CL		1 none	91
Number	6	6	5	6	4	6	6	6	0	3	2
Average	1965	54	.22	73	2E-3	67	.59	CL	-		90% Q 87% H
Standard Deviation	5.8	24	.10	37	4.4E-3	36	.05	-			- 5

TOTAL NUMBER OF DAMS

6

* For Description of Table, See Table B-8, in Appendix B.

TABLE 5

UPSTREAM BLANKET CASE HISTORIES - SUMMARY*

	DAM			SUBSURFACE CONDITIONS		S.R.M. DATA						ASSOC. MEASURE	EFFICIENCY & PER-FORMANCE
						THICKNESS (m)			LENGTH (m)				
	DATE	HEAD (m)	GRADIENT	DEPTH (m)	PERM (m/s)	min	max	X_t	L	X_L	PERM (m/s)		
Range													
Low	1937	12	.03	4.6	1 E-7	.5	1	3.3	76	.6	1 E-9	P.C. 4 G.C. 4 none 5 R.W. 9	50%
High	1978	138	.12	210	1 E-3	.3	12	92	1432	32	1 E-5		65%
Number	21	21	17	19	12	10	15	16	18	18	3	20	4
Average	1961	36	.06	61	2.4E-4	1.5	3.8	19	349	12	3E-6	-	58%Q 56%H
Standard Deviation	9.4	35	.03	58	4E-4	.9	2.9	20	319	8	6E-6	-	- 8

TOTAL NUMBER OF DAMS
21

*For Description of Table, See Table B-8, in Appendix B.

TABLE 6

GROUT CURTAIN CASE HISTORIES - SUMMARY*

	DAM			SUBSURFACE CONDITIONS		S.R.M. DATA				ASSOC. MEASURE	EFFICIENCY & PERFORMANCE
	DATE	HEAD (m)	GRADIENT	DEPTH (m)	PERM (m/s)	DEPTH (m)	WIDTH (m)	LOCATION	PERM (m/s)		
Range											
Low	1953	7	.07	5	1 E-6	7	2.4	8 u/s	1 E-7	3 Gallery 3 u/s B	59%
High	1977	122.5	.28	225	1 E-2	225	38	10 CL	1 E-4	5 none 7 R.W.	98%
Number	18	18	17	18	14	17	15	18	12	18	6
Average	1965	49	.16	69	1.3E-3	75	15	CL	8E-6	-	80%Q 82%H
Standard Deviation	5.6	35	.06	58	3E-3	59	11	-	3E-5	-	- 13

TOTAL NUMBER OF DAMS
18

* For Description of Table - See Table B-8, in Appendix B.

The available data presented in the tables were chosen so that the design of each relief well system could be assessed individually, with the design methods discussed in Chapter 2.0.

The 17 case histories reviewed are presented in Table B-9 in Appendix B. For convenience, these data are summarized in Table 7.

TABLE 7

RELIEF WELL CASE HISTORIES - SUMMARY*

	DAM		SUBSURFACE CONDITIONS		WELL DATA				ASSOC.	
	HEAD (m)	GRADIENT	DEPTH (m)	PERMEABILITY (m/s)	SPACING (m)	PENETRATION (%)	RADIUS (m)	NUMBER	MEASURE	PERFORMANCE
Range										
Low	8	.03	18	1 E-7	10	30	.007	6	1 none 1 G.C. 1 S.T. 2 C.P. 10 u/s B	10 Good
High	138	.2	183	1 E-3	100	100	.1	54		10 Poor
Number	17	15	14	10	14	14	14	11	16	17
Average	29	.09	58	3.9E-4	34	84	.044	20	-	-
Standard Deviation	30	.05	49	4.8E-4	23	24	.023	15.4	-	-

TOTAL NUMBER OF DAMS

17

* For Description of Table, See Table B-8, in Appendix B.

4.0 ANALYSES OF CASE HISTORIES

The following chapter summarizes and analyses data compiled in Chapters 2.0 and 3.0.

4.1 Acceptable and Unaccepatable Seepage

The purpose of compiling these data was to establish a quantity of seepage beneath a dam which would be a measure of both acceptable performance and the effectiveness of a specific seepage measure. Therefore, to best determine this value, a statistical analysis of the data was considered necessary.

Statistics are a powerful tool when analysing data, in that the final answer obtained is to an extent a measure of what you want from your information. It is therefore, of utmost importance to chose the correct method of analysis and to understand the meaning of the values derived from this analysis.

The major concern of the author in analyzing the data presented in Appendix A, was that orders of magnitude separated the two extreme values of seepage quantities, This, as previously discussed, is due to the fact that the assessment of what is acceptable and unacceptable are different. Generally, all the data presented are below the value of 1.0 cumecs and not less than 1E-9 cumecs. It is intuitively obvious that by simply taking the arithmetic mean of these two values the higher number would be more dominant. Therefore, the value of acceptable seepage determined in this manner would not be conservative as higher seepage values would dominate.

If however, the arithmetic mean of the logarithm of these values was taken, the lower number would be the more dominant and thus the mean value would be very conservative. This value was also not considered to be representative of the data.

The median value of the logarithms was finally chosen to be the most representative and fortunately the more convenient way to evaluate this information. This value is also thought to be conservative when evaluating acceptable seepage quantities beneath an embankment.

The acceptable and unacceptable values of seepage are plotted separately on both the histograms and cumulative frequency distribution diagrams, presented on Figures 7, 8, 9 and 10.

It is apparent from the shape of the histograms on Figures 7 and 9, that the distribution of values of acceptable and unacceptable seepage are close to the same. This would indicate that the median values of acceptable and unacceptable seepages are the same. However, by examining the gap between the respective cumulative frequency distributions on Figure 8 and 10, it is evident that the two distributions are separable and that two individual median values can be obtained with confidence. It is believed the concentration of points around the median represent different opinions of what is acceptable or not.

Dams that perform unacceptably are more often documented than those that perform acceptably. Therefore, it is believed that the data is not totally representative of all hydroelectric dams. If more data were available it is believed that the acceptable seepage distributions would shift to the left, or in other words the median values would be less. Therefore, to properly use these data it is recommended that values less than 0.006 cumecs or 1 E-6.0 cumecs/ ln m of dam be considered as acceptable. If values of seepage are recorded greater than these, more precaution should be carried out in the monitoring program. The degree to which monitoring should be increased would be dependent upon the magnitude of seepage above these values. (See Table 8).

TABLE 8

ACCEPTABLE AND UNACCEPTABLE SEEPAGE

MEDIAN VALUES

	ACCEPTABLE		UNACCEPTABLE	
Q (cumecs)	0.029	$\leq$ 0.0006 $\geq$	0.064	Based on Available Data Recommened
Q/ ln m (cumecs/ ln m)	7.0 E-5	$\leq$ 1.0 E-6 $\geq$	1.6 E-4	Based on Available Data Recommended

The unacceptable value of Q agrees closely with the value of 0.1 cumecs which, as discussed in Section 2.1, is considered an unacceptable quantity of seepage by Norwegian standards. Presented in Table 9 is a comparison of the median values from Table 8 with values of seepage for individual continents. In reviewing the data worldwide, median values presented do not generally vary too much from continent to continent. The only location that is consistently above the world median values is the Asian continent. This however, is probably due to the lack of case histories and subsequent bias of the data. Therefore, it is concluded the median values presented in Table 8 are generally applicable as performance measures throughout the world.

Acceptable and unacceptable seepage quantities from case histories of various seepage reduction measures are presented in Table 10. The purpose of comparing these data with the median values is to determine if seepage beneath a dam can be used as a peformance indicator of a seepage measure. In reviewing the data presented in Table 10, no consistent deviations from the world median values can be observed for any of the seepage reduction measures. It is therefore concluded, acceptable and unacceptable seepage values presented in Table 8 are indicative of the effectiveness of any seepage reduction measure.

TABLE 10

ACCEPTABLE AND UNACCEPTABLE SEEPAGES
FOR VARIOUS SEEPAGE REDUCTION MEASURES

SEEPAGE REDUCTION MEASURE	SEEPAGE (cumecs)				SEEPAGE/LINEAL METER (cumecs/ln m)			
	ACCEPTABLE	NO.	UNACCEPTABLE	NO.	ACCEPTABLE	NO.	UNACCEPTABLE	NO.
Slurry Trench	0.08	2	0.2	1	1E-4	2	1.3E-4	1
Intersecting Piles	0.016	4	-		9.2E-5	3	-	
Panel Walls	0.045	7	-		9.5E-5	7	-	
Overlapped Piles	0.001	3	0.24	1	3.8E-6	3	4.7E-4	1
Blankets	0.052	14	0.069	6	8.1E-5	14	1.3E-4	6
Grout Curtain	0.018	10	0.056	5	5.6E-5	9	2.0E-4	5
World	.029	66	.064	30	7E-5	61	1.6E-4	29

Two other interesting factors which arose from a review of the data presented in Table 9 are the time, in years after construction, at which dams are subject to some form of increased seepage and the consistency of nominal hydraulic gradients from dam to dam throughout the world.

The distribution of the time of various incidents after the completion date of the dam is shown on Figure 11. The average time to unacceptable performance was 4.0 years. As expected the majority of incidents take place at first filling or in the following three years. However, what is important to note is the distinct presence of unacceptable performance 50 years after construction. This fact suggests the need for safety evaluations of older embankments.

Also presented in Table 9, are the average nominal hydraulic gradients from the case histories throughout the world. As noted on the bottom of the table, the average nominal gradient was determined to be 0.15. The two deviations from this were that, European dams generally have a steeper gradient and African dams have a shallower gradient. However, in reviewing the gradients from dams which were subject to unacceptable performance, an average gradient of 0.15 was also determined. Therefore, deviations from the mean are more representative of site conditions, available construction materials and preferred design practice rather than a measure of anticipated performance.

In reviewing the above section one must realize that the values presented are based on different judgements throughout the world and what is acceptable one place may not be at another. However, the values presented are considered valid guidelines and performance factors with regard to seepage reduction measures.

4.2 Seepage Reduction Measures

A summary of the seepage reduction measures considered in Section 3.2 is presented in Table 11. Included in the table are mean and standard deviation values taken from information in Tables 1 to 6 inclusive. The standard deviation values, the numbers in brackets, have been included so that the reader can assess at a glance the range of data.

In the course of assembling these data it became apparent that there were consistencies in the values of the depth of foundation, permeability of the foundation material and nominal hydraulic gradient for the various

TABLE 11

SEEPAGE REDUCTION MEASURES - SUMMARY

AVERAGE (STANDARD DEVIATION)

	DAM				FOUNDATION		S.R.M. DATA			PERFORMANCE	
MEASURE	NUMBER	DATE	HEAD (m)	GRADIENT	DEPTH (m)	PERM (m/s)	WIDTH OR THICKNESS (m)	DEPTH OR LENGTH (m)	PERM (m/s)	EFFICIENCY %	ACCEPT-ABLE
Slurry Trench	17	1967 (6.7)	22.2 (16.6)	.12 (.06)	82 (112)	1E-3 8E-3	2.4 (.72)	18.7 (9)	2E-7 (4E-7)	73 H (13)	1 NO 8 YES
Intersecting Piles	6	1962 (6.0)	20 (6.4)	.17 (.08)	27.2 (13)	3E-3 (5E-3)	.65 (.13)	31 (8.7)	1 E-7 -	60 Q -	5 YES
Panel Walls	14	1968 (6.3)	38 (23)	.19 (.05)	53 (37)	1.1E-3 (2.7E-3)	.63 (.12)	46 (19)	1 E-8 (0)	87Q 85H (15) (14)	2 NO 8 YES
Overlapped Piles	6	1965 (5.8)	54 (24)	.22 (.10)	73 (37)	2E-3 (4.4E-3)	.59 (.05)	67 (36)	- -	90 Q 87H - (5)	2 NO 4 YES
Upstream Blankets	21	1961 (9.4)	36 (35)	.06 (.03)	61 (58)	2.4E-4 (4E-4)	1.5 - 3.8 (.9) (2.9)	349 (319)	3E-6 (6E-6)	58Q 56H (8)	10 NO 9 YES
Grout Curtains	18	1965 (5.6)	49 (35)	.16 (.06)	69 (58)	1.3E-3 3E-3	15 (11)	75 (59)	8E-6 3E-5	80Q 82H (13)	6 NO 12 YES

seepage reduction measures. This however, was expected as these parameters, in their crudest form, are analogous to the parameters as defined by Darcy's Law.

Darcy's Law may be expressed as follows:

$$Q = k\ i\ A$$

where Q = quantity of seepage

k = coefficient of permeability

i = hydraulic gradient

A = total cross-sectional area normal to the direction of flow

For the purpose of this thesis, k was treated as permeability of the foundation soil, i was taken as the difference between the forebay and tailrace level divided by the base length of the dam and A was taken as the depth of foundation material to an impervious base (This analogy is explained on Figure 12).

Since the quantity of seepage beneath a ~~dam was~~ concluded to be indicative of performance and the quantity of flow through the foundation soil is governed by Darcy's Law, the data from the case histories were plotted with respect to permeability, nominal hydraulic gradient and depth of foundation material or area.

These data are plotted on a three dimensional (3-D) grid and three, two dimensional (2-D) plots (See Figures 13 and 14, 15 and 16, respectively),

As indicated by Figure 13, no major relationship can be drawn from the 3-D plot. However, this is not felt to be the case for the three, 2-D plots.

Figure 14 is a plot of the depth of foundation soil (Area) versus the nominal hydraulic gradient beneath the dam for the various seepage reduction measures. Similar to this, Figure 15 is a plot of permeability of the foundation soil versus area and Figure 16 is a plot of permeability versus nominal hydraulic gradient.

The limits of each seepage reduction measure have been bounded separately on Fi res 14, 15 and 16. If a seepage reduction measure had not been taken to the base of the pervious material, i.e., a partial cutoff, it was then only represented by a data point and was not bounded.

For the convenience, of the reader, ranges of the area, gradient and permeability for each seepage reduction measure were plotted on Figures 17, 18 and 19 respectively.

A detailed description of each figure will be discussed later with respect to the individual seepage reduction measure. However, in brief it is interesting to note the following:

1) Area - Grout curtains are used over the broadest range of depths. Whereas, slurry trenches are the shallowest form of cutoff.

2) Gradient - Overlapped pile walls are used when the gradient beneath a dam is steepest. However, upstream blankets are used when this gradient is the least.

3) Permeability - Generally all the seepage reduction measures discussed are used over a wide range of permeability.

4.2.1 Slurry Trench Cutoffs

Slurry trenches have generally been used as the shallowest form of complete cutoff. They have normally been constructed in fine to medium grain material. However, as indicated by the broad range of permeabilities on Figure 15, 16 and 19 they can be used over a very wide margin of foundation materials.

The broad range of nominal gradients would suggest slurry trenches may be constructed beneath a wide range of dam profiles. However, as apparent in Table 11, these dams are generally of low head and have broad base lengths compared to those of other measures, i.e., the average nominal gradient is 0.12.

The average width of a slurry trench was determined to be approximately 2.4 m. The expected permeability of this form of cutoff may be in the order of 1E-7 m/s.

Of the 17 case histories reviewed, the position of the cutoff trench could only be determined for 16 dams. These data indicate that 56% of the case histories located the cutoff trench upstream, whereas 44% of the dams had the trench located beneath the centreline. In all cases where the information was available, the top of the trench was flared in order to minimize the effects of differential settlements between the granular alluvium and plastic soil - bentonite mixture. Based on available data from the Duncan, Francisco Zarco and Khancoban Dams, differential settlement has not been a problem. In the case of the latter two dams, the core material has settled with the trench material. However, in the case of the Camanche 2 Dam, where the cutoff is located upstream and covered by an extension of the core, the trench material was found to settle more than the overlying blanket material. The performance of this measure was considered acceptable, however, concern over a potential seepage path was raised.

In approximately sixty percent of the case histories studied, the cutoff trench was excavated and keyed into bedrock. However, 40% of the walls were terminated in what was considered to be an impervious stratum. The latter trenches are considered to be partial cutoffs and thus less efficient. However, from available data these walls were found to be just as effective in reducing seepage as were the complete cutoffs.

Based on available case histories, slurry trench cutoffs are generally used in conjunction with relief wells. However, they have also been used effectively with grout curtains and upstream blankets.

The efficiency of a slurry trench, which is ranked fourth compared to other measures, does not vary in a noticeable trend from one extreme boundary limit to the other on any of Figures 14, 15 or 16. The only recorded unacceptable performance of a slurry trench was at the D-20 Dam in Quebec. It was determined that a portion of granular material had not been excavated during construction. Consequently, a pervious window was created in the wall.

On review Figures 14 through 19 inclusive, overlapped regions of permeability, area and gradient can be observed between many of the seepage reduction measures. This would suggest that in certain cases either measure would be applicable. In the case of slurry trench cutoffs, it is suggested that they would be a better alternative to upstream blankets in these regions because of their greater efficiency and lower cost. In fact, one upstream blanket case history (Camanche 2) in this overlapped region was later repaired using a slurry trench cutoff.

4.2.2 Intersecting Pile Walls

Intersecting pile walls were the fore-runners to the other concrete diaphragm walls. They have been used under a variety of dam profiles, as indicated by the range of gradients on Figure 18. However, as in the case of slurry trench cutoffs they have only been incorporated beneath relatively low head dams.

Based on available data, intersecting pile walls have been used when the foundation material is very pervious and generally fine to coarse grained.

As shown on Figures 14, 15 and 17, intersecting pile walls have been used over a very narrow depth range due to construction limitations.

In regards to the location of this seepage reduction measure, it appears 50% of the case histories were upstream and 50% were located beneath the centreline of the dam. In all cases some form of plastic capping material was placed at the top of the wall to reduce the risk of cracking the core material.

All of the intersecting pile walls reviewed were complete cutoffs and were found to perform well. However, due to construction techniques the efficiency of this measure is low in comparison to the other diaphragm walls. Therefore, it is recommended that other seepage reduction measures be used if a high degree of efficiency is required.

4.2.3 Panel Walls

Panel walls have normally been used for foundations which were too deep to use intersecting pile walls and too shallow to use overlapped pile walls. As represented on Figure 14 by the narrow span of gradients, the range of dam profiles under which panel walls have been placed has been very restrictive in comparison to most other seepage reduction measures.

Panel walls have been successfully constructed in alluvium, containing 1 m diameter boulders, to depths greater tha 65 m. In the case of the Bighorn Dam, in Alberta, it was necessary to carry out blasting procedures in order that the trench could be excavated.

Of the 14 case histories reviewed, 60% of the panel walls were constructed beneath the centreline of the dam, whereas 40% of the walls were located upstream. The position of the measure does not seem to have any influence on its performance record. As in the case of intersecting pile walls, some form of capping measure was constructed at the top of each panel wall so as to protect the core material.

Panel walls have generally been used as complete cutoffs. However, approximately 12% of the case histories were found to be partial cutoffs. The performance of these walls were also found to be good.

In three cases throughout the world, double concrete panel walls have either been constructed or proposed. In each case, the walls were 3 m apart and were extended to bedrock. At the Obra and Tenughat Dams, the foundation soil in between the walls was to be grouted. The peformance record of these dams was not available. However, the effectiveness of the double diaphragm wall constructed at the Manicouagan 3 Dam was well documented. Dascal (1979a) reports that the upstream wall cracked under stresses induced by the dam's self-weight and reservoir loads. The efficiency of the upstream and downstream walls was found to be about 65 and 92%, respectively. Upon completion of a grouting program, in 1976, the efficiency of the upstream wall was increased to 70%. Even though the double panel wall did not perform well, Dascal (1979a) states the use of the wall was justified and that a single wall would have deteriorated even faster.

Dascal (1979b) also notes 85% of the vertical deformation of the wall was mainly due to the load transmitted by negative skin friction, whereas compression due to the self-weight of the dam was only 15%.

The efficiency of a panel wall was found to be quite high and very consistent throughout the various settings that they have been constructed. However, of the case histories studied, the performance of 20% of the walls was considered unacceptable. This fact is believed to be a reflection of design philosophy, in that 43% of the case histories relied solely on the effectiveness of the panel wall to reduce and control seepage quantities.

4.2.4 Overlapped Pile Walls

As indicated on Figure 14 and through 19, inclusive, overlapped pile walls have been used over a very narrow range of foundation permeabilities, foundation depths and dam profiles.

Generally they have been used when foundation depths exceed 55 m and the dam profile results in steep nominal hydraulic gradients. Overlapped pile walls have also been successfully constructed through alluvium containing a high percentage of boulders. As in the case of the Bighorn Dam, blasting was carried out in order to advance the cutoff wall at the Manicouagan 5 site.

Based on available data, 67% of the case histories located the cutoff beneath the centreline of the dam, whereas 33% of the cutoffs were situated upstream. Some form of pile cap had been provided for each of the case histories reviewed. The position of the cutoff did not tend to influence the effectiveness of this reduction measures.

Efficiencies were found to be very high and consistent throughout the range of case histories. However, poor performance was observed in 2 out of the 6 case histories studied. In the cases of both the Zoccolo and Manicouagan 3 Dams, seepage rates increased due to the deterioration of the overlapped pile walls.

4.2.5 Upstream Impervious Blankets

Upstream Blankets have tended to be used when the permeability of the foundation material is in the order of 1E-4 m/s. This is one order of magnitude less than the other five seepage reduction measures discussed.

The average nominal hydraulic gradient beneath a dam using an upstream blanket is 0.06. This is in close agreement with an acceptable value of 0.067 presented in Chapter 2.0.

Upstream blankets generally have had a poor track record. In over 50% of the case histories studied, the performance of upstream blankets has been unacceptable. Based on available data, 60% of the incidents took place at first filling and 40% at some later date. Unacceptable behaviour was attributed to the following factors:

1) Inadequate stripping of pervious foundation material at the Hills Creek Dam.

2) The blanket was not long enough as in the case of the Mohawk, Townshend and Camanche 2 Dams.

3) Inadequate seepage control measures had been installed to release uplift pressures at the toe of the dam.

However, in approximately 30% of the case histories reviewed, the quantity of seepage was found to decrease with time. It appears that if this decrease does not take place in the first three years after first filling, it will not occur.

4.2.6 Grout Curtains

As shown on Figures 14 through 19 inclusive, grout curtains have been used under a wide variety of circumstances. However, on average they are used when foundation depths are greater than 70 m and the nominal hydraulic gradient is in the order of 0.16.

Of the 18 case histories studied, 66% of the grout curtains were located beneath the centreline of the dam and 44% were located upstream. In all cases the grout curtains were complete cutoffs. The position of the cutoff, also did not tend to influence its overall performance.

The performance record of a grout curtain is similar to that of an overlapped pile wall, in that 67% of the case histories were effective and 33% were not. In the cases of the Durlassboden and Girna Dams, the efficiency of the grout curtains decreased with time. This is believed to be the result of progressive deterioration of the curtain. However, in three instances the quantity of seepage beneath the dam was found to decrease with time suggesting some form of healing or improved efficiency with time.

In three of the case histories reviewed, the core of the dam above the cutoff cracked. However, this phenomenon was the result of arching between abutments rather than due to the presence of the grout curtain.

It is believed overlapped pile walls could have been used in place of grout curtains in many instances. However, due to personal preferences this has not been the case.

4.3 Relief Wells

Relief wells were initially developed to be used in conjunction with upstream blankets. Therefore, as expected a high percentage, 60%, of the case histories studied involved the use of relief wells with upstream blankets.

Based on available data, initial relief well installations are only adequate 50% of the time. This is thought to be a reflection of the design philosophy, in that relief well systems are designed to a large extent by an observational approach. It is interesting to note however, unacceptable performance was usually associated with wider spacing between wells and higher nominal hydraulic gradients. The degree of penetration, available head and permeability of the foundation material did not seem to influence the performance. The size of well appeared only to have a marked influence on the performance of the system if it was too small to handle seepage quantities.

To design a relief well system in accordance with the design procedures presented in Chapter 2.0, the engineer must pass judgement on an acceptable head midway between the wells. This is considered proportional to the extent and thickness of the impervious top layer downstream, if one exists. However, typically this layer is semipervious or nonexistent. Although the design procedures can account for this non-ideal situation, the designer must still decide what head downstream he will consider acceptable.

Back calculations to determine acceptable heads were performed. The calculated values were normalized through division by the total available head for the dam.

In the case of upstream blankets a very good correlation between acceptable performance and the ratio of the head midway between the wells to the available head was observed. It appears for fully penerating wells if this ratio is less than 4%, the well spacing is adequate. However, for partly penetrating wells this value must be even lower (See Figure 20).

For those cases which used relief wells with cutoff walls, little consistency was found amongst the calculated values. It is difficult therefore, to assess acceptable ratios of head downstream to upstream. Possibly if the design procedures were expanded upon to account for the presence of a cutoff wall, consistent ratios could be determined. Thus it is recommended that a theory for the design of relief well systems with cutoff walls be developed. However, since a very effective cutoff wall would not need relief wells the design procedure would have to assume either the cutoff wall would have a low initial efficiency or a deteriorated efficiency would take place with time.

5.0 CONCLUSIONS AND RECOMMENDATIONS

This thesis has examined the performance of seepage measures beneath earth and rockfill dams on pervious soil foundations. It consisted essentially of a compilation and analysis of available case histories throught the world.

The following are the conclusions of this study:

1) Acceptable seepage beneath a dam associated with a hydroelectric scheme founded on alluvium is less than 0.0006 cumecs or 1.0 E-6 cumecs/ ln m of dam.

2) Unacceptable seepage is greater than 0.0006 cumecs or 1.0 E-6 cumecs/ ln m of dam. The degree c un-acceptability is governed by the order of magnitude above these values.

3) The average hydraulic gradient beneath fill dams constructed on pervious foundations throughout the world is 0.15. This nominal gradient does not appear to influence the performance of the dam.

4) The average time to some form of increased seepage or incident after construction is 4.0 years. How-

ever, unacceptable behaviour may be anticipated even after 50 years of acceptable performance.

5) Seepage quantities are considered to be a valid measure of performance for any seepage measure.

6) Slurry trench cutoffs are effective measures to be used beneath low head dams to depths of 30 m. It is recommended slurry trench cutoffs be used in place of upstream blankets if the depth of alluvium is below 30 m and the nominal gradient is less than 0.1.

7) Concrete intersecting pile walls are not considered the most efficient seepage reduction measure. However, they have a very good performance record.

8) Concrete panel walls are thought to be a very good form of seepage reduction measure to depths of 60 m.

9) Overlapped pile walls should be considered as the most efficient method of sealing alluvium to depths of 120 m. However, their efficiency with time should be questioned.

10) Upstream blankets are not considered to be the best alternative in light of the present, more efficient

seepage reduction measures available. Positive effects because of sedimentation behind the dam should not be counted on.

11) Grout curtains are best suited to be used in conjunction with other seepage reduction measures, such as extending beneath slurry trenches or providing a plastic zone at the top of a diaphragm wall.

12) There does not seem to be any conclusive evidence to suggest where the optimum location of a cutoff wall should be. However, the author would recommend the upstream location over the centreline position on account of accessibility both during and after construction.

13) The phenomenon of negative skin friction must be accounted for in the structural design of a cutoff wall.

14) The present design for relief wells when used in conjunction with upstream blankets is good. However, this design method does not appear to be applicable when relief wells are to be used in conjunction with cutoff walls.

15) Construction procedures and control play a major róle in the future performance of a seepage control and/or reduction measure.

16) Groundwater chemistry will have either a positive or negative influence on the overall performance of a cutoff wall.

Based on the work carried out to date, the author would recommend the following studies to be carried out:

a) Model studies to determine the optimum location of a cutoff wall with respect to both soil-structure-dam interaction and overall stability.

b) The effects that high hydraulic gradients and increased compressive loads have on the long term effectiveness of a concrete diaphragm wall should be assessed in light of their poor performance.

c) Theoretical design procedures for relief wells when used in conjunction with low efficiency cutoff walls should be developed.

d) The effects that groundwater chemistry has on the healing and/or deterioration of both grout curtains and concrete diaphragm walls should be evaluated.

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FIGURE 1 : CONSTRUCTION SEQUENCE OF A SLURRY TRENCH CUTOFF

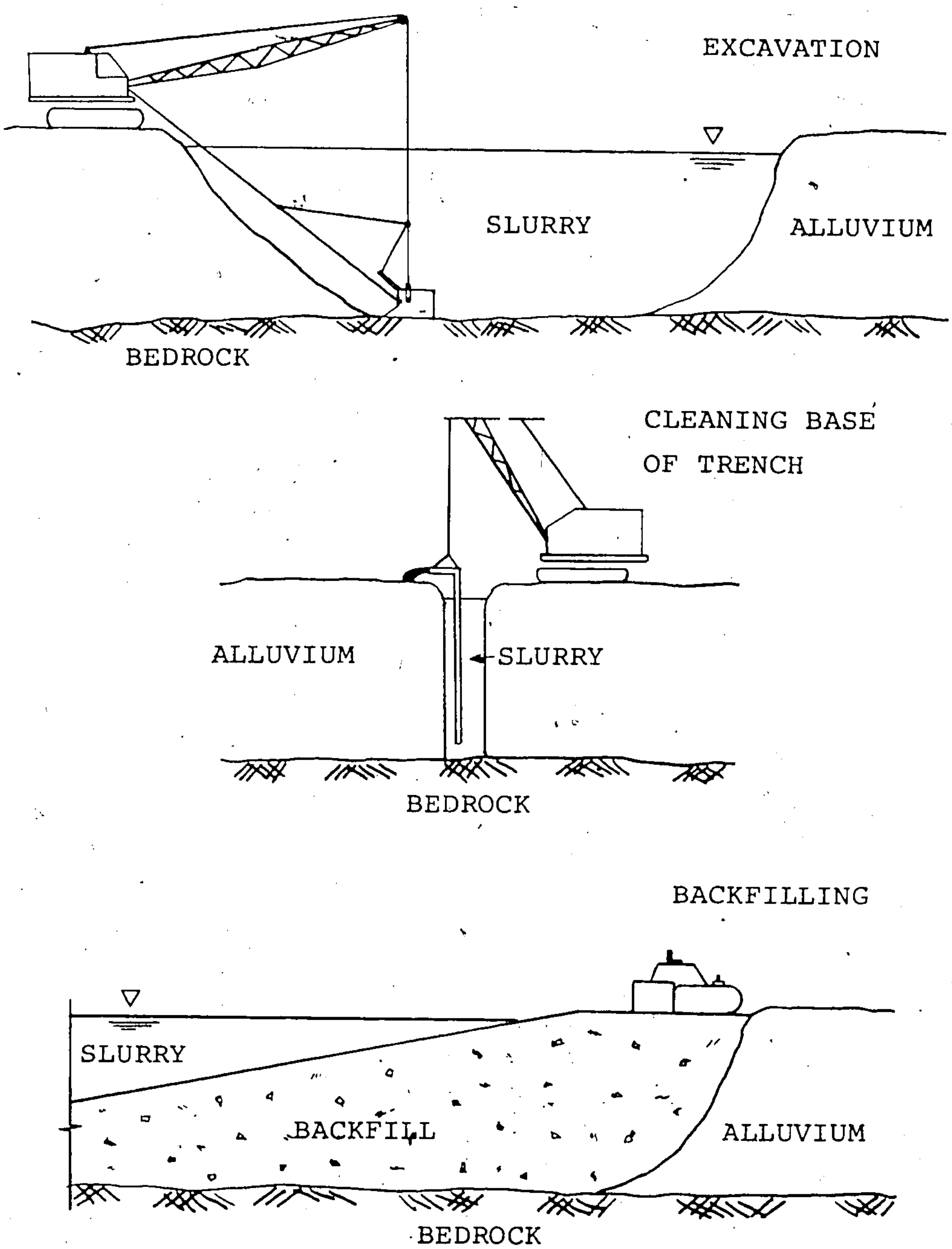


FIGURE 2 : TYPICAL RANGE OF BLENDED SOIL USED FOR SLURRY TRENCH BACKFILL

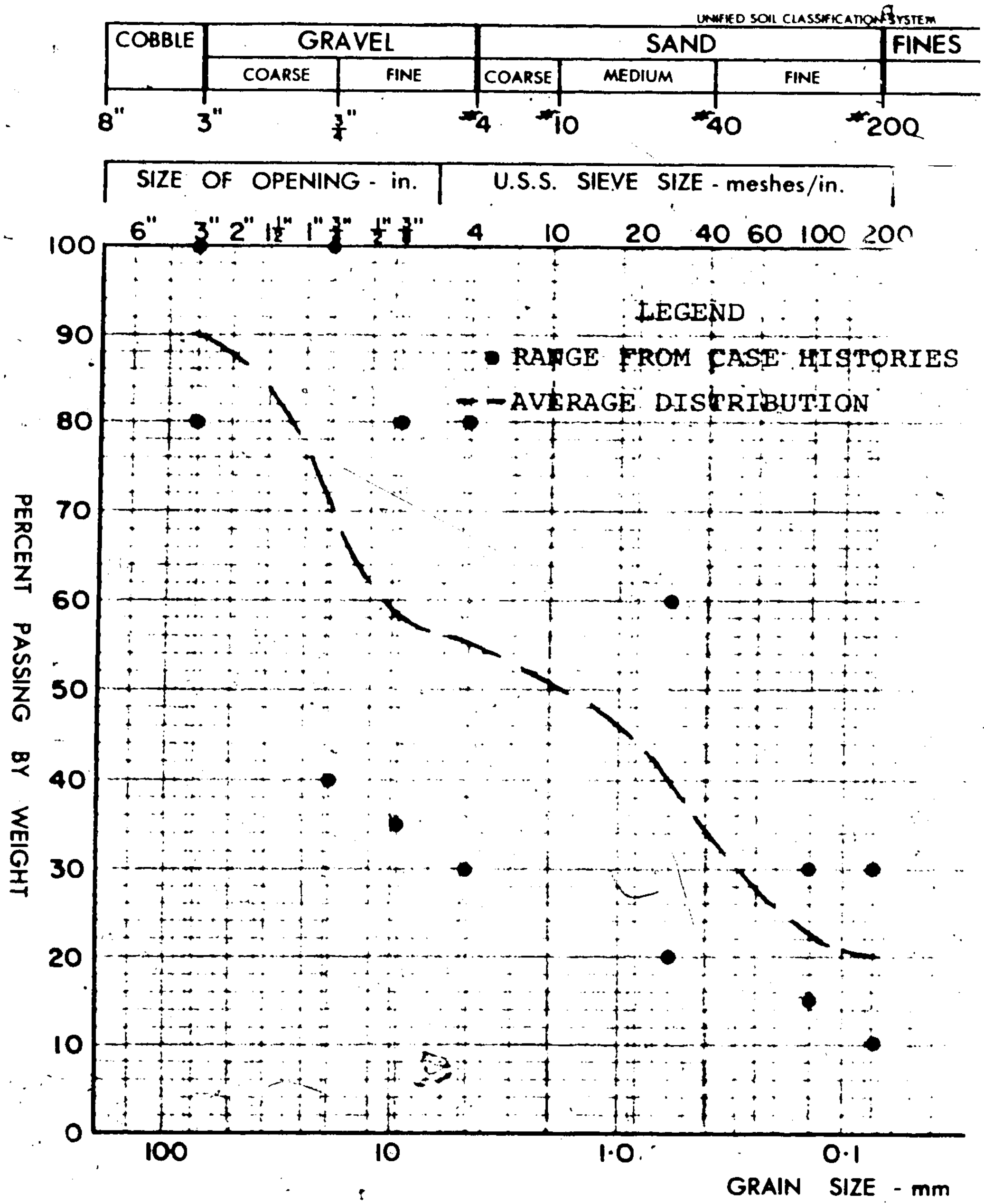
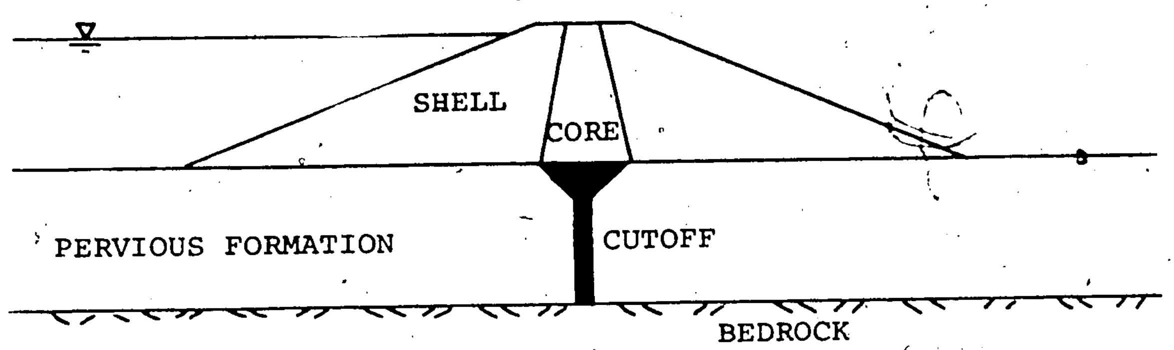


FIGURE 3 : TYPICAL LOCATIONS OF CUTOFFS

a) CENTRELINE LOCATION



b) UPSTREAM LOCATION

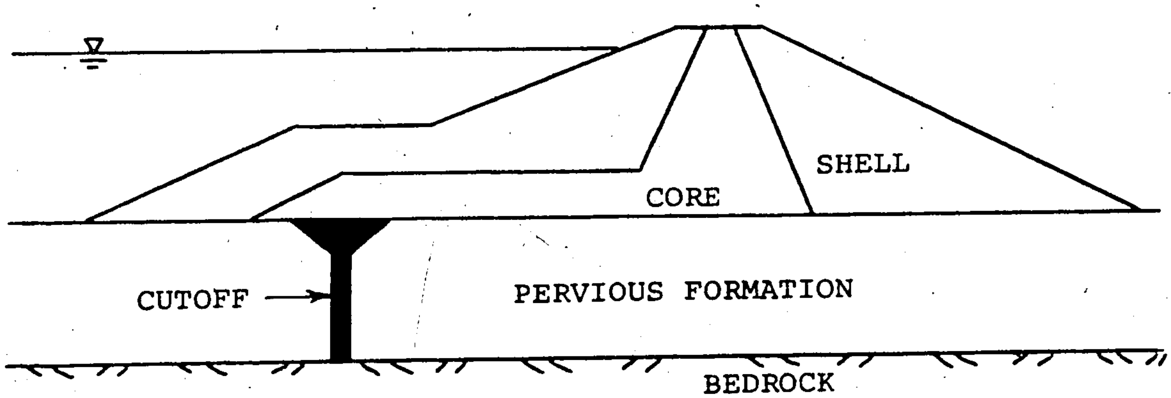


FIGURE 4 : CONSTRUCTION SEQUENCE OF AN INTERSECTING PILE WALL CUTOFF

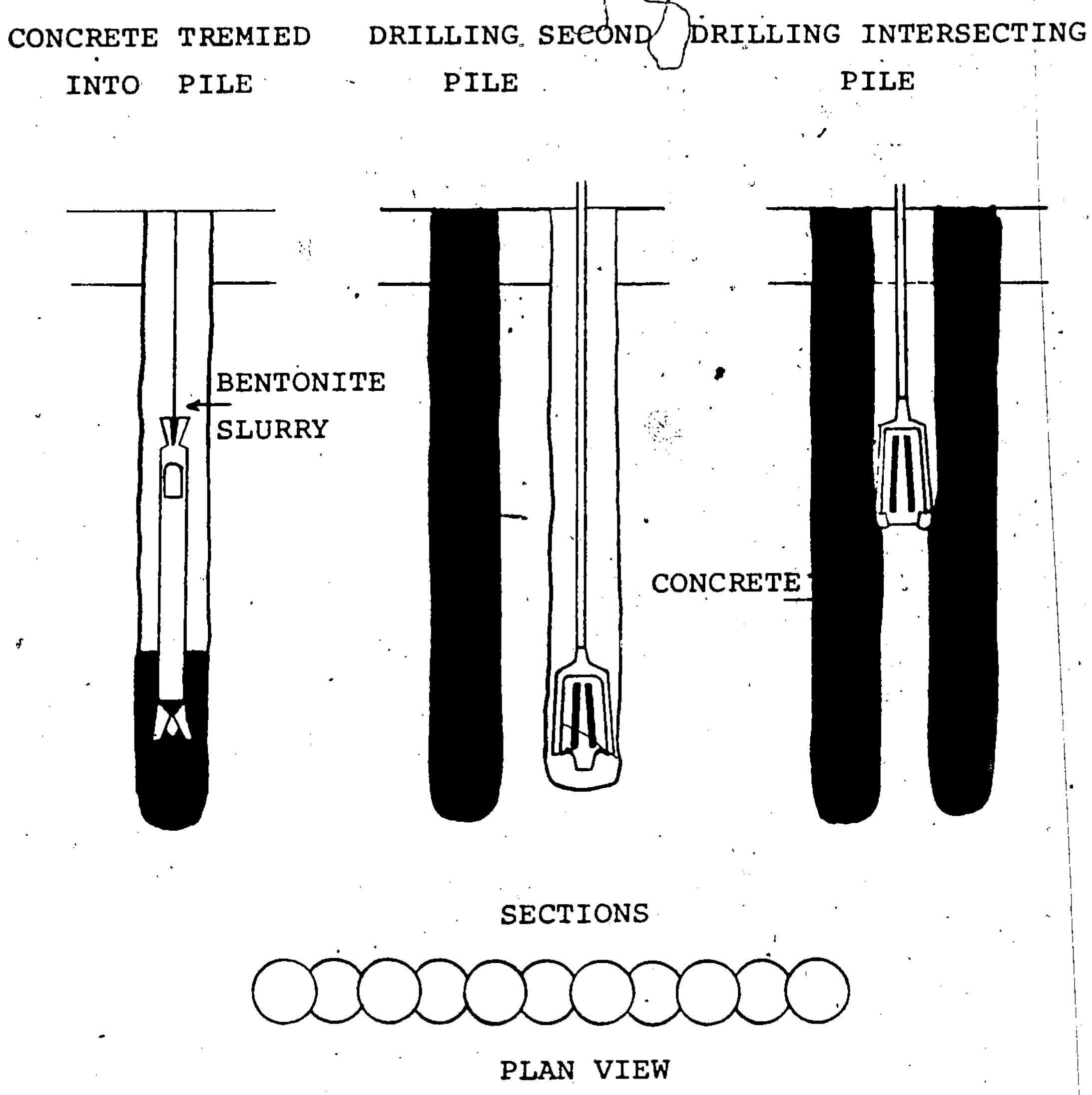
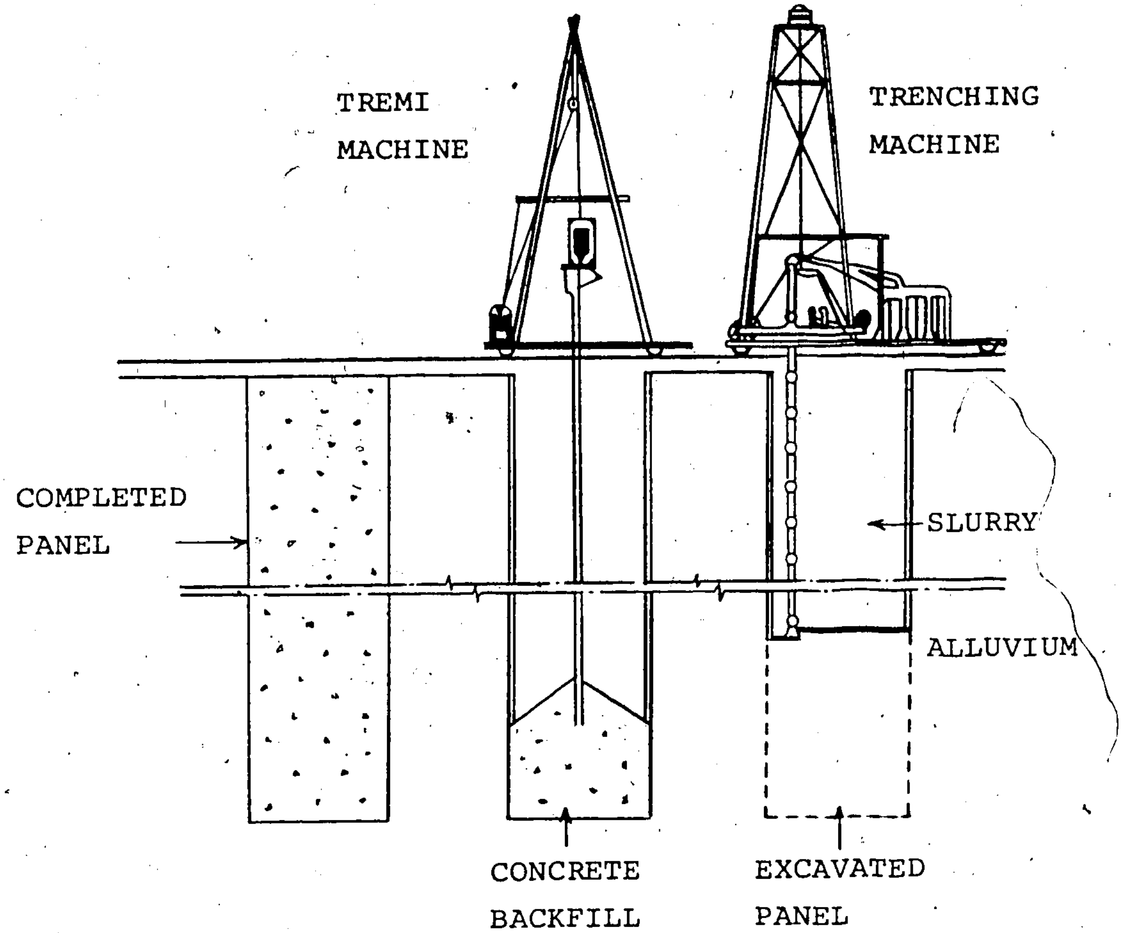


FIGURE 5 : CONSTRUCTION SEQUENCE OF A PANEL WALL CUTOFF



SECTION

FIGURE 6 : CONSTRUCTION SEQUENCE OF AN OVERLAPPED PILE WALL CUTOFF

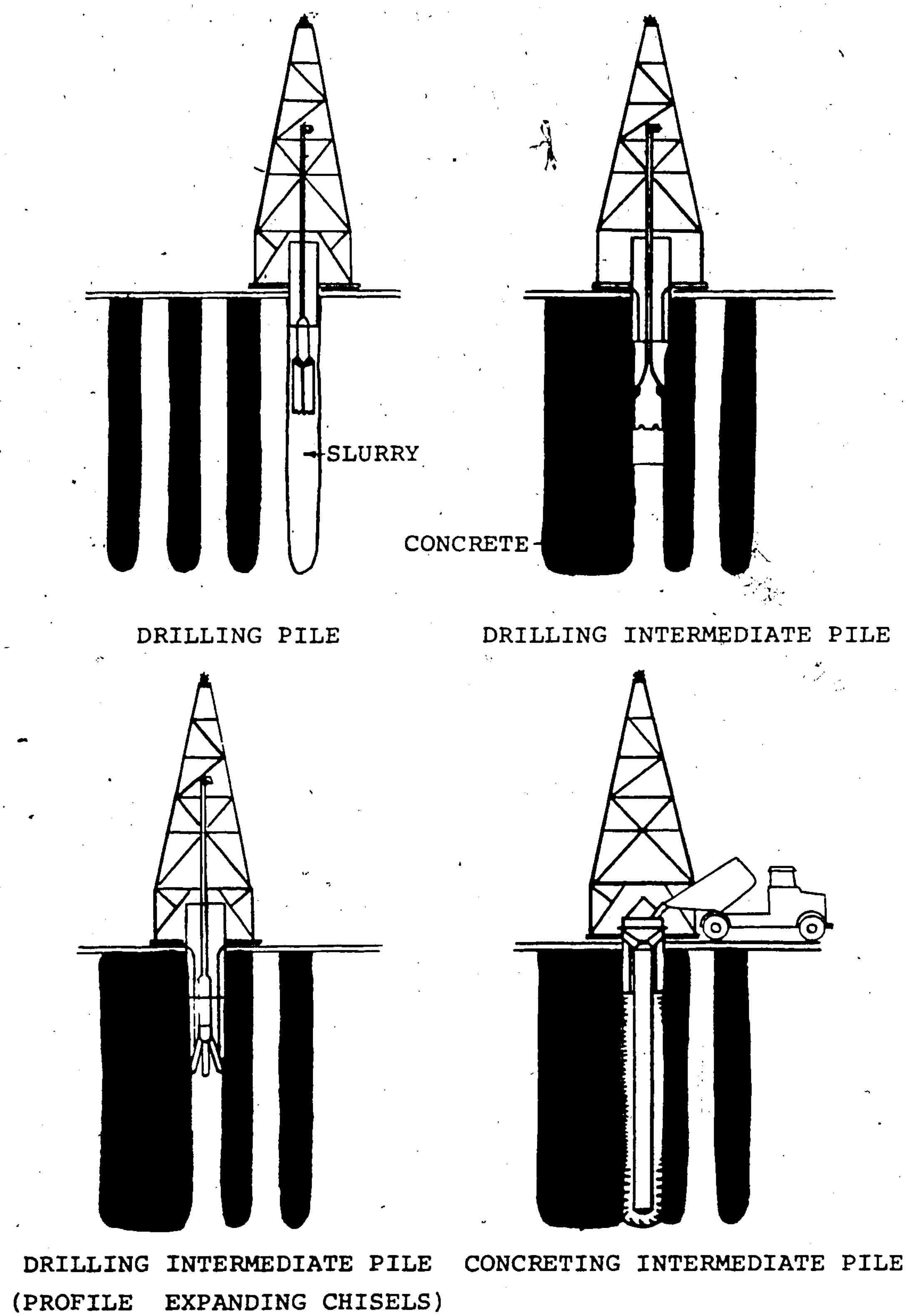


FIGURE 7 : ACCEPTABLE & UNACCEPTABLE SEEPAGE QUANTITIES

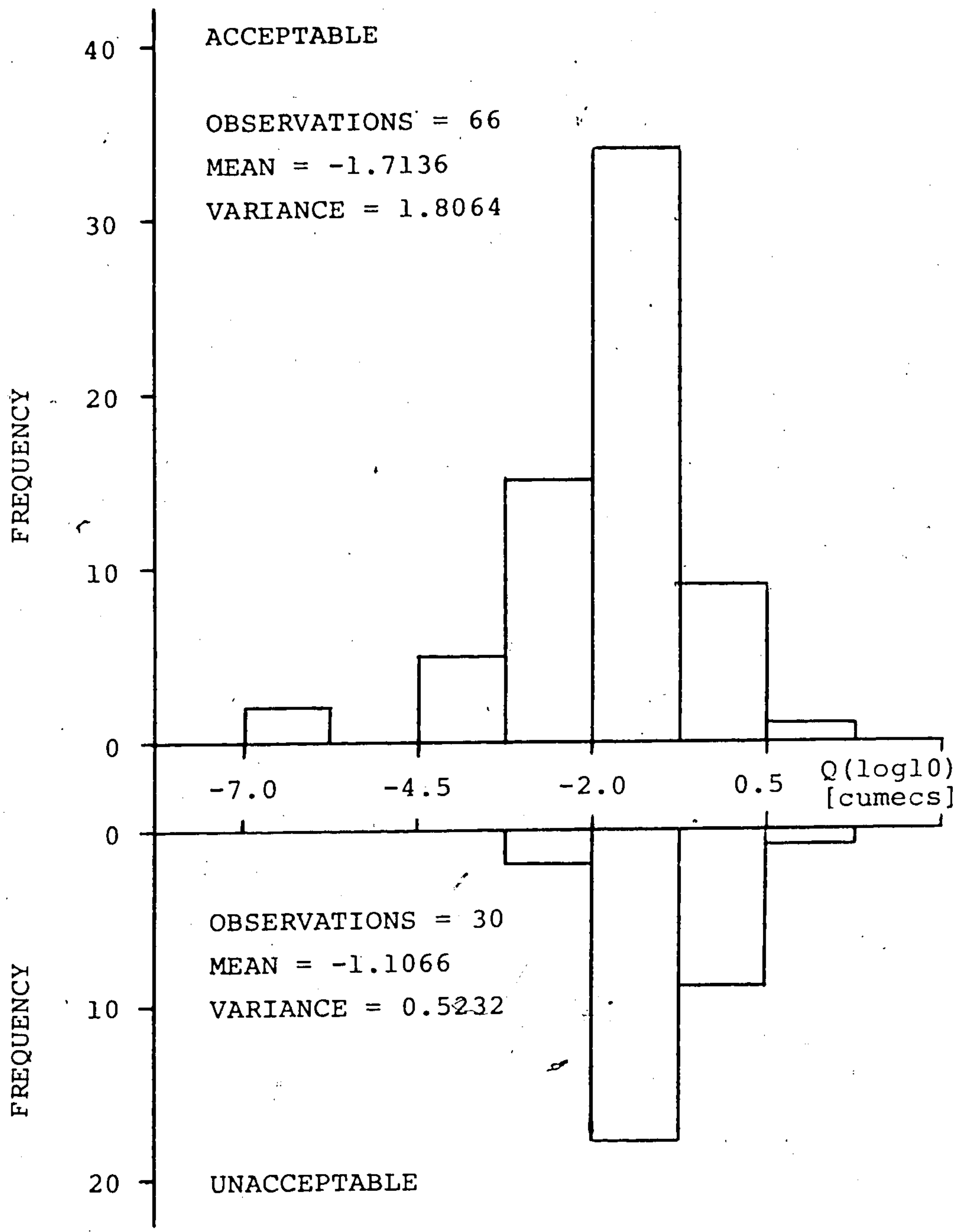


FIGURE 8: CUMULATIVE FREQUENCY DISTRIBUTION : Q

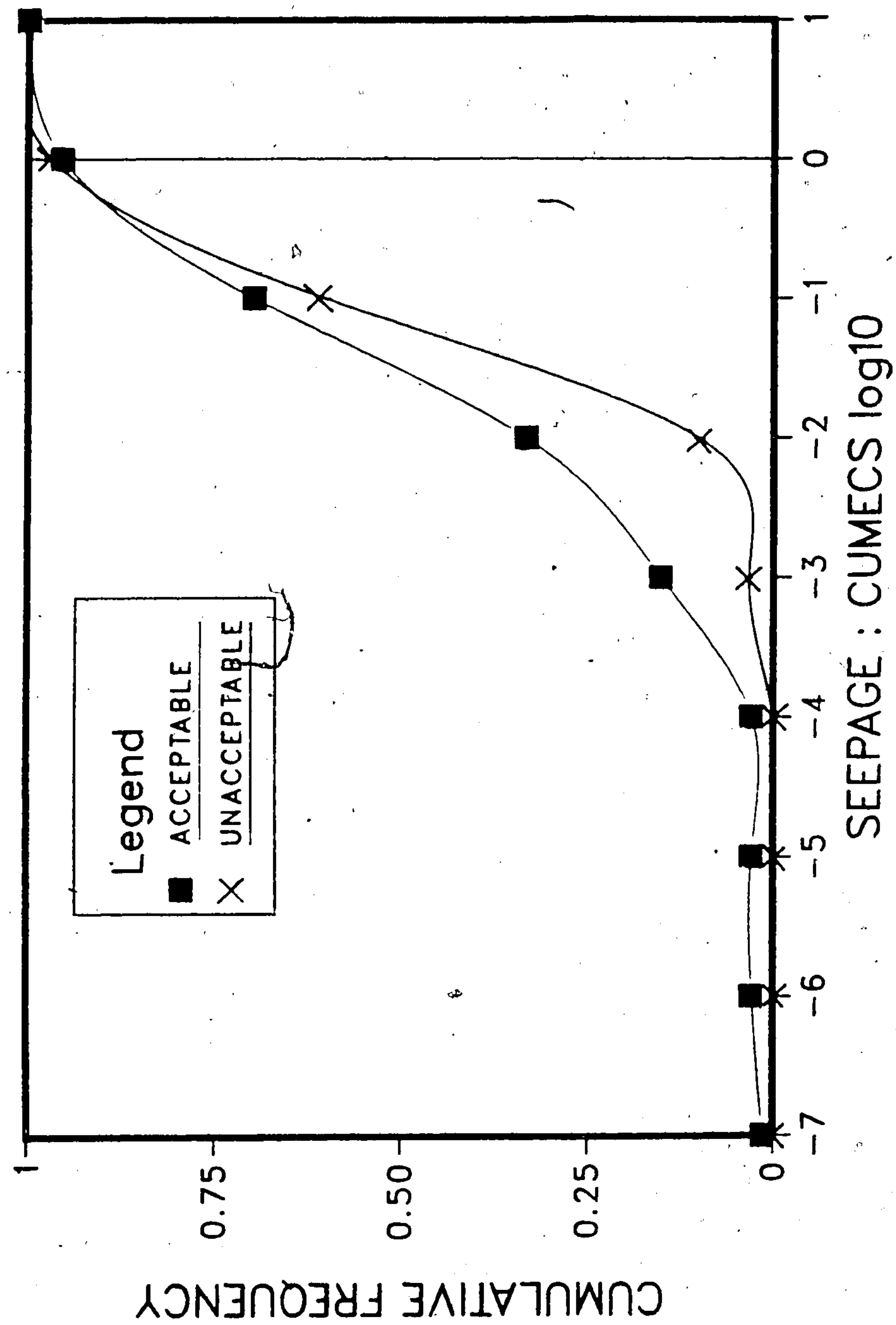


FIGURE 9 : ACCEPTABLE & UNACCEPTABLE SEEPAGE PER LINEAL METRE

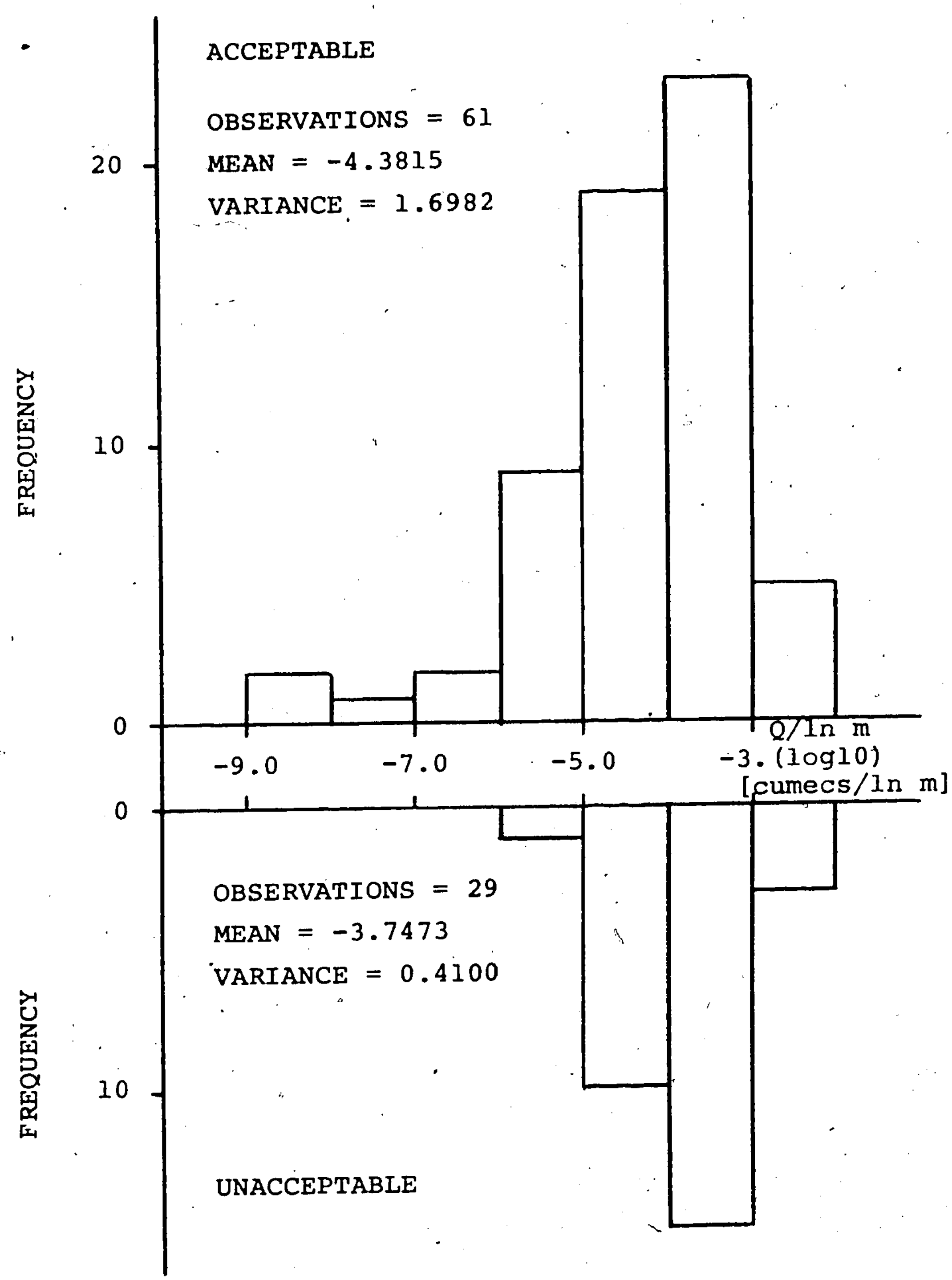


FIGURE 10:CUMULATIVE FREQUENCY DISTRIBUTION : Q PER LN M

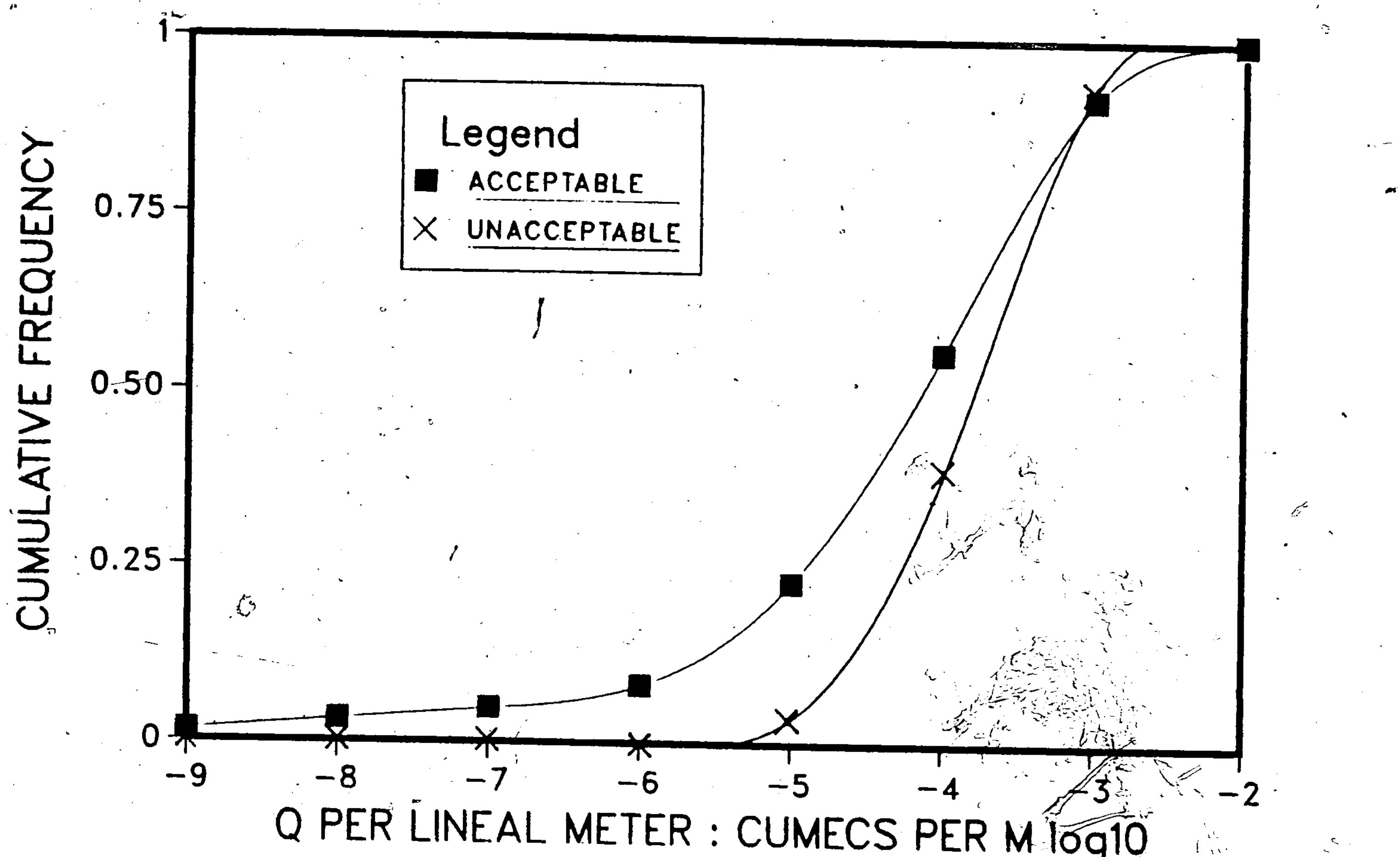


FIGURE 11 : TIME OF UNACCEPTABLE SEEPAGE IN YEARS AFTER CONSTRUCTION

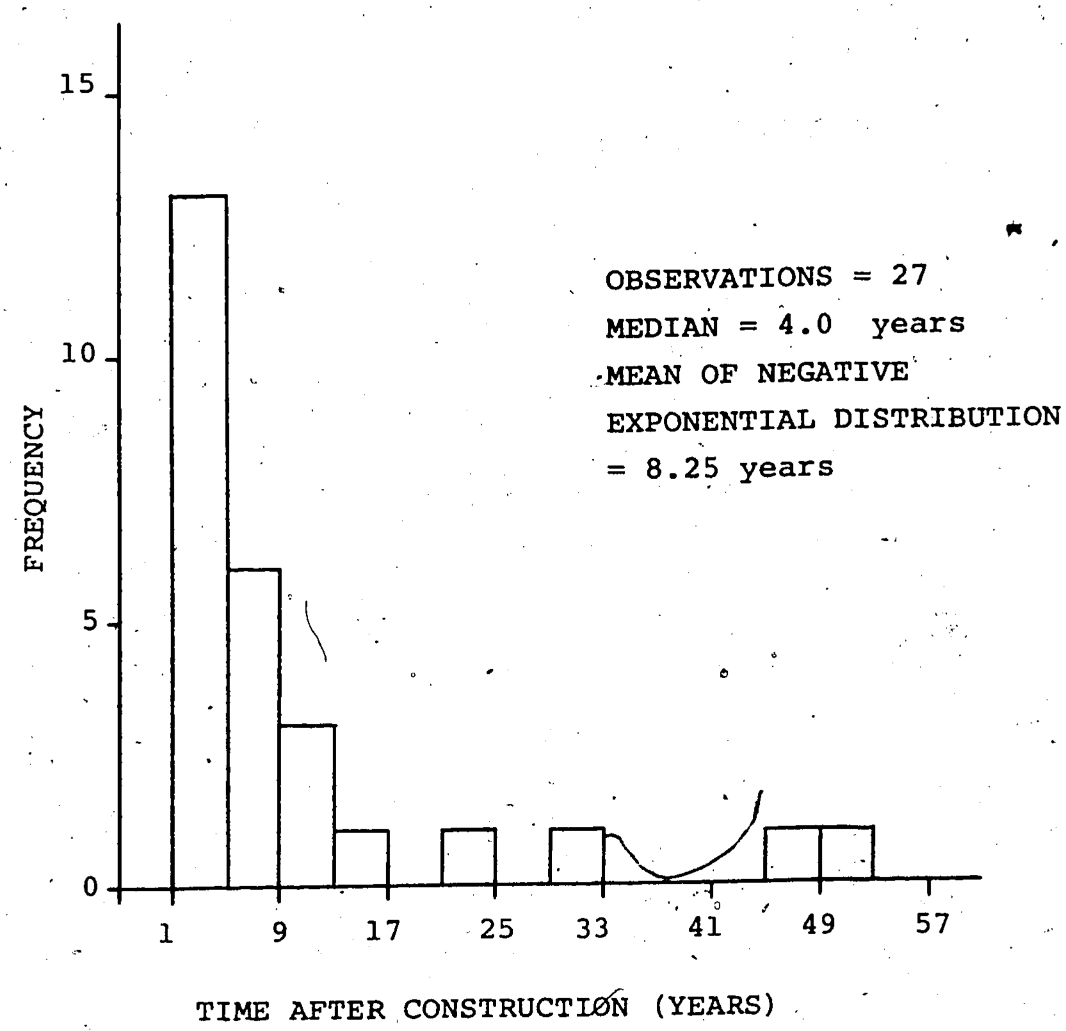
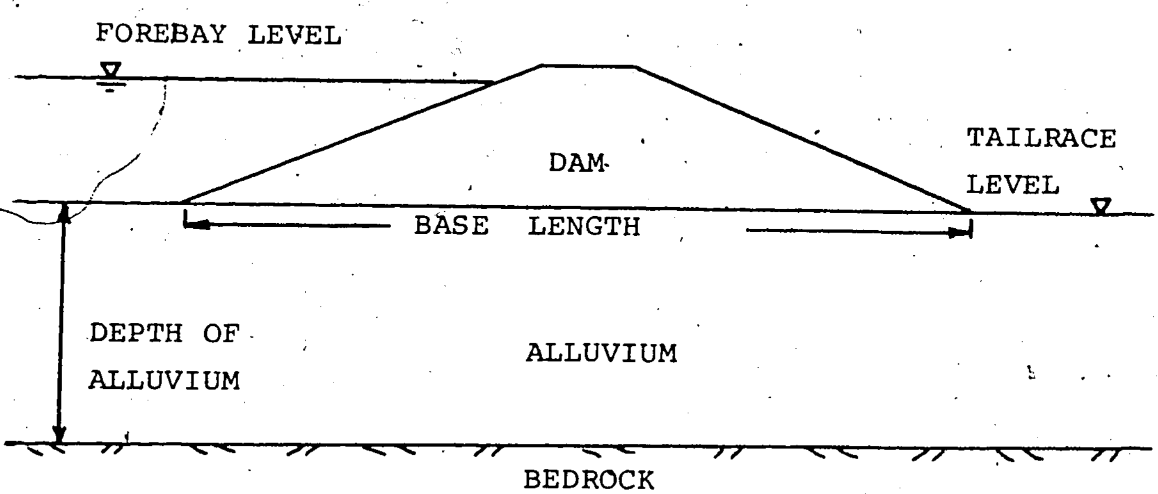


FIGURE 12 : SCHEMATIC REPRESENTATION OF AREA, HYDRAULIC GRADIENT & PERMEABILITY

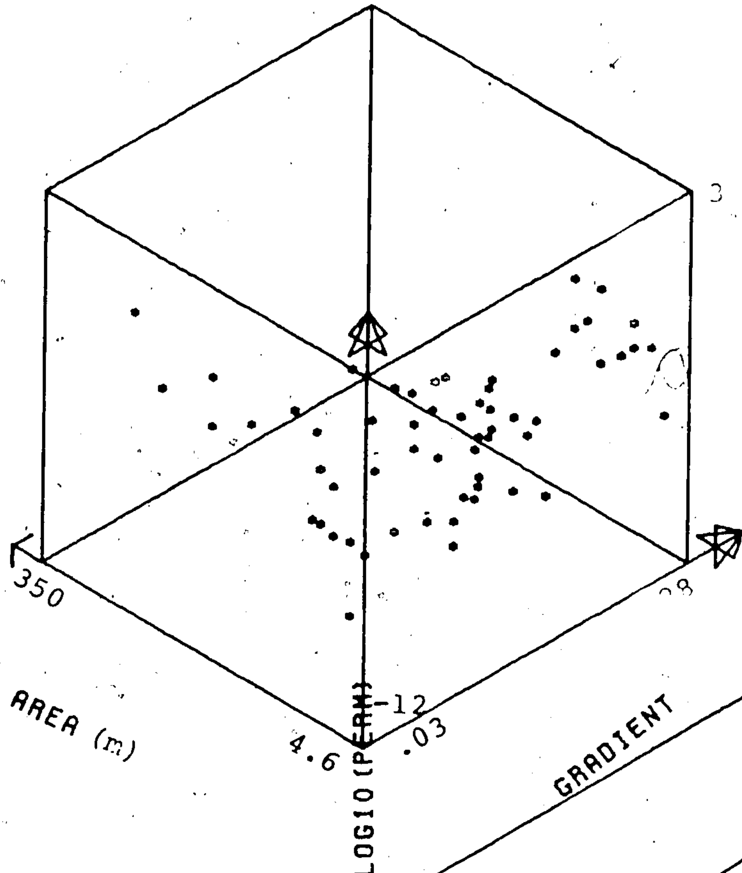


$$\text{HYDRAULIC GRADIENT (NOMINAL)} = \frac{\text{FOREBAY - TAILRACE LEVEL}}{\text{BASE LENGTH}}$$

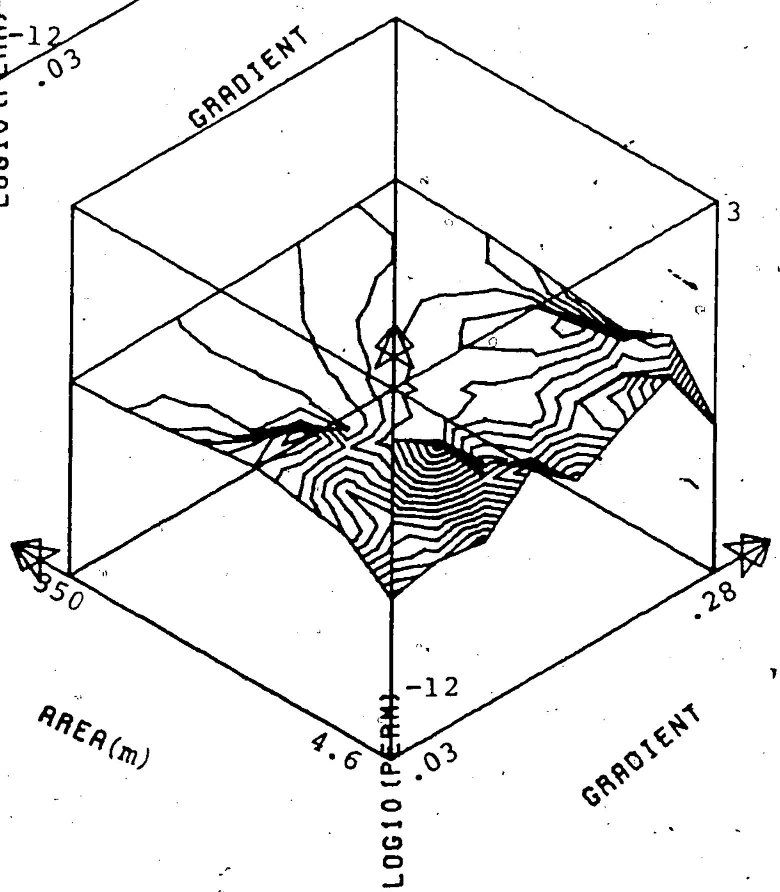
AREA = DEPTH OF ALLUVIUM

PERMEABILITY = PERMEABILITY OF ALLUVIUM

FIGURE 13 : THREE DIMENSIONAL PLOT OF AREA vs GRADIENT vs PERMEABILITY (log10)



DATA POINTS



CONTOURS OF DATA

FIGURE 14 : AREA VS GRADIENT

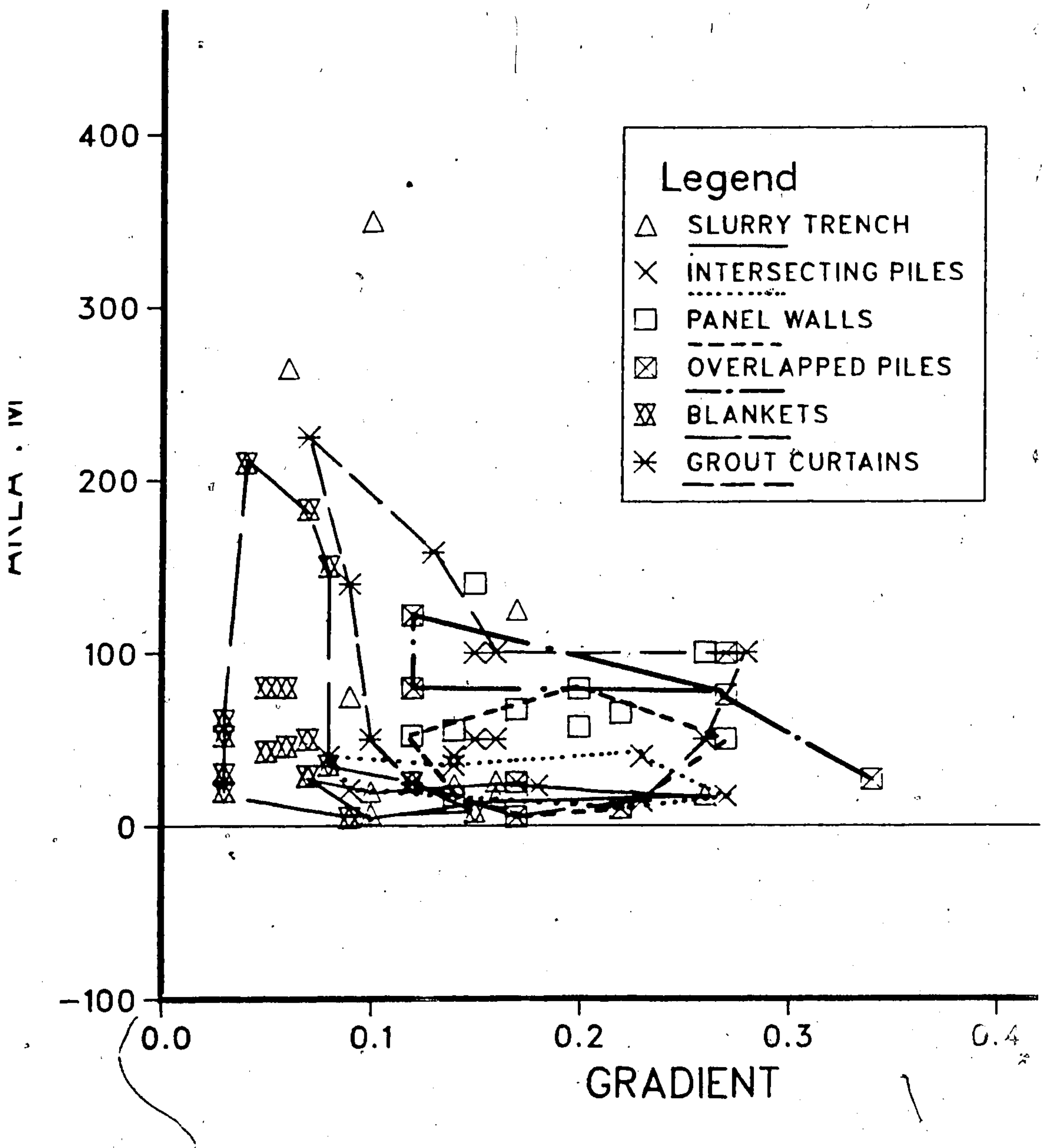


FIGURE 15 : PERMEABILITY VS AREA

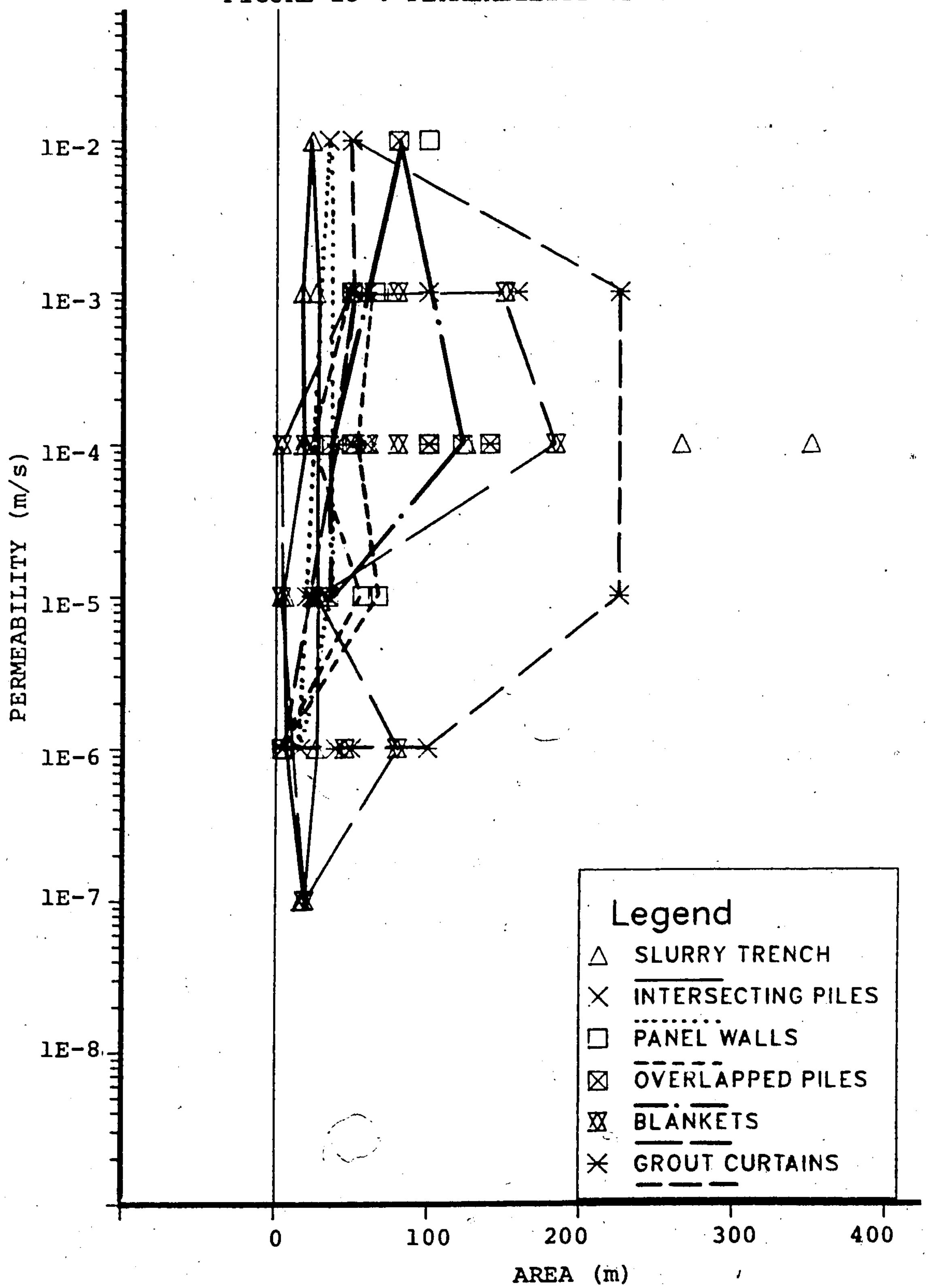


FIGURE 16 : PERMEABILITY VS GRADIENT

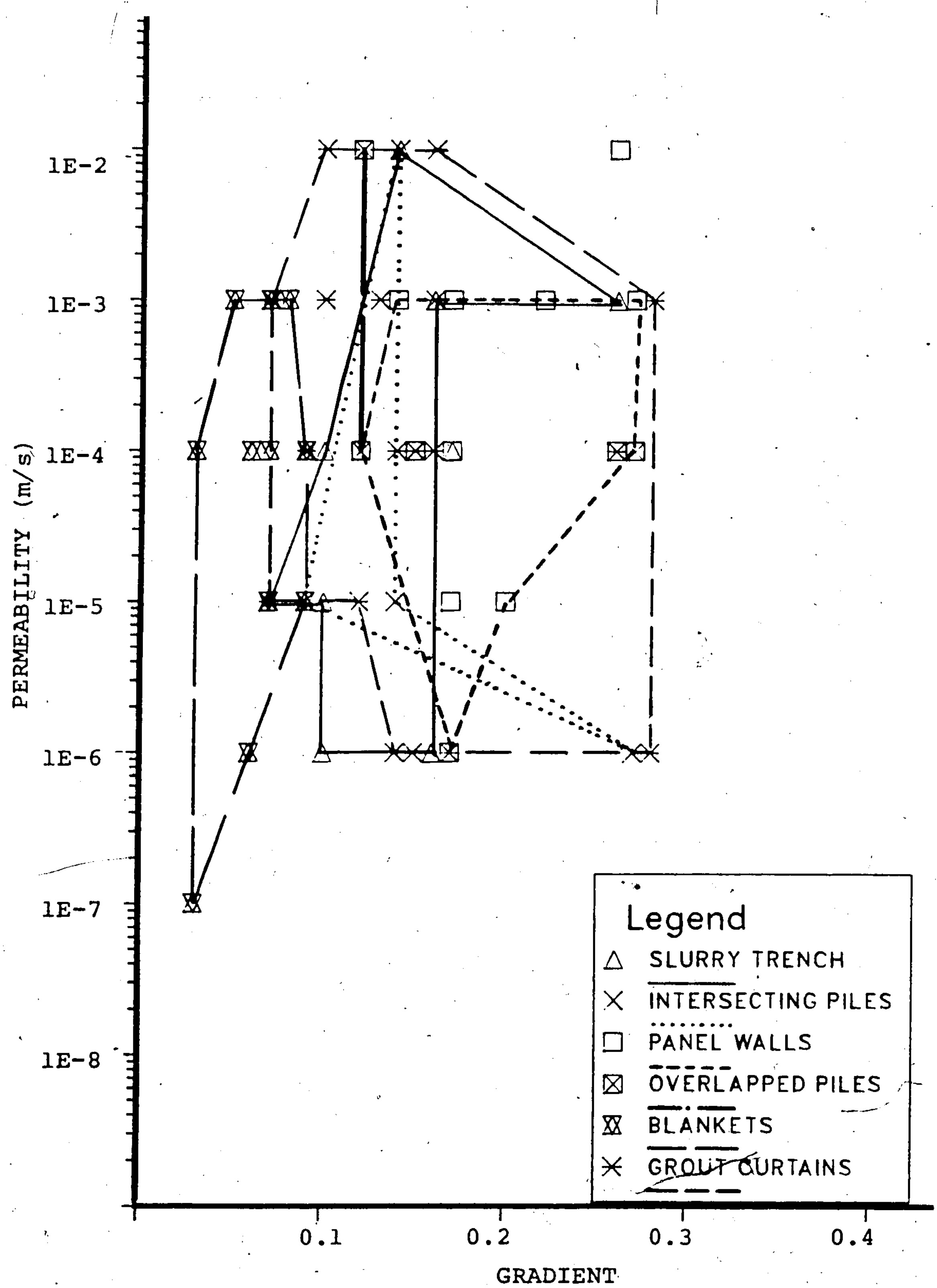


FIGURE 17 : RANGE OF AREAS

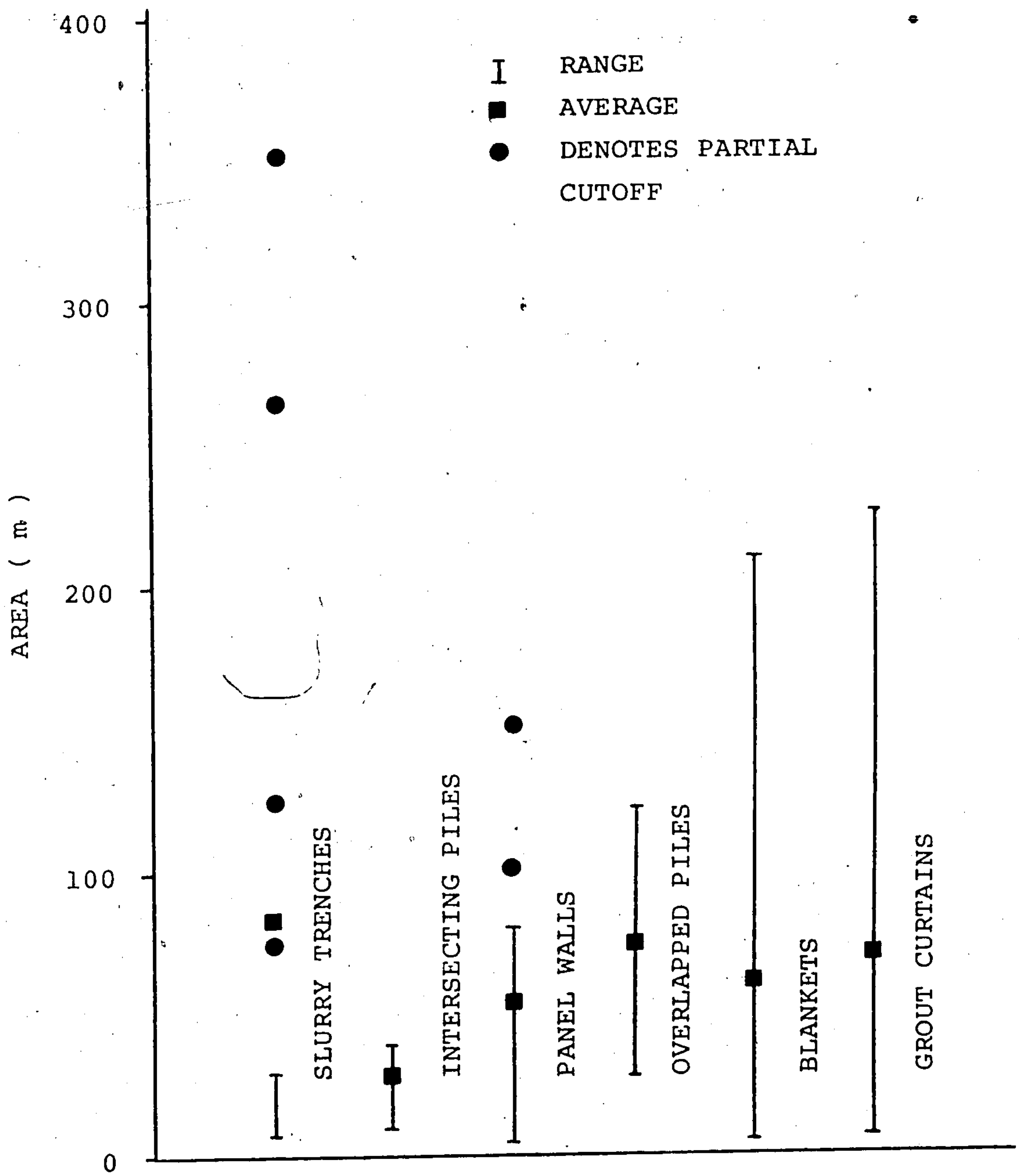


FIGURE 18 : RANGE OF GRADIENTS

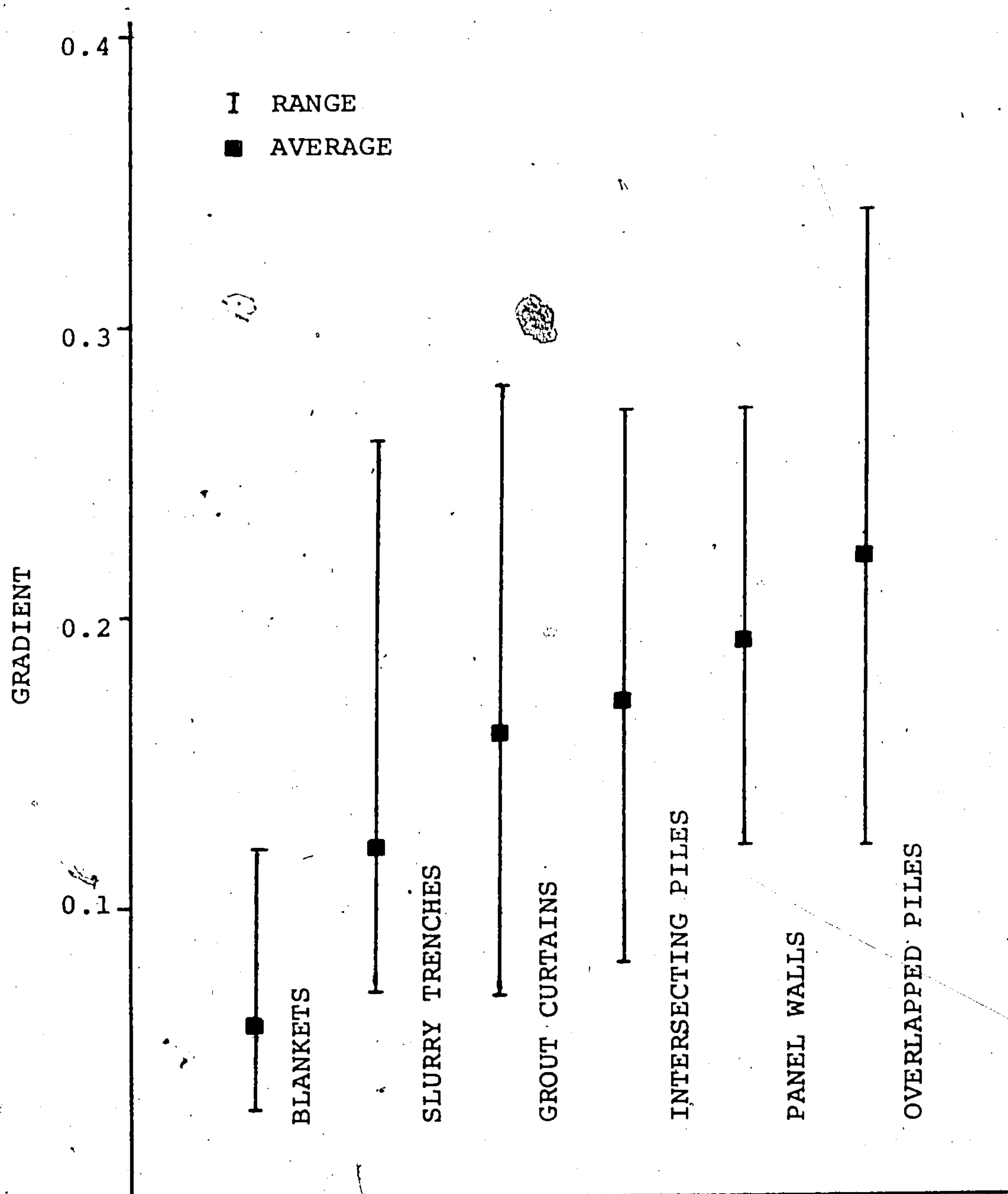


FIGURE 19 : RANGE OF PERMEABILITIES

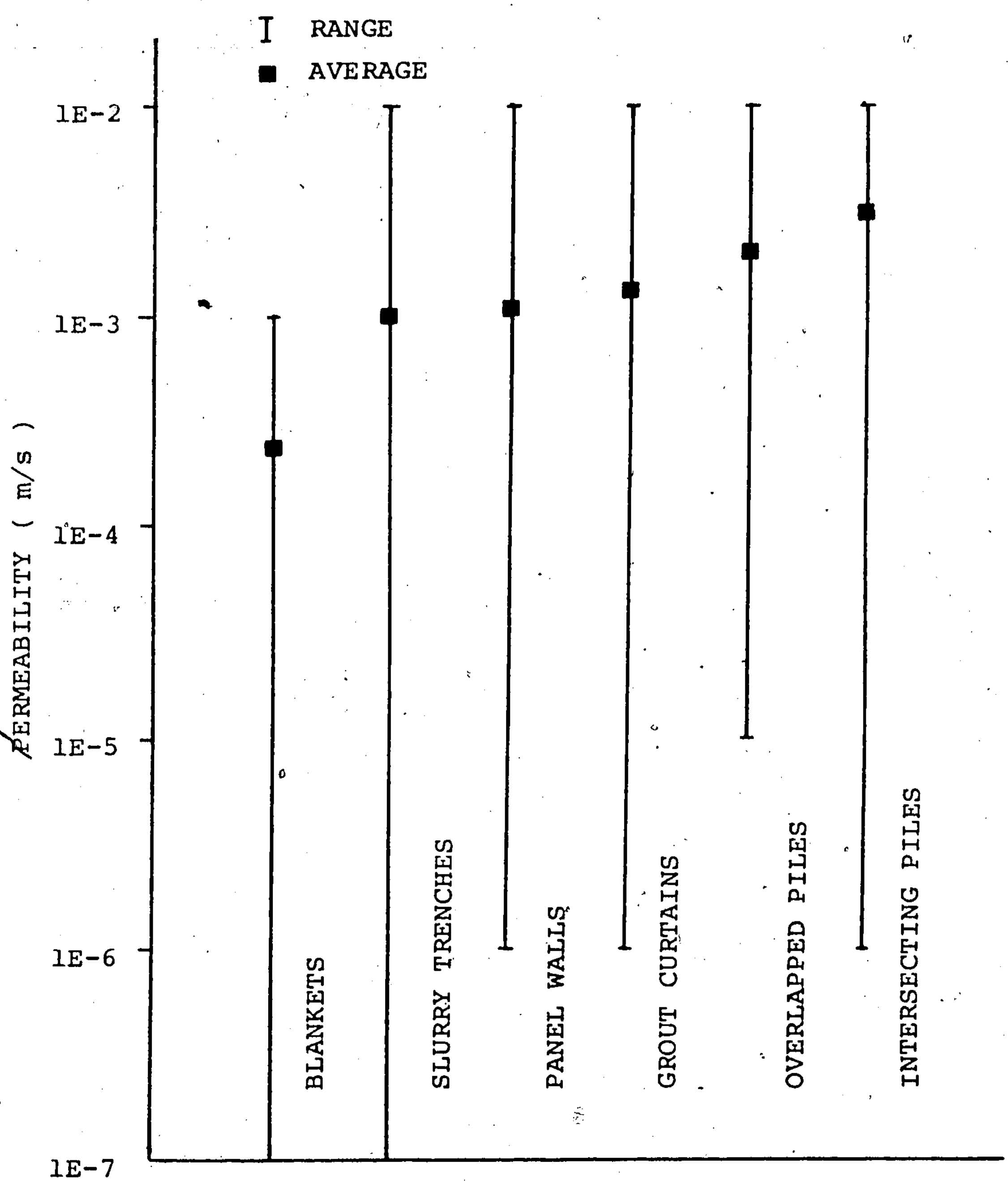
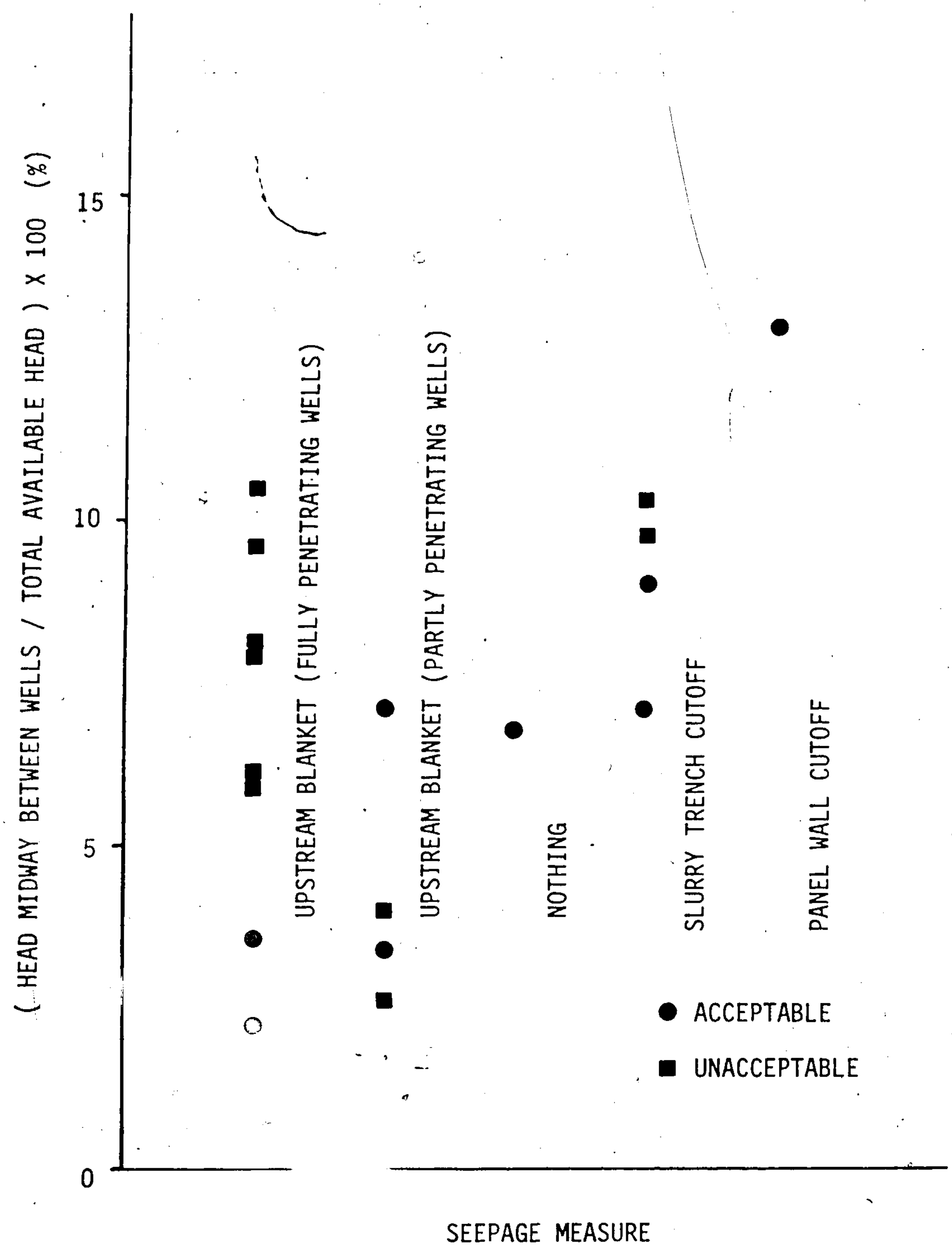


FIGURE 20 : PERFORMANCE OF RELIEF WELLS



APPENDIX A

SEEPAGE RECORDS

SUMMARY TABLES

TABLE A-1

ACCEPTABLE AND UNACCEPTABLE SEEPAGE DATA*[1]

DAM & LOCATION	SEEPAGE MEASURE	HYDRAULIC GRADIENT	SEEPAGE (cumecs)	SEEPAGE/ln m (cumecs/ln m)	ACCEPT-ABLE	TIME TO INCIDENT (years)	REFERENCES
Africa							
*Grou	u/s B, R.W.	.03	.04	8E-5	Y	-	Benisty & Tonnon (1970)
*Kainji	R.W.	.12	.006	-	Y	-	Umolu (1976)
Mogoto	CoW	-	.001	5E-6	N	52	Legge & Grobbelaar (1979)
Asia							
*Sarda Sagar	u/s B, R.W.	-	-	2.3E-4	N	2	ICOLD (1974)
May	-	-	3.4	8.1E-3	N	5	Alpsu (1967)
Dong Song	-	.18	.053	2.5E-5	N	-	Kim (1979)
*Funagira	G.C.	-	.07	1E-4	Y	-	Murakami & Hozumi (1982)
Europe							
Bila Desna	C.T.	.21	.004	-	N	1	ICOLD (1974)
Luhacovice	CoC	.11	.003	1.3E-5	Y		Krejci (1948)
Traryd	CoC	.07	.12 .04	6E-4 2E-4	N Y	1	Werner & Ljung (1948)
Lac Noir	C.W.	.20	.08 .0004	1.1E-3 5.3E-6	N Y	7	Ischy (1948)

TABLE A-1 (Continued)

ACCEPTABLE AND UNACCEPTABLE SEEPAGE DATA*[1]

DAM & LOCATION	SEEPAGE MEASURE	HYDRAULIC GRADIENT	SEEPAGE (cumecs)	SEEPAGE/ln m (cumecs/ln m)	ACCEPT-ABLE	TIME TO INCIDENT (years)	REFERENCES
Holleforsen	CoW	.29	.05	2E-4	Y		Westerberg et al. (1951)
	u/s F	.18	.6	9.3E-4	Y		Simek (1964)
San Valentino	C.D.	.15	.015	3.5E-5	Y		Italian Subcommittee (1964)
Vernago	C.D., G.C.	.13	.08	2.4E-4	Y		Italian Subcommittee (1964)
Gepatsch	C.T., G.C.	.28	.018	—	Y		Schober (1967)
Balderhead	CoC, G.C.	.09	.045 .005	4.9E-5 5.4E-6	N Y	1	Vaughan et al. (1970)
*Durl[illegible]sboden	G.C., R.W.	.26	.028	6E-5	Y		Kropatschek & Rienossl (1970) Rienossl & Schnelle (1976)
*Feistritz	C.P.	.26	3.2	1.28E-3	Y		Magnet & Mussnig (1970)
*Zoccolo	C.Pi	.27	.24 .12	4.7E-4 2.3E-4	N Y	10	Croce & Dolcetta (1970) Croce et al. (1979) Italian Subcommittee (1964)
*Bastusel	u/s B, G.C.	.12	.065 .018	8.1E-5 2.3E-5	N Y	1	Bernell (1976)
*Sylvenstein	G.C.	.16	.01 .001	8.3E-5 8.3E-6	N Y	11	Beier et al. (1979)
*Mattmark	G.C.	.28	.11	1.4E-4	Y		Fruhauf (1965) Gilg, et al. (1982)
*Eberlaste	C.P.	.15	.15	3.3E-4	Y		Kropatschek & Rienossl (1970) Rienossl & Schnelle (1976)

TABLE A-1 (Continued)

ACCEPTABLE AND UNACCEPTABLE SEEPAGE DATA*[1]

DAM & LOCATION	SEEPAGE MEASURE	HYDRAULIC GRADIENT	SEEPAGE (cumecs)	SEEPAGE/ln m (cumecs/ln m)	ACCEPT-ABLE	TIME TO INCIDENT (years)	REFERENCES
*Montta	CPi	.08	.006	-	Y		Sistonen (1967)
*Seitakorva	CPi	.34	1E-7	1E-9	Y		Sistonen (1967)
*Bohemia	u/s B	.09	.6	9.2E-4	Y		Simek (1964)
*Lossen	u/s B	.07	.6	4 E-4	Y		Emmelin & Welinder (1967)
*Serre-Poncon	G.C.	.15	.07	1.1E-4	Y		Barge et al (1964)
*Maria Al Lago	CPi	.23	.15	4.6E-4	Y		Edison Group (1961) et al.
*Kruth-Wildenstein	G.C.	.23	.096	3.6E-4	N		Corda et al. (1970)
			.026	9.6E-5	Y		
North America							
Montpelier Creek	C.T., G.C.	.15	.056	2.1E-4	N	1	ASCE (1975)
*Townshend Lake	u/s B	-	.064	1.2E-4	N	8	ASCE (1975)
			.021	4.E-5	Y		
Fontenelle	C.T., G.C.	-	.6	3.6E-4	N	1	ICOLD (1974), Bellport (1967)
Julesburg	u/s F	.09	.04	3.3E-5	N	5	ICOLD (1974)
Mill Creek	C.T.	-	.8	8.7E-4	N	6	ICOLD (1974)
Sinker Creek	CoC	.16	.014	4.2E-5	N	24	ICOLD (1974)
Wister	Nothing	.12	.53	2.0E-4	N	1	ICOLD (1974) Bertram (1967)

TABLE A-1 (Continued)

ACCEPTABLE AND UNACCEPTABLE SEEPAGE DATA*[1]

DAM & LOCATION	SEEPAGE MEASURE	HYDRAULIC GRADIENT	SEEPAGE (cumecs)	SEEPAGE/ln m (cumecs/ln m)	ACCEPT-ABLE	TIME TO INCIDENT (years)	REFERENCES
Netzahualcoyotl	G.C.	.20	.1	2.1E-4	Y		Instituto de Ingenieria (1976)
Dyke 1	C.T.	.10	.002	2.3E-6	Y		Instituto de Ing. (1976)
*Senator Wash	u/s B, C.T.	.03	1.4	2.0E-3	Y		Doming (1970)
Mica	G.C.	.20	.0036	4.5E-6	Y		Khilnan & Webster (1976)
Chicoasen	C.T.	.20	.30	-	Y		Moreno & Alberro (1982)
*D-20	S.T.	.16	0.2 .16	1.3E-4 1.1E-4	N Y	1	Pare et al. (1982)
*D-20	u/s B, R.W.	.06	0.1 .06	8.3E-5 5E-5	N Y	1	Pare et al. (1982)
Lake Patagonia	G.C., C.T.	.16	.0002 .00015	7E-7 5.4E-7	Y Y		Gizienski & Scott (1982)
GF-8	C.T.	.17	.106	3.9E-5	Y		Seemel & Colwell (1976)
GF-9	C.T.	.17	.044	1.3E-5	Y		Seemel & Colwell (1976)
GJ-12	C.T.	.17	.019	2.4E-5	Y		Seemel & Colwell (1976)
GL-7	C.T.	.17	.004	1.1E-5	Y		Seemel & Colwell (1976)
GL-8	C.T.	.17	.0004	1.8E-6	Y		Seemel & Colwell (1976)
*Camanche 2	u/s B	.07	.036	-	N		Anton & Dayton (1972)
*Arrow	C.P.	.27	.041	3.9E-4	Y		Gadsby & Bares (1968)

TABLE A-1 (Continued)

ACCEPTABLE AND UNACCEPTABLE SEEPAGE DATA*[1]

DAM & LOCATION	SEEPAGE MEASURE	HYDRAULIC GRADIENT	SEEPAGE (cumecs)	SEEPAGE/ln m (cumecs/ln m)	ACCEPT-ABLE	TIME TO INCIDENT (years)	REFERENCES
Yorba	-	-	.29	1.0E-3	N	45	ICOLD (1974)
Black Rock	C.T.	.18	.029	1.3E-4	N	2	ICOLD (1974)
Tieton	CoW	-	.0057	2E-5	Y		Niederhoff (1951)
Kachess	CoW	-	.02	4.7E-5	Y		Niederhoff (1951)
Denison	S.P.	.13	.14	2.7E-5	Y		Niederhoff (1951)
*Mohawk	u/s B	.08	.28	3.9E-4	N	32	Niederhoff (1951)
			.142	2.0E-4	Y		Coffman & Franks (1982)
Hardy	S.P.	.19	.14	1.5E-4	Y		Niederhoff (1951)
El Infiernillo	C.Pi, G.C.	.17	.56	1.6E-3	Y		Marsal & de Arellano (1966) Instituto de Ingerieria (1976)
*Seymour Falls	u/s B	.04	.4	8.8E-4	Y		Ripley & Campbell (1964)
Great Salt Plains	-	-	.006	3.3E-6	Y		Bertram (1967)
*Grenada	R.W.	-	.170	4E-5	N	1	Bertram (1967)
*East Branch	G.C.	.17	.28	5.3E-4	N	4.5	Bertram (1967)
			.007	1.3E-5	Y		
Requena	CoW	.27	.013	9.3E-5	N	16	Instituto de Ingenieria (1976)
*Francisco Zarco	S.T.	.17	.04	8.3E-5	Y		Instituto de Ingenieria, (1976) Gamboa et al. (1970)

TABLE A-1 (Continued)

ACCEPTABLE AND UNACCEPTABLE SEEPAGE DATA*[1].

DAM & LOCATION	SEEPAGE MEASURE	HYDRAULIC GRADIENT	SEEPAGE (cumecs)	SEEPAGE/ln m (cumecs/ln m)	ACCEPT-ABLE	TIME TO INCIDENT (years)	REFERENCES
*Manic 2	C.P. & CPi	.22	.152	9.8E-4	Y		Conlon & MacDonald (1967)
*El Infiernillo	CPi	.27	.0006	1.7E-6	Y		Marsal & de Arellano (1966) Inst. de Ing. (1976)
*Abelardo Rodriques	u/s B	.05	1.8	1.2E-3	Y		Marsal & Resendiz (1971)
*Dalles Closure	u/s B	-	3.6E-7	2E-9	Y		Brown (1961)
*Garrison	u/s B	.03	.063	1.7E-5	Y		Lane & Wohlt (1961)
*Hills Creek	u/s B	.15	.025	5.3E-5	N	9	Brown (1961)
			.002	4.2E-6	Y		Bertram (1967)
*Outardes 4	G.C.	.18	.0008	-	Y		Brown & Comeau (1970)
*Terzaghi	G.C.	.13	.085	2.6E-4	Y		Terzaghi & Lacroix (1964) Taylor (1969)
*Manic 5	CPi	.27	.055	2.3E-4	Y		Icos (1968)
South America							
*Paiva Castro	R.W.	.13	.0075	3.6E-5	Y		Massad & Gehring (1981)
El Yeso	-	.11	.32	9.1E-4	N	7	Larenas et al. (1982)
*Convento Viejo	C.P.	.20	1.4E-4	2.5E-8	Y		Alvarez et al. (1982)
*	G.C., C.T.	.17	.032	2.2E-4	N		Ruiz et al. (1976)
Saracurna	C.P.	.17	.012	8.6E-5	Y		

TABLE A-1 (Continued)

ACCEPTABLE AND UNACCEPTABLE SEEPAGE DATA*[1]

DAM & LOCATION	SEEPAGE MEASURE	HYDRAULIC GRADIENT	SEEPAGE (cumecs)	SEEPAGE/ln m (cumecs/ln m)	ACCEPT-ABLE	TIME TO INCIDENT (years)	REFERENCES
*Huinco	u/s B	.06	.53	6.6E-3	Y		Halter & Roa (1973)
*Sesquile	C.P.	.2	.07	2E-4	Y		Shuk et al. (1970)

* Dams Described in Appendix B.

*[1] For Legend of Table, See Table A-2.

TABLE A-2

LEGEND FOR TABLE A-1

HEADING	EXPLANATION
Dam & Location	Official Name of Dam Continent in which the dam is located
*	Star denotes dam discussed in detail within Appendix B.
Seepage Measure	Major seepage control and reduction measure incorporated into the design of the dam. Abbreviations are defined below: u/s B — Upstream Blanket RW — Relief Well CoW — Core Wall GC — Grout Curtain CT — Cutoff Trench CoC — Concrete Cutoff CW — Cutoff Wall u/s F — Upstream Facing CD — Concrete Diaphragm Wall CP — Concrete Panel Wall CPi — Concrete Pile Wall SP — Sheet Pile Wall ST — Slurry Trench Cutoff PC — Partial Cutoff
Hydraulic Gradient	Difference between the Forebay and Tailrace Water level divided by the base length of the dam (See Figure 12)
Seepage	Quantity of water recorded seeping through the dam's foundation.
Seepage/ln m	Quantity of water recorded seeping through the dam's foundation divided by the crest length of the dam.
Acceptable	If quantity of seepage was considered acceptable by the owner or reference author (Y-yes, N-no).
Time to Incident	Time to unacceptable performance in years after dam was completed.

APPENDIX B

CASE HISTORIES

SUMMARY TABLES

TABLE B-1

DAMS SUMMARIZED IN APPENDIX B

Number	Dam	Continent	References
1	Bjarnalaekur	Europe	Flygenring et al. (1976)
2	Brokopondo (cofferdam)	South America	Jones (1967)
3	Camanche #2	North America	Jones (1967) Anton & Dayton (1972)
4	D-20	North America	Pare et al. (1982)
5	Duncan	North America	Jones (1967) Duguid et al. (1971) Hindley et al. (1973)
6	Francisco Zarco	North America	Gamboa et al. (1970) Marsal & Resendiz (1971) Instituto de Ingerieria (1976)
7	Grahamstown	Australia	Hindley et al. (1973)
8	Kennewick	North America	Jones (1967)
9	Khancoban	Australia	Kotowicz (1967)
10	Mangla (Closure Dam)	Asia	Jones (1967)
11	Nechranice	Europe	Basta (1967)
12	Omatako	Africa	Jordaan et al. (1982)
13	Saylorville	North America	Jones (1967)

TABLE B-1 (Continued)

14	Wanapum	North America	Sherard et al. (1963) Jones (1967) Hindley et al. (1973)
15	Wells	North America	ENR (1965) Jones (1967)
16	West Point	North America	Jones (1967) Johnson (1968) Hindley et al. (1973)
17	Yards Creek	North America	ENR (1964) Jones (1967)
18	El Infiernillo (cofferdam)	North America	Marsal & de Arellano (1966) ICOS (1968) Marsal & Resendiz (1971) Instituto de Ingerieria (1976)
19	Manicouagan #2 (cofferdam)	North America	Conlon & MacDonald (1967) ICOS (1968) Wilson & Squier (1969)
20	Maria al Lago	Europe	Edison Group et al. (1961)
21	Melo	Europe	Korvenkontio (1970)
22	Montta	Europe	Sistonen (1967)
23	Selevir	Asia	Sezginer & Karacaoglu (1967)
24	Arrow (cofferdam)	North America	Gadsby & Bares (1968) Henry & Grant (1968) Wilson & Squier (1969) Dreville et al. (1970)

TABLE B-1 (Continued)

25	Bighorn	North America	Consedine (1972) Gorden & Rutledge (1972) Forbes et al. (1973)
26	Convento Viejo	South America	Alvarez et al. (1982)
27	D-20	North America	Pare et al. (1982)
28	Eberlaste	Europe	Kropatschek & Rienossl (1970) Rienossl & Schnelle (1976)
29	Feistritz	Europe	Magnet & Mussnig (1970)
30	Isola Serafini	Europe	Edison Group et al. (1961)
31	Kinzua	North America	Fuguay (1967) Wilson & Squier (1969) Dreville et al. (1970)
32	Manicouagan #3	North America	Pigeon (1974) Dascal (1979a) Dascal (1979b)
33	Obra	Asia	Garg & Agrawal (1967)
34	Peneos	Europe	Gofas (1965) Wilson & Squier (1969)
35	Saracuruna	South America	Ruiz et al. (1976)
36	Sesquile	South America	ENR (1963) Wilson & Squier (1969) Shuk et al. (1970)
37	Jose Maria Morelos	North America	Wilson & Squier (1969) de Alba & Gamboa (1970) Marsal & Resendiz (1971)

TABLE B-1 (Continued)

38	Manicouagan #5	North America	Galbiati (1963) Baribeau (1967) ICOS (1968) Wilson & Squier (1969) Dreville et al. (1970)
39	Seitakorva	Europe	Sistonen (1967)
40	Vodo	Europe	Edison Group et al (1961)
41	Zoccolo	Europe	Italian Sub-committee (1964) ICOS (1968) Croce & Dolcetta (1970) Croce et al. (1979)
42	Abelardo Rodriguez	North America	Marsal & Resendiz (1971)
43	Arrow	North America	Golder & Bazett (1967) Henry & Grant (1968)
44	Altinapa	Asia	Ural et al. (1967)
45	Bastusel	Europe	Bernell (1976)
46	Bohemia	Europe	Simek (1964)
47	Camanche #2	North America	Anton & Dayton (1972)
48	D-20	North America	Pare et al. (1982)
49	Dalles Closure	North America	Brown (1961)
50	Fort Randall	North America	Thorfinnson (1959) Lane & Wohlt (1961)
51	Garrison	North America	Seybold (1949) Lane & Wohlt (1961)

TABLE B-1 (Continued)

52	Gavins Point	North America	Lane & Wohlt (1961).
53	Grou	Africa	Benisty & Tonnon (1970)
54	Hills Creek	North America	Brown (1961) Bertram (1967) Asce (1975)
55	Huinco	South America	Halter & Roa (1973)
56	Losser	Europe	Emmelin & Welinder (1967)
57	Mohawk	North America	Niederhoff (1951) Coffman & Franks (1982)
58	Sarda Sarger	Asia	ICOLD (1974)
59	Senator Wash	North America	Doming (1970)
60	Seymour Falls	North America	Ripley & Campbell (1964)
61	Tarbella	Asia	Khan & Nagui (1970)
62	Townshend Lake	North America	Asce (1975)
63	Arbon	Europe	Franco & Laa Gomez (1970) Londe (1970)
64	Asen	Europe	Helot & Persson (1970)
65	Backwater	Europe	Geddes & Pradoura (1967)
66	Durlassboden	Europe	Kropatschek & Rienossl (1970) Reinossl & Schnelle (1976) Londe (1970)

TABLE B-1 (Continued)

67	East Branch	North America	Bertram (1967)
68	El Horcajo	South America	Pronsato & Zarazaga (1967) Londe (1970)
69	Funagina	Asia	Murakami & Hozumi (1982)
70	Girna	Asia	Londe (1970) Murti et al (1970)
71	High Aswan	Africa	Wafa & Labib (1967) Wilson & Squier (1969) Londe (1970)
72	Kruth-Wildenstein	Europe	Corda et al. (1970) Londe (1970)
73	Mattmark	Europe	Fruhauf (1965) Wilson & Squier (1969) Gilg (1970) Londe (1970) Gilg et al (1982)
74	Notre-Dame de Commiers	Europe	Bonazzi (1965) Wilson & Squier (1969) Londe (1970)
75	Outards 4 (cofferdam)	North America	Brown $ Comeau (1970) Londe (1970)
76	Serre-Poncon	Europe	Barge et al (1964) Wilson & Squier (1969) Londe (1970)
77	Sylvenstein	Europe	Lorenz (1967) Wilson & Squier (1969) Londe (1970) Beier et.al. (1979)

TABLE B-1 (Continued)

78	Terzaghi	North America	Terzaghi & Lacroix (1964) Taylor (1969) Wilson & Squier (1969) Londe (1970)
79	Kainji	Africa	Umolu (1976)
80	Grenada	North America	Bertram (1967)
81	Paiva Castro	South America	Massad & Gehring (1981)
82	Mactaquac	North America	Tawil & Watson (1976)

TABLE B-2

SLURRY TRENCH CASE HISTORIES*[1]

DAM*[2]	DATE	HEAD (m)	GRADIENT	SUBSURFACE CONDITIONS DEPTH (m)	SOIL TYPE	PERM (m/s)	S.R.M. DATA DEPTH (m)	WIDTH (m)	LOCATION	PERM (m/s)	ASSOC. MEASURE	EFFICIENCY PERFORMANCE
1	1970	10	.16	17	Sand	-	17	3.3	u/s	-	G.C.	Acceptable
2	1959	12.6	-	-	s,g	1 E-4	4.9	1.3	-	-	-	-
3	1968	13	.07	29	c,ø,s,g	1 E-5	29	2.4	u/s	-	u/s B	Acceptable 66% H
4	1978	17	.16	26	c,ø,s,g	1 E-3 1 E-6	26	1.5	CL	-	R.W.	Unacceptable 75% H
5	1967	32	.06	265	ø,s,g	1 E-4	23	3	u/s	-	R.W., u/s B	Acceptable 89%
6	1968	30	.17	125	s,g	1 E-4 kh/kv = 10	20	3	CL	1 E-6	Gallery	Acceptable 60% H
7	1969	7.7	.1	20	Sand	-	30	1.5	u/s	1 E-7 1 E-8	-	Acceptable
8	1952	4.9	-	-	ø,s,g	1 E-3	7	1.9	CL	-	-	-
9	1965	16	.09	75	Alluvium	-	4.5	1.8	u/s	1 E-9	C.T	Acceptable
10	1964	72.4	.26	18	s,g,b	1 E-3	18	3.3	CL	-	None	Acceptable

TABLE B-2 (Continued)

SLURRY TRENCH CASE HISTORIES*[1]

				SUBSURFACE CONDITIONS			S.R.M. DATA					
DAM*[2]	DATE	HEAD (m)	GRADIENT	DEPTH (m)	SOIL TYPE	PERM (m/s)	DEPTH (m)	WIDTH (m)	LOCATION	PERM (m/s)	ASSOC. MEASURE	EFFICIENCY & PERFORMANCE
11	1967	44	.1	350	Sand	1 E-4	31	3	u/s	-	G.C., R.W.	-
12	1982	8	.1	7	c,s	1 E-5 – 1 E-6	7	1.5	u/s	1 E-8	R.W.	Acceptable
13	1969	29.2	-	-	c,s,g	1 E-3	18	2.5	u/s	-	-	-
14	1962	27.8	.14	24	s,g	1 E-2	29	3.3	CL	1 E-9	G.C.	-
15	1964	16.8	-	-	Gravel	-	24	2.5	CL	-	-	-
16	1966	19.2	-	18	Alluvium	1 E-4 – 1 E-7	18	1.6	u/s	-	u/s B	-
17	1964	16.8	-	-	s,g,b	-	12	2.5	CL	-	-	-

*[1] For Legend of Table, See Table B-8

*[2] Refer to Table B-1

TABLE B-3

INTERSECTING PILE WALL CASE HISTORIES*[1]

DAM*[2]	DATE	HEAD (m)	GRADIENT	SUBSURFACE CONDITIONS			S.R.M. DATA				ASSOC. MEASURE	EFFICIENCY & PERFORMANCE
				DEPTH (m)	SOIL TYPE	PERM (m/s)	DEPTH (m)	WIDTH (m)	LOCATION	PERM (m/s)		
18	1963	20	.27	17	Alluvium	1 E-6	22	.6	CL	-	None	Acceptable
19	1963	25	.22	10	Alluvium, b	-	25	.76	u/s	-	None	Acceptable
20	1955	16	.23	40	Alluvium, b	-	41	.6	u/s	-	None	Acceptable
21	1971	19	.14	35	Fluvial	1 E-2 1 E-5	35	.5+.9	CL	-	None	-
22	1955	12	.08	40	Till + Fluvial		40	.6	CL	-	None	Acceptable
23	1962	30	.09	21	Alluvium	1 E-5	23	.62	u/s	1 E-7	u/s B	Acceptable 60% Q

*[1] For Legend of Table, See Table B-8

*[2] Refer to Table B-1

TABLE B-4

PANEL WALL CASE HISTORIES*[1]

DAM*[2]	DATE	HEAD (m)	GRADIENT	SUBSURFACE CONDITIONS DEPTH (m)	SOIL TYPE	PERM (m/s)	S.R.M. DATA DEPTH (m)	WIDTH (m)	LOCATION	PERM (m/s)	ASSOC. MEASURE	EFFICIENCY & PERFORMANCE
24	1967	35	.27	50	Alluvium	1 E-3 1 E-4	50	.76	CL	-	-	Acceptable
25	1972	86	.22	65	Alluvium, b	1 E-3 kh/kv = 100	66	.6	CL	-	u/s b	-
26	1980	34	.2	57	Alluvium	1 E-5	55	.8	CL	1 E-8	None	Acceptable 93.5% Q
27	1978	20	.17	67	s,g,b	1 E-3 1 E-5	67	.6	CL	-	-	Acceptable 95% H 100% Q
28	1968	24	.15	140	~~s~~,s,g,b	1 E-4 kh/kv = 10-100	53	-	CL	1 E-8	R.W.	Acceptable
29	1969	21.4	.26	100	Sand	1 E-2 1 E-4	47	.5	u/s	-	u/s b	Acceptable
30	1959	-	-	20	c,s	-	20	-	u/s	-	None	Acceptable

TABLE B-4 (Continued)

PANEL WALL CASE HISTORIES*[1]

DAM*[2]	DATE	HEAD (m)	GRADIENT	SUBSURFACE CONDITIONS DEPTH (m)	SOIL TYPE	PERM (m/s)	S.R.M. DATA DEPTH (m)	WIDTH (m)	LOCATION	PERM (m/s)	ASSOC. MEASURE	EFFICIENCY & PERFORMANCE
31	1965	38	.14	55	Outwash, b	1 E-3	55	.76	u/s	-	u/s B Drains	95 - 100%
19	1963	25	.22	10	Alluvium, b	-	25	.76	CL	-	None	Acceptable
32	1975	92	.12	52	Alluvium, b	1 E-4	52	.6	CL	-	PC	Unacceptable 70-92% H, 90% Q
33	1962	23.2	.17	25	Sand	1 E-4	26	.6	CL	-	R.W.	-
34	1965	38	.14	17	s,g	-	18	.6	u/s	-	None	-
35	1964	28	.17	5	s,g	1 E-6	33	.4	u/s	-	R.W., G.C.	Acceptable 60% Q
36	1963	30	.2	79	s,g,b	-	77	.55	u/s	-	None	Unacceptable 90% Q

*[1] For Legend of Table, See Table B-8.

*[2] Refer to Table B-1.

TABLE B-5

OVERLAPPED PILE WALL CASE HISTORIES*[1]

				SUBSURFACE CONDITIONS			S.R.M. DATA					
DAM*[2]	DATE	HEAD (m)	GRADIENT	DEPTH (m)	SOIL TYPE	PERM (m/s)	DEPTH (m)	WIDTH (m)	LOCATION	PERM (m/s)	ASSOC. MEASURE	EFFICIENCY & PERFORMANCE
37	1968	42	.12	80	Alluvium	1 E-2	88	.6	CL	-	-	Acceptable 88% H
32	1975	92	.12	122	Alluvium, b	1 E-4	122	.66	CL	-	PC	Unacceptable 82-91% H 90% Q
38	1964	70	.27	76	Alluvium, b	-	76	.6	CL	-	-	Acceptable
39	1960	24	.34	27	s,g	-	27	.5+.6	CL	-	None	Acceptable
40	1959	38	-	35	g+b	1 E-4 - 1 E-5	34.5	.55	u/s	-	-	Acceptable
41	1965	60	.27	100	Alluvium, b	1 E-4	55	.6	u/s	-	Drain	Unacceptable

*[1] For Legend of Table, See Table B-8.

*[2] Refer to Table B-1

TABLE B-6

UPSTREAM BLANKET CASE HISTORIES*[1]

DAM*[2]	DATE	HEAD (m)	GRADIENT	SUBSURFACE CONDITIONS: DEPTH (m)	SOIL TYPE	PERM (m/s)	S.R.M. DATA: THICKNESS (m) MIN	MAX	X_t	LENGTH (m) L	X_L	PERM (m/s)	ASSOC. MEASURE	EFFICIENCY & PERFORMANCE
42	1950	21.8	.05	80	Alluvium	1 E-3	3	6	3.6 7.3	300	14	-	Filters	Acceptable 58% Q
43	1968	24	.08	150	s,g	1 E-3	.9	5	20 25	550	23	-	None	-
44	1952	25	-	26	Alluvium	1 E-5	.5	1	25 50	350	14	1 E-9	None	-
45	1972	34	.12	25	Till, Alluvium	-	-	2	17	200	6	-	G.C.	Unacceptable
46	1961	20	.09	4.6	s,g	1 E-4 - 1 E-5	-	1	20	100	5	-	S.P.	Acceptable
47	1962	13	.07	29	c,ø,s,g	1 E-5	-	2.4	5.4	-	-	-	None	Unacceptable
48	1978	13	.06	80	ø,s,g	1 E-4 - 1 E-6	1.5	3	4 9	98	7.5	-	R.W.	Unacceptable
49	1957	122	-	25	s,g	-	-	-	-	76	.6	1 E-5	None	Acceptable
50	1953	37	.03	52	Alluvium	1 E-4	-	3	12	427	12	-	R.W.	Acceptable 50% H

TABLE B-6 (Continued)

UPSTREAM BLANKET CASE HISTORIES*[1]

DAM*[2]	DATE	HEAD (m)	GRADIENT	SUBSURFACE CONDITIONS DEPTH (m)	SOIL TYPE	PERM (m/s)	S.R.M. DATA THICKNESS (m) MIN	MAX	X_t	LENGTH (m) L	X_L	PERM (m/s)	ASSOC. MEASURE	EFFICIENCY & PERFORMANCE
51	1954	36	.03	30	ø,s,g	-	3	4.6	8 12	381	11	-	R.W., S.P.	Acceptable 50% - 60% H
52	1956	14	.05	43	s,g	-	-	-	-	137	10	-	R.W.	Acceptable 65% H
53	1968	17	.03	20	ø,s	1 E-4 1 E-7	1	5	3.4 17	450	26	-	None	Unacceptable
54	1961	84	.15	8	s,g	-	1.8	-	54	280	3	-	G.C. Gallery	Unacceptable
55	1970	12.5	.06	46	s,g,b	1 E-6	.5	3	4 25	150	12	1 E-7	P.C., CoC	Acceptable
56	1961	25	.07	50	ø,s,g	1 E-3	-	-	-	167	7	-	R.W.	Unacceptable
57	1937	19.7	.08	35	Drift	-	1.5	6	3.3 13	200	10	-	S.P., G.C.	Unacceptable
58	1961	12	-	-	Permeable	-	-	-	-	-	-	-	R.W.	Unacceptable
59	1965	22	.03	61	Fluvial	1 E-4	-	1	22	700	32	-	Well	Acceptable

TABLE B-6 (Continued)

UPSTREAM BLANKET CASE HISTORIES*[1]

DAM*[2]	DATE	HEAD (m)	GRADIENT	SUBSURFACE CONDITIONS DEPTH (m)	SOIL TYPE	PERM (m/s)	S.R.M. DATA THICKNESS (m) MIN	MAX	X_t	LENGTH (m) L	X_L	PERM (m/s)	ASSOC. MEASURE	EFFICIENCY & PERFORMANCE
60	1960	19	.04	210	ǥ,s,g	-	-	1.5	12.7	300	16	-	Well	acceptable
61	1976	138	.07	183	Alluvial	1 E-4	1.5	12	92 12	1432	10	-	R.W., Gallery, G.C.	Unacceptable
62	1961	39	-	-	Till	-	-	-	-	-	-	-	-	Unacceptable

*1 For Legend of Table, See Table B-8

*2 Refer to Table B-1

TABLE B-7

GROUT CURTAIN CASE HISTORIES*[1]

				SUBSURFACE CONDITIONS			S.R.M. DATA					
DAM*[2]	DATE	HEAD (m)	GRADIENT	DEPTH (m)	SOIL TYPE	PERM (m/s)	DEPTH (m)	WIDTH (m)	LOCATION	PERM (m/s)	ASSOC. MEASURE	EFFICIENCY & PERFORMANCE
63	1967	28	.14	40	Alluvium	1 E-4 – 1 E-6	40	-	CL	-	None	Acceptable
64	1963	20	.1	50	Alluvium	1 E-2 – 1 E-3	50	6	u/s	1 E-4 – 1 E-5	Drain	Acceptable
65	1967	46	.15	50	Alluvium + Till	1 E-4 – 1 E-6	65	15	CL	1 E-7	None	Acceptable
45	1972	34	.12	25	Till + Alluvium	-	25	-	u/s	-	u/s B	Unacceptable
66	1968	65	.26	50	Alluvium	1 E-4	75	24	CL	1 E-6 – 1 E-7	R.W.	Acceptable
67	1953	27	.17	25	Over-burden	-	49	2.4	CL	-	None	Unacceptable
68	1972	113	.09	140	Alluvium	1 E-4	150	-	u/s	1 E-6	R.W., u/s B	80% Q
69	1977	12	-	60	s,g,b	1 E-3 – 1 E-4	-	10 – .5	CL	1 E-6 – 1 E-7	None	Acceptable 76 to 96% H

TABLE B-7 (Continued)

GROUT CURTAIN CASE HISTORIES*[1]

DAM*[2]	DATE	HEAD (m)	GRADIENT	SUBSURFACE CONDITIONS DEPTH (m)	SUBSURFACE CONDITIONS SOIL TYPE	SUBSURFACE CONDITIONS PERM (m/s)	S.R.M. DATA DEPTH (m)	S.R.M. DATA WIDTH (m)	S.R.M. DATA LOCATION	S.R.M. DATA PERM (m/s)	ASSOC. MEASURE	EFFICIENCY & PERFORMANCE
70	1967	36	.12	24	s,g	1 E-5	7	10	CL	1 E-6	R.W.	Acceptable 81-94% H
71	1967	71	.07	225	Alluvium	1 E-3 1 E-5	225	40 5	CL	1 E-6	R.W.	Acceptable 59% H
72	1964	35	.23	14	Alluvium	-	23	6	u/s	-	Gallery, u/sF	Unacceptable
73	1967	110	.28	100	Alluvium + Till	1 E-3 1 E-6-	110	35 14	u/s	1 E-7	Gallery	Acceptable
74	1963	40	.16	50	Alluvium, b	1 E-2 1 E-4	55	15 6	u/s	1 E-6	-	Acceptable
75	1965	7	.18	23	Talus + Alluvium, b	-	30	6.3	CL	-	None	Acceptable
35	1968	28	.17	5	s,g	1 E-6	15	5	u/s	-	R.W.	Unacceptable
76	1960	122.5	.15	100	Alluvium	1 E-4	100	38 15.	CL	1 E-7	Gallery	Acceptable 98%

TABLE B-7 (Continued)

GROUT CURTAIN CASE HISTORIES*[1]

				SUBSURFACE CONDITIONS			S.R.M. DATA					
DAM*[2]	DATE	HEAD (m)	GRADIENT	DEPTH (m)	SOIL TYPE	PERM (m/s)	DEPTH (m)	WIDTH (m)	LOCATION	PERM (m/s)	ASSOC. MEASURE	EFFICIENCY PERFORMANCE
77	1958	34	.16	100	Alluvium	1 E-3	100	23 9	CL	1 E-6	u/s B, G.C., P.C.	Unacceptable
78	1960	55	.13	158	Alluvium + clay	1 E-3	150	18	u/s	1 E-6	R.W., S.P. Drains	Acceptable 90%H

*[1] For Legend of Table, See Table B-8.

*[2] Refer to Table B-1.

TABLE B-8

LEGEND FOR TABLES B-2 to B-7 & B-9

HEADING	EXPLANATION
Dam	Number of dam, refer to Table B-1 for name of dam, location and respective references.
Date	The year when the dam was completed and ready for use.
Head	Difference between the forebay and tailrace level.
Gradient	The hydraulic gradient, is the Head divided by the base length of the dam.
Subsurface Conditions	Information with respect to the foundation soils.
Depth	The depth of soil beneath the dam to bedrock.
Soil Type	The description of the soil with respect to grain size. c - clay g - gravel ø - silt b - boulders s - sand
Perm	The permeability of the foundation soil.
S.R.M. Data	Seepage Reduction Measure Data
Depth	The depth to which the cutoff extends.
Width	The width or diameter of the cutoff.
Location	The position of the cutoff beneath the dam. (CL - centreline, u/s upstream, See Figure 3)
Perm	Permeability of the seepage reduction measure.
Thickness	Thickness of upstream blanket.

TABLE B-8 (Continued)

min	Minimum thickness of upstream blanket.
max	Maximum thickness of upstream blanket.
$\frac{X}{t}$	Thickness of the upstream blanket divided by the head.
Length (L)	Length of the upstream blanket from the upstream toe of the dam.
$\frac{X}{L}$	Length of the upstream blanket divided by the head.
Spacing	Distance between wells.
Penetration	The percentage to which the well was extended into the alluvium.
Radius	Inside radius of well.
Number	Total number of wells.
Assoc. Measure	The seepage control and/or reduction measure used in conjunction with the major measure. Refer to Table A-2 for abbreviations.
Efficiency & Performance	The observed percent efficiency and behaviour of the seepage reduction measure. Q - efficiency based on quantity of seepage H - efficiency based on head loss

TABLE B-9

RELIEF WELL CASE HISTORIES*[1]

DAM*[2]	HEAD (m)	GRADIENT	SUBSURFACE CONDITIONS DEPTH (m)	SOIL TYPE	PERM (m/s)	WELL DATA SPACING m	PENETRATION (%)	RADIUS (m)	NUMBER	ASSOC. MEASURE	PERFORMANCE
4	17	.16	26	c,ø,s,g	1 E-3	45-60	100	.038	8	S.T.	Poor
					1 E-6	15-30	100	.038	14		Good
28	24	.15	140	ø,s,g,b	1 E-4	25	40	.063	15	C.P.	Good
36	30	.2	79	s,g,b	-	10-60	-	.019	9	C.P.	Poor
						50		.026	8		Poor
48	13	.06	80	c,ø,s,g,b	1 E-3	12-24	100	.038	-	u/s B	Poor
					1 E-6						
50	37	.03	52	Alluvium	1 E-4	21	60	.051		u/s B	Good
						30	100	.051	36		
51	36	.03	30	ø,s,g	-	61	80	.051	54	u/s B, S.P.	Good
52	14	.05	43	s&g	-	-	70	-	48	u/s B	Good
53	17	.03	20	Alluvium	1 E-4	-	100	.016	20	u/s B	Good
					1 E-7						
56	25	.07	50	Alluvium	1 E-3	50-100	100	.075+.1	-	u/s B	Poor
57	19.7	.08	35	Alluvium	-	30	100	.063	14	u/s B	Poor
						15	100	.063	7		Good

TABLE B-9 (Continued)

RELIEF WELL CASE HISTORIES*1

DAM*2	HEAD (m)	GRADIENT	SUBSURFACE CONDITIONS DEPTH (m)	SOIL TYPE	PERM (m/s)	WELL DATA SPACING m	PENETRATION (%)	RADIUS (m)	NUMBER	ASSOC. MEASURE	PERFORMANCE
58	12	-	-	-	-	15	-	.013	-	u/s B	Poor
61	138	.07	183	s,g,b	1 E-4	15	30 to 50	.039		u/s B	Poor
70	36	.12	24	Alluvium	1 E-5	15-20	100	-	18	G.C.	Good
79	8	.12	18	c & s		61	100	.007	18	u/s B	Poor
80	18	-	-	Sand	-	15	-	.038	-	-	Poor
81	15	.13	-	Sand	1 E-3 – 1 E-5	-	75	-	-	Filters	Good
82	35	.09	35	Till & s	1 E-3	36.5	100	.038	6	None	Good

*1 See Table B-8 for a description of this table.

*2 Refer to Table B-1