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Geological Survey of Victoria

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DEPARTMENT OF INDUSTRY, TECHNOLOGY AND RESOURCES

ENGINEERING GEOLOGY OF MELTON -
ENGINEERING GEOLOGY

P G DAHLHAUS

UNPUBLISHED REPORT 1986/5

ABSTRACT

An engineering geological mapping program has been conducted to provide essential geological information for use by planners and engineers working in the Melton Development Area, Victoria.

A review of past and current examples of thematic mapping for land use purposes was initially conducted. A data base of over 800 sampled locations was collated from previous work, and supplemented by additional drilling and testing in areas where little was known of the geological materials. This information was compiled using available computer facilities and combined with traditional field mapping methods. A map folio presenting individual aspects of the engineering geology was produced.

Large areas of expansive soil have been identified and mapped, and an area affected by soil subsidence was examined in detail. Statistical methods (block kriging) have been used to determine the thickness of soil in the map area. Assesments of the suitability for urban development have been made.

Computer draughting was used to produce the maps, providing the ability for rapid future revision.

This report, which is one of seven unpublished reports on the map area, describes the engineering geology.

KEYWORDS: Engineering Geological Mapping, Medium Scale, Soils, Maps, Engineering Geology, Engineering Soils, Kriging, Urban Planning, Hydrogeological Maps

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LIST OF ABBREVIATIONS used in reporting the Melton Engineering Geology Mapping Project.

Abbreviation	Definition
AASHO	American Association of State Highway Officials
AEBIRA	Australian Engineering and Building Industry Research Association
AHD	Australian Height Datum
AMG	Australian Metric Grid
AS1276	Australian Standard AS1276 - SAA Site Investigation Code
AS1289	Australian Standard AS1289 - Methods of Testing Soils for Engineering Purposes
AS2870	Australian Standard AS2870 - Residential Slabs and Footings
CAD	Computer Aided Drafting/Design
CBD	Central Business District
CBR	California Bearing Ratio
CRB	Country Roads Board
CSIRO	Commenwealth Scientific and Industrial Research Organisation
DITR	Department of Industry, Technology and Resources
DOE	Department of Environment
DVA	Dandenong Valley Authority
ECS	Engineering Computer Services Pty. Ltd.
EDP	Electronic Data Processing
EPA	Environment Protection Authority
F&L	Farley and Lewers Pty Ltd
FAO	Food and Agriculture Organisation
FS	Free Swell
GEOSIS	Geoscience Spatial Information System
GLQ	Genesis-Lithology-Qualifier
GSV	Geological Survey of Victoria
IAEG	International Association of Engineering Geology
IGS	Institute of Geological Sciences
LL	Liquid Limit
LPS	Land Protection Service
LS	Linear Shrinkage
MMBW	Melbourne Metropolitan Board of Works
MPE	Ministry for Planning and Environment
MSA	Melton Sewage Authority
MSICC	Melton - Sunbury Interim Co-ordinating Committee
MURL	Melbourne Underground Rail Loop
OGS	Ontario Geological Survey
PL	Plastic Limit
PI	Plasticity Index
RCA	Road Construction Authority
SAA	Standards Association of Australia
SCA	Soil Conservation Authority
SCS-USDA	Soil Conservation Service - United States Department of Agriculture
TDS	Total Dissolved Solids
UBR	Uniform Building Regulations
ULA	Urban Land Authority
USGS	United States Geological Survey
VBR	Victorian Building Regulations
WHO	World Health Organisation
XRD	X-Ray Diffraction

INTRODUCTION

The City of Melton is located on the Western Highway 39 km WNW of Melbourne and was chosen by the Victorian Government for satellite township development in 1973.

The Melton Engineering Geological Mapping Project commenced in March 1983, as part of an ongoing mapping scheme conducted by the Geological Survey of Victoria (GSV), now a branch of the Department of Industry, Technology and Resources (DITR). The project aims at the production of a map (or maps) depicting relevant geological features and properties in a useful manner for engineers and planners working in the Melton Development Area.

An engineering geological map is a thematic map which provides a generalized representation of all those components of a geological environment of significance in land-use planning, and in design, construction and maintenance as applied to civil engineering.

A 'state-of-the-art' review of mapping methods for land-use planning was conducted to examine the past and present progress in a broad context. In particular, medium-scale engineering and environmental mapping methods, and their map presentation formats, were examined.

A review of readily accessible data highlighted shortcomings in both the quality and quantity of data outside of the established City of Melton. Consequently, a drilling, sampling and testing program was conducted. Research of previous work and additional geological mapping supplemented the data analysis. The presentation of the study has been largely cartographic, with each component of the geology being a separate theme on a basic map.

Seven reports have been produced in the GSV Unpublished Report series:

Unpublished Report 1986/1
Engineering Geological Mapping - A Review

Unpublished Report 1986/2
Engineering Geology of Melton - The Melton Development Area

Unpublished Report 1986/3
Engineering Geology of Melton - Drilling, testing and mapping program

Unpublished Report 1986/4
Engineering Geology of Melton - Geology and geomorphology

Unpublished Report 1986/5
Engineering Geology of Melton - Engineering geology

Unpublished Report 1986/6
Engineering Geology of Melton - Map presentation of data

Unpublished Report 1986/7
Engineering Geology of Melton - Summary

ENGINEERING GEOLOGY

1 Isopleth Mapping of Soil Thickness

Thickness is a soil property of importance to urban development at Melton. The depth of soil cover over the basalt has influence on the cost of water and sewerage supply, the foundation stability of light structures, and road making costs.

Isopleth maps may be used in engineering geological mapping to depict classes of soil, based on similarities of soil properties. It is implied that areas mapped as a particular class will have values of soil properties similar to those recorded for that class, and different from those of at least some of the other classes. Isopleth mapping is often referred to as 'contouring', by analogy with the mapping of topographic height. However, caution is required, as topographic contours are usually drawn to join points of equal measured height, whereas isopleths join points of inferred equal value. In practice topographic contours can be followed continuously, either on the ground or on a pair of aerial photographs, with the result that they can be drawn as accurately as the surveying equipment allows. Soil isopleths on the other hand must be derived from a set of more or less widely spaced points and are therefore subject to sampling variation.

Isopleths maps of soil thickness may be computed from borehole and test-pit observations by utilising computer packages.

1.1 Numeric surfaces

When mapping soil properties, the distribution of a single property may be displayed by assigning to each class on the map the typical value of that property within its class. The value at any one place is not actually recorded - unless there is a sample point (i.e. a borehole or test-pit) - it is predicted. It is realised that the actual value there will differ from the predicted value. The statistical rationale can be expressed as:

$$z_{ij} = \mu + \alpha_j + \epsilon_{ij} \qquad (1)$$

where z_{ij} the value of a property at any place i in class j is the sum of three terms:

μ the general mean of the property for the whole area;
α_j is the difference between the general mean and the mean of class j ; and
ϵ_{ij} is a random component distributed normally with zero mean and variance σ_w^2 .

The parameters μ , α_j , σ_w^2 and can all be estimated from data as say $\bar{z}$, a_j and s_w^2 respectively by the least squares analysis and analysis of variance. The predicted value for an unrecorded point in class j is $\bar{z} + a_j$, and confidence limits are determined from s_w^2 , the sample within-class variance. The smaller is σ_w^2 more precise will any prediction be, and the more valuable the map.

Where measured data are sparse, as they often are, this approach to prediction and mapping is the only feasible one. It obviously depends on there being an association between the property of interest and the classification, even though the classes are recognized independently in terms of the model, $|\alpha_j|$ must on average be substantially greater than zero, otherwise the classification does not help to predict the property. However the procedure takes no account of the spatial arrangement of the data points and their relations to predicted points, nor of any gradation of values across boundaries. These can only be of consequence when data are dense, specifically when they are spatially dependant, and in that event a means of prediction and mapping that uses the spatial information is obviously to be preferred.

In such circumstances interpolation provides an alternative to classification for predicting values of a property at unvisited points. Mapping can be achieved by envisaging such values as forming a continued statistical surface over the map plane, which can be represented by isopleths.

1.2 Interpolation techniques

Several interpolation techniques are available, especially in computer programs for automatic contouring. Webster (1977) has breifly reviewed some of these viz: linear interpolation across triangulation, inverse square distance weighted averaging, least square polynomials, Theisson polygons, and kriging.

Apart from linear interpolation across triangles, most of these techniques consist of placing a fine, rectangular (or square) mesh over the entire area and computing a new value of the property at each mesh point. This process is suited to various forms of processing such as smoothing, filtering and fourier transformations. The size of the mesh interval will govern the frequency content of the resultant model and thus the isopleth map detail. Both the type of data and the desired type of map influence the choice of mesh interval and the choice of interpolation technique.

In using weighted interpolation techniques (such as kriging), three steps are involved.

1 Choosing the number of data points in the vicinity of the mesh point being considered. The selection algorithm may locate the closest 'n' points in 'x' number of sectors surrounding the mesh point normally within a certain distance.

2 Determine whether the chosen points are adequate in both number and in distribution.

3 Interpolate the mesh value by taking a weighted average of the data values. The weight is a function of the assigned value, the distance between the data values and an optional smoothing operator distance.

Grids generated by computer packages can be manipulated to:
- limit the grid to a desired polygon,
- expand to fill a given area,
- mask out (or in) any areas,
- take into account faults, discontinuities, and trends, fill
- be scaled, added, subtracted, multiplied, divided, or restricted to positive values by logarithmic gridding.

The algorithm which chooses data points is important in that the number of points for interpolation plays a significant role. Where the data is suited, kriging has an advantage over other methods which often represent compromises between the mathematically desirable and the computationally feasible. Though these other methods are reasonable for many applications they may give biased interpolation, whilst they provide no estimate of the error of interpolation, nor do they attempt to minimise that error.

1.3 Kriging

Kriging is a form of weighted local averaging that is an optimal means of spatial prediction in the sense that it provides estimates of values at unrecorded places without bias and with minimum and known variance. It is based on the theory of regionalised variables developed by Matheron (1963) and Krige (1966) for the estimation of ore reserves in mining.

1.3.1 Variograms

Kriging depends on first computing an accurate semi-variogram, which measures the nature of spatial dependance for the property. Estimates of semi-variance are then used to determine the weights applied to the data when computing the averages, and are presented in the kriging equations.

The semi-variance is expressed as:

$$\gamma(h) = \tfrac{1}{2} VAR\left[z(i) - z(i+h)\right] = \tfrac{1}{2}\sum\left[z(i) - z(i+h)\right]^2 / n \qquad (2)$$

and is a measure of the similarily, on average, of an observation z at point i and another point at a given distance h away. In other words, the semi-variance is the average half-squared difference between all pairs of points separated by the same distance, h. The quantity $\gamma(h)$ can be estimated for integer values of h from the data and the graph of $\gamma(h)$ versus h is the semi-variogram.

The semi-variogram has certain important characteristics which (a) reveal the nature of the geographic variation in the property of interest, and (b) are needed to provide kriged estimates at previously unrecorded points. These are described in reference to Figure 1.

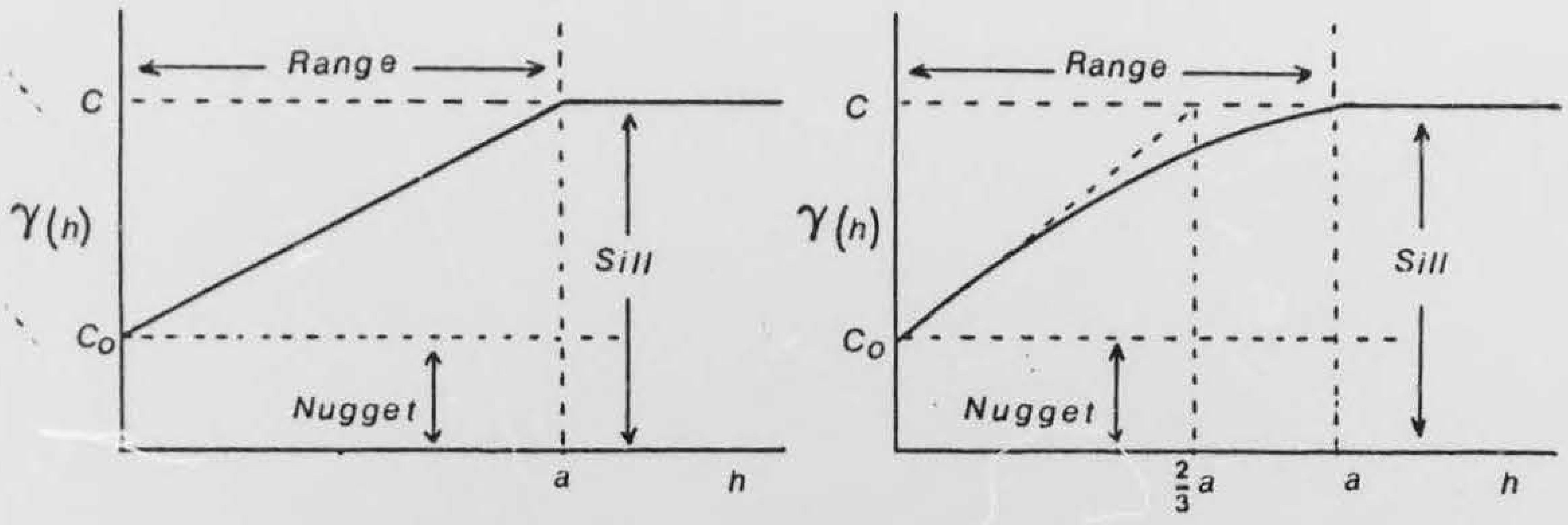


Figure 1. Theoretical Variograms.

In most cases it is found that $\gamma(h)$ increases with increasing h to a maximum, approximately the variance of the data, at a moderate value of h, say a. The distance is known as the range. Points closer than the range are spatially dependant; points further apart bear no relation to one another, unless there is a periodic variation in the soil. When interpolating, the aim is to use only those points closer than the range to the predicted point.

By definition $\gamma(h) = 0$ when $h = 0$. However, in practice, any smooth curve that approximates the values of the semi-variance is unlikely to pass through the origin. Instead there appears to be a positive finite value to which $\gamma(h)$ approaches as h approaches 0. This intercept is known as the nugget variance, and in general is known as the nugget effect. The terms derive from sampling practice in gold mining where the inclusion of a gold nugget in narrow core is a somewhat chance event. The nugget effect accounts for different results in sampling the same site twice. This may occur for reasons such as poor analytical precision, poor sampling practice, or actual erratic values at low scale. Most semi-variograms of soil properties show nugget effects (Burgess and Webster, 1980a). The nugget variance embraces fluctuation in the soil that occurs over distances much shorter than the sampling interval, and limits the precision of interpolation.

The value at which $\gamma(h)$ levels out is known as the sill. It represents the range of variance due to spatial dependence in the data.

There is no general mathematical formula to describe the shape of soil semi-variograms. A linear model, $\gamma(h) = C_0 + mh$, is simplest, and will often describe $\gamma(h)$ well within range. A spherical model, given by

$$\begin{cases} \gamma(h) = C_0 + C\,\frac{3}{2}\frac{h}{a} - \frac{1}{2}\left(\frac{h}{a}\right)^3 & \text{for } 0 < h \leq a \\ \gamma(h) = C_0 + C & \text{for } h > a \end{cases} \qquad (3)$$

may also be used. Other models (De Wysian, exponential, ah^{λ} and hole effect) are described by David (1977).

1.3.2 Simple kriging

When estimating a value $z(X_0)$ of a property z at point(X_0) where X is the vector notation (i.e. $X = [x, y]$), the linear sum, or weighted average, of the observed value is expressed as:

$$z_0 = \lambda_1 z(X_1) + \lambda_2 z(X_2) + \ldots\ldots\ldots\ldots \lambda_n z(X_n). \qquad (4)$$

where the λ are coefficients or weights associated with the data points, as discussed in section 1.2.

In kriging, the weights are so chosen that the error associated with the estimate is less than for any other linear sum. The weights take account of the known spatial dependences expressed in the semi-variogram and the geometric relationships among the observed points. In general, near points carry more weight than distant points, points that occur in clusters carry less weight than lone points, and points lying between the point to be interpolated and more distant points screen the distant points in that the latter have less weight than they would otherwise.

The model for simple kriging, analogous to equation (1), section 1.1, for usual survey practice, is:

$$z(X) = \mu_v + \epsilon(X) \qquad (5)$$

where $z(X)$ is the value of the property at X within a neighbourhood v, μ_v is the mean value in that neighbourhood and $\epsilon(X)$ is a spatially dependent random component with zero mean and variation defined by:

$$VAR\left[\epsilon(X) + \epsilon(X + H)\right] = E\left[\left\{\epsilon(X) + \epsilon(X + H)\right\}^2\right] = 2\gamma(H) \qquad (6)$$

and equals $2\gamma(h)$ if variation is isotropic. It is assumed that μ_v is constant for the neighbourhood, though different neighbourhoods may have different means, and that the semi-variogram is the same over the whole area. The last assumption implies that there are no sharp boundaries (faults, cliffs). If such boundaries are known to exist then interpolation is carried out separetely on either side.

The coefficients (or weights) are calculated using the equations:-

$$\sum_{j=1}^{n} \lambda_j \gamma(X_i, X_j) + \mu = \gamma(X_i, X_0) \quad \text{for } i = 1,2,3,\ldots,n \qquad (7)$$

where μ is a Lagrange multiplier

The matrix notation is given by : $\begin{bmatrix} \lambda \\ \mu \end{bmatrix} = A^{-1}B$

$$A = \begin{bmatrix} \gamma(X_1,X_1) & \gamma(X_2,X_1) & \gamma(X_3,X_1) & 1 \\ \gamma(X_1,X_2) & \gamma(X_2,X_2) & \gamma(X_3,X_2) & 1 \\ \vdots & \vdots & \vdots & \\ \gamma(X_1,X_n) & \gamma(X_2,X_n) & \gamma(X_n,X_n) & 1 \\ 1 & 1 & 1 & 0 \end{bmatrix} \quad B = \begin{bmatrix} \gamma(X_1,X_0) \\ \gamma(X_1,X_0) \\ \vdots \\ \gamma(X_n,X_0) \\ 1 \end{bmatrix} \tag{8}$$

$$\begin{bmatrix} \lambda \\ \mu \end{bmatrix} = \begin{bmatrix} \lambda_1 \\ \lambda_2 \\ \vdots \\ \lambda_n \\ \mu \end{bmatrix} \tag{9}$$

The minimum estimation variance is σ_E^2 given by:

$$\sigma_E^2 = B^T \begin{bmatrix} \lambda \\ \mu \end{bmatrix} \tag{10}$$

The accuracy of kriged estimates depends on the goodness of the computed semi-variogram and two precautions are taken to ensure that the values of $\gamma(h)$ used in the kriging equations are satisfactory. First, the spatial analysis should be performed on long runs of data (or a number of short runs), so that the semi-variances at short lags can be computed from many pairs of comparisons. Second, a sensible model must be chosen to describe the results, and individual estimates of $\gamma(h)$ can be weighted according to the number of comparisons on which the they are based when fitting the model.

An example of simple kriging (Farrelly, 1985) is shown in Appendix I

1.3.3 Block kriging

In simple kriging, the grid points at which we make estimates represent volumes with the same size and shape as the volumes of soil from which the original property was measured. For example, if observations are derived from 10cm diameter cores, then the estimated grid points are strictly cylinders 10cm diameter. If the observations were test pits, then the computed grid points would also represent test pits. Although sampling is carried out in this fashion for convenience, economics and time, the observation at a single sample point is usually taken by the observer to represent the surrounding area, or at least the area nearer to it than any other sample point. When interpolating the geologist may wish to interpolate an average value for an area or block many times larger than the actual sampled volume.

Kriging can be carried out over areas, in a procedure known as block kriging. In block kriging, instead of considering a point X, we consider a region ϑ with an area H with its centre at X.

The semi-variances between data points and the interpolated point are replaced by the average semi-variances between the data points and all the points in the region. Thus each $\gamma(X_i, X_0)$ of equation 9 is replaced by the integral $\int \gamma(X_i\ X)\, p(x) d(x)$ where $p(x)$ is given as follows:

$$p(x) = \frac{1}{H_\vartheta} \quad \text{if } X \text{ belongs to } \vartheta \qquad (11)$$

$$p(x) = 0 \quad \text{otherwise,}$$

and
(12)

$$\int p(x)\, d(x) = 1$$

The weights for block kriging are therefore given by

$$\begin{bmatrix} \lambda \\ \mu \end{bmatrix} = A^{-1} S \qquad (13)$$

where

$$S = \begin{bmatrix} \int \gamma(X_1, X)\ p(x) d(x) \\ \int \gamma(X_2, X)\ p(x) d(x) \\ \vdots \quad \vdots \quad \vdots \\ \int \gamma(X_n, X)\ p(x) d(x) \\ 1 \end{bmatrix} \qquad (14)$$

The estimated variance for the area H is

$$\sigma_H^2 = S^T \begin{bmatrix} \lambda \\ \mu \end{bmatrix} - \iint \gamma(X, Y)\, p(x)\ p(y)\ d(x)\ d(y) \qquad (15)$$

Although a map drawn from point estimates is the more accurate isopleth map, local minor variation can obscure regional trends. Block kriging results in a smoother map showing average values calculated over a number of broader areas.

1.3.4 Universal kriging

A third means of kriging, universal kriging, takes into account local trends in data when minimising the error associated with estimation. The presence of such trends or drifts is identified qualitatively, and their form found quantitatively by one of two methods. Either (1) a structural analysis may be carried out, which simultaneously estimates semi-variances of the differences between the drift and the actual data. The resulting semi-variograms are then used for the interpolation. Or (2) prior generation of a regional surface and semi-variograms are calculated for the residuals. Simple kriging is then used to produce the numeric surface.

Universal kriging is not comprehensively applicable to soil survey (Webster and Burgess, 1980), mainly because of the large nugget variances usually encountered, which arise in part because measurements are made on small widely separated volumes of soil. These effectively prevent any distinction between constant and changing drift.

Universal kriging would not be applicable to the data obtained at Melton for these reasons.

1.4 Soil thickness mapping - Definitions and parameters

Soil thickness, like many geological parameters, has a certain amount of subjective judgement incorporated into a definition. Past work in the area (MacIsaac and Key, for the Melton Sewage Authority, 1972) has set a definition of soil thickness as being the depth to powered-auger refusal. A map depicting "rock contours" was produced and used as a guide in the drafting of tenders for the installation of sewerage and storm-water pipelines. Since cost of excavation increases rapidly when basalt is encountered, such guides are valuable.

In the engineering geological sense, soil is defined by "all unconsolidated materials above bedrock" (Bates and Jackson, 1980). This definition includes cobbles and boulders ('floaters') surrounded by soil which are common in residual basaltic soils and such floaters are sufficiently large and unweathered to cause powered-auger penetration refusal when encountered. Thus, power-auger refusal may be a misleading definition for soil depth, although the only feasible one. This in turn, results in a variation soil depth over short distances (which accounts for the large nugget effects in the semi-variograms).

Where a sample point represents an excavated test-pit, the soil depth is taken as depth to excavator refusal. If the sample point represents a borehole drilled by a percussion or diamond drill, then the soil depth is judged from the borelogs as being the top of the first encountered rock which would refuse penetration by a powered-auger.

Sample points where rock was not encountered and points where penetration refusal occured on alluvial gravels or calcareous nodules were not included in the computations. However, they were used in checking the accuracy of the isopleth map on completion.

Approximately 1100 sample points (boreholes and test-pits) were available for scrutiny within the map area. Of these, 648 were selected into the data base which formed the basis for the soil depth computations. The selected points represented those for which a positive soil depth value could be given.

Since the data were collected from several sources, the quality varied according to the origin. For some of the data the locations were approximate, as the coordinates were scaled from locality diagrams included in reports. For other data the sample locations were surveyed.

The soil depth values were generally precise. The only exception was where data from percussion drilling was included - the depths tended to correspond to the length of a drill-rod (i.e. 1.50m, 3.00m, etc.) in most of the locations. However, for one metre isopleths, this data remains valid.

The quality of the data is tabulated below.

Source	Number of data points	Location technique	Estimated error (metres)	Drilling method	Estimated soil depth error (metres)
GSV	184 24	Surveyed Scaled	Nil 5	Auger + pit Unknown	Nil 1
MSA	207	Scaled	5	Auger + diamond	Nil
RCA	105	Scaled	2	Auguer + pit + diamond	Nil
F&L	105	Scaled	2	Percussion Diamond	0.5 Nil
Other	23	Scaled	20	Various	0.5

Table 1. Soil Depth Data Quality.

The distribution of data (Fig. 2) presents a challenging difficulty in numeric surface calculations. The 'clumping' of the sample points makes gridding difficult, because the uneven distribution makes the selection of mesh size a problem.

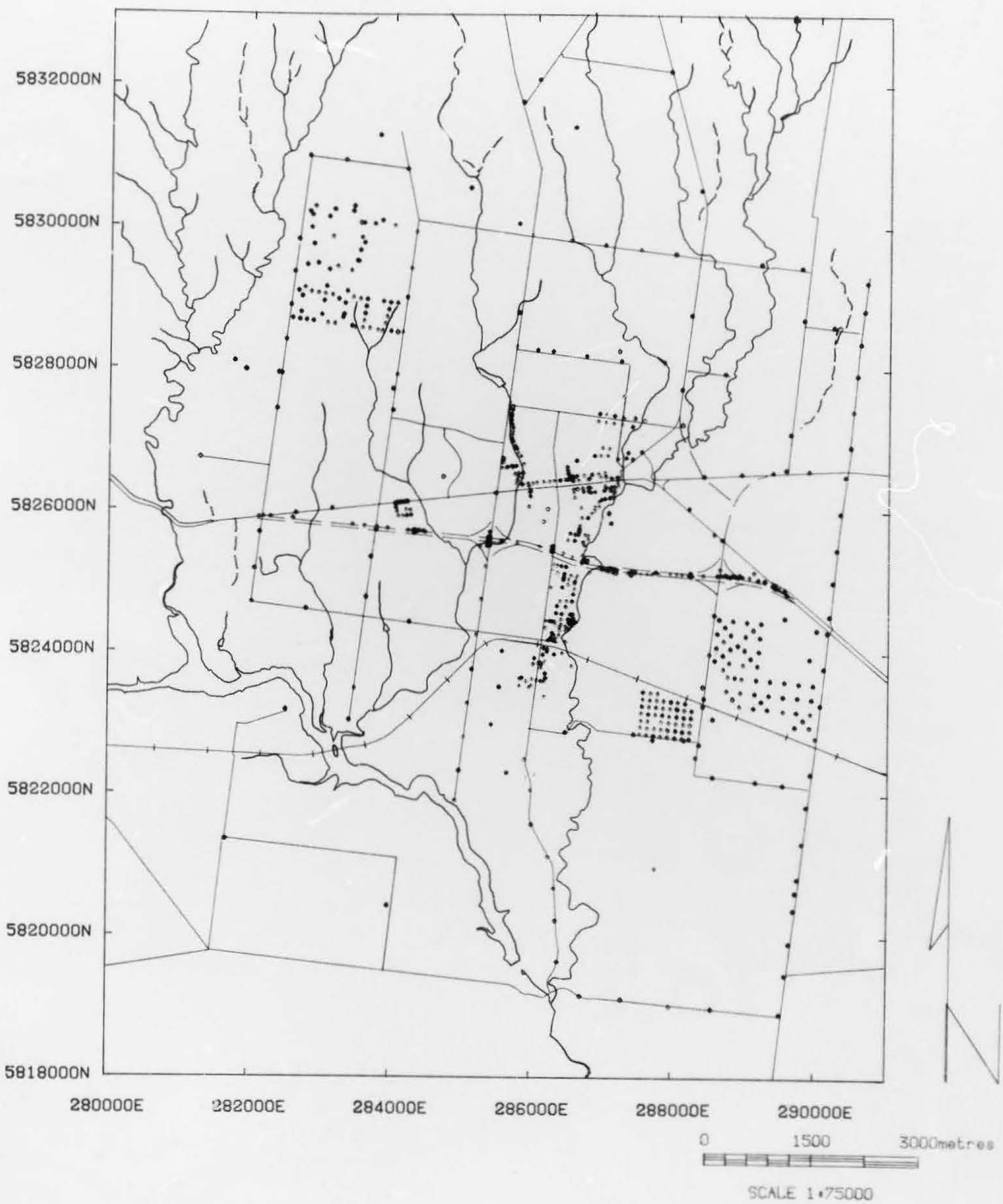
1.5 Numeric surface computation

The numeric surface representing soil depth was computed using a package supplied by Engineering Computer Services Pty Ltd (ECS). The program - GPCKRG - is part of an interactive general purpose gridding and contouring package known as GPC/GPCINT.

GPCKRG allows the computation of the semi-variograms, the fitting of either a simple linear or spherical model, and kriging using simple, block or universal kriging methods. During the computation of the grid manipulations may be made such as

* including trends or faults,
* expansion of the grid beyond the data points,
* applying smoothing operators to the grid,
* restricting the gridding to a defined polygon,
* limiting the grid to positive values only,
* the use of a sample location tolerance to simulate sampling error,
* limiting the interpolation to a given range, and
* including data from outside the grid area.

10



ENGINEERING GEOLOGICAL MAPPING
OF MELTON, VICTORIA.

Figure 2.
DISTRIBUTION OF SOIL DEPTH DATA

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The computed grid may then be masked to include or exclude given areas.

1.5.1 Variogram computation

For the data set available, semi-variograms were computed to establish the suitability of the data to kriging. The shape of the variogram computed (Fig. 3) showed a distinct nugget effect, range and sill value. This indicated that the data set is suited to treatment by kriging.

Semi-variograms were computed in the four cardinal directions to check for any possible anisotropies (Fig. 4). The resultant variograms showed no substantial differences in their shapes, which indicated a lack of anisotropy in the data set. The similarities of shape in the directional variograms also indicate the absence of strong regional trends (or drift), which alleviates the need for universal kriging.

From the total variogram, the modelling parameters were chosen. A spherical model was judged to best fit the data, with a nugget of 0.5 metres, a sill of 3.8 metres, and a range of 400 metres. The model is plotted on the variogram in Figure 5.

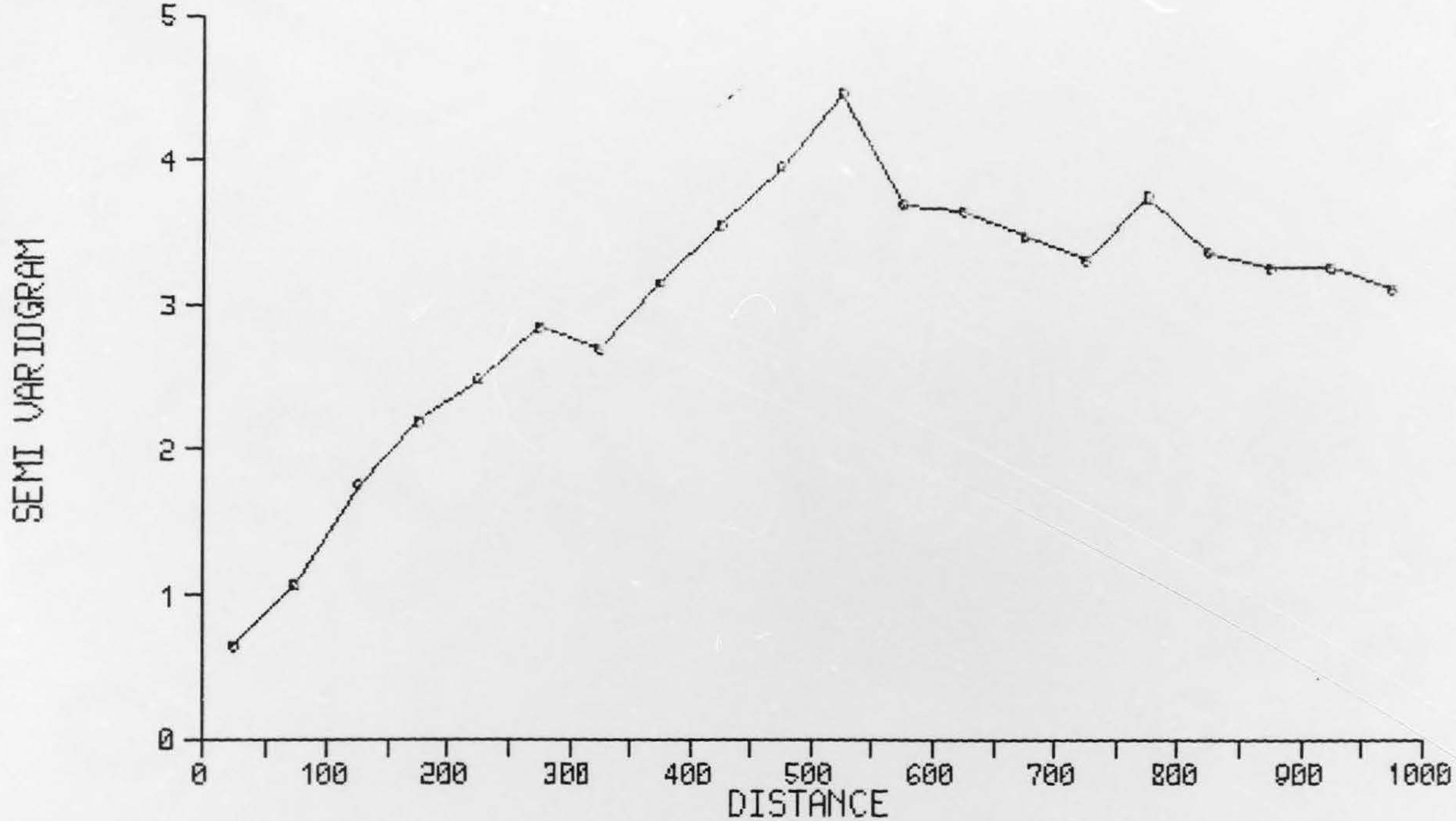
Several interesting observations can be made from the variograms. The average nugget effect indicates that the uncertainty in sampling the same location twice is 0.5 metres. More simply, this means that the soil depth can only ever be predicted to the nearest half-metre, even in the most frequently sampled locations. The average range of 400 metres indicates that the soil depth can be predicted (with a calculated confidence) from an observation in one place to another place up to 400 metres distant, after which there is no relationship. The sill of 3.8 metres represents the average difference in observations greater than 400 metres apart, or more simply, the maximum error in prediction.

1.5.2 Kriging

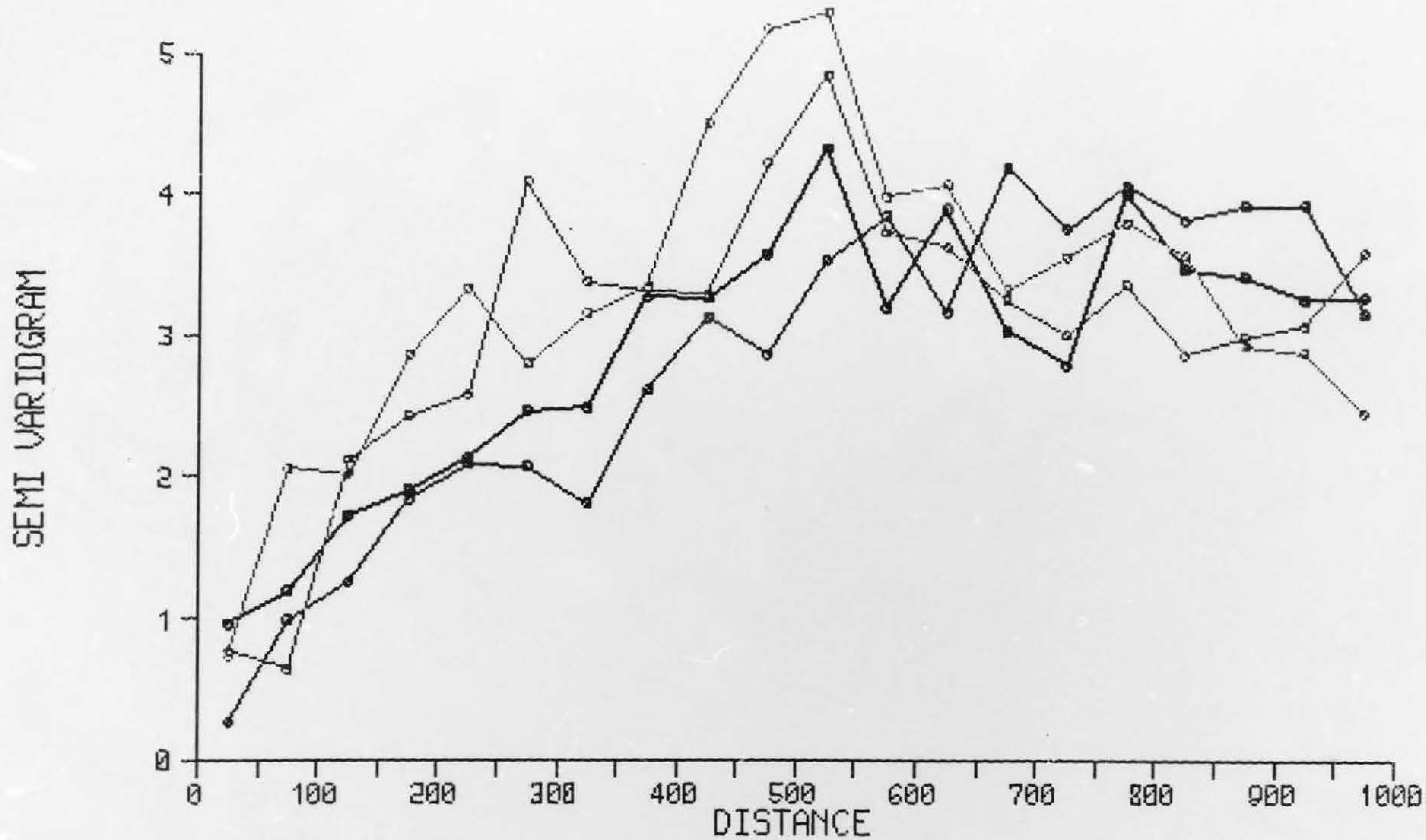
The variograms illustrate that the data set is suited to either block or simple kriging techniques. Kriging the data in blocks of 100 X 100 metres (same as the grid mesh size) was chosen as the most applicable method. This choice is based on an examination of the end requirement, ie. the production of a soil depth map which indicates the average thickness in an area, without being site-specific. By using block kriging the map indicates the average thickness that would be encountered over 100 X 100 metre cells (ie. 10,000m^2) which provides a suitable basis for making decisions pertaining to an area, rather than a specific site.

The gridding parameters were as follows:

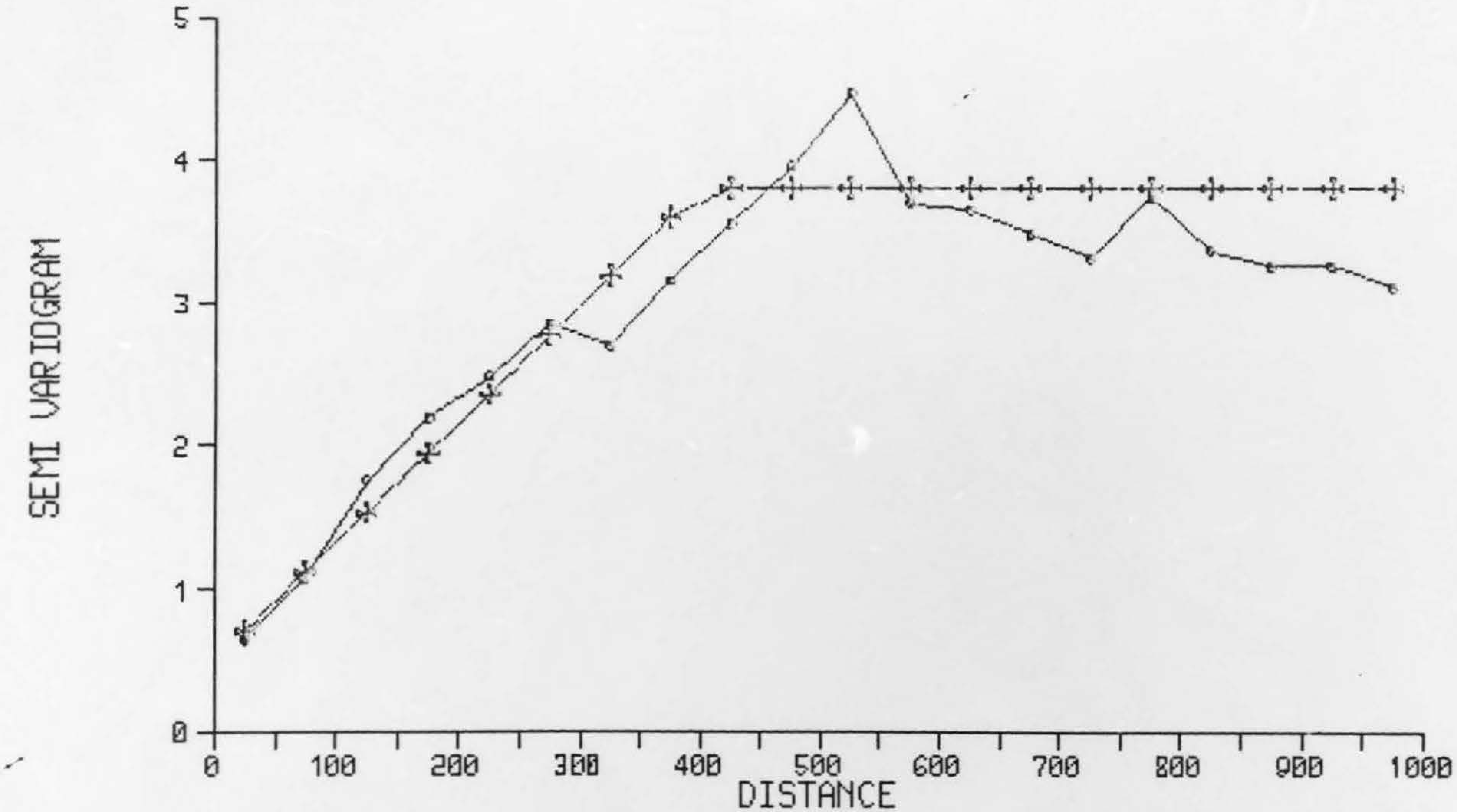
mesh size	100 X 100 metres
scan distance	2800 metres
data distance tolerance	5 metres
points searched per octant	2



No.	Covariance	Population																			
0	3.54	233	508	767	736	997	950	071	037	266	106	161	146	267	118	229	229	301	382	505	520



No.	Covariance	Population																			
1	3.48	70	191	219	231	291	229	305	251	348	278	349	264	360	291	359	288	302	322	392	324
2	3.19	37	88	184	180	237	252	227	217	232	232	199	258	222	184	220	209	210	222	204	248
3	3.68	85	155	187	195	271	230	297	291	362	354	345	344	375	336	358	412	455	454	478	502
4	3.67	41	74	177	130	198	239	242	278	324	242	268	280	310	307	292	320	334	384	431	446



No.	Covariance	Population																			
0	3.54	233	508	767	736	997	950	1071	1037	1266	1106	1161	1146	1267	1118	1229	1229	1301	1382	1505	1520

There was no:

- grid rotation,
- extrapolation beyond the mesh points,
- logarithmic gridding,
- smoothing,
- limit to interpolation, and
- restriction to a boundary polygon,

The grid was later masked to exclude the gridded area outside of the Melton development area boundary. This was carried out by utilising another GPC program (GRDMSK).

The gridding parameters were chosen to best fit the density and distribution of the data. The scan radius naturally presents a problem, since it can be seen from the variograms that the radius of search should be 400 metres. Using sample points 2800 metres away to predict mesh point values is not reliable (even though their weighting would be very small), but unfortunately necessary to prevent gaps in the grid. The only alternative would be to acquire more data.

By using a data distance tolerance two problems are simultaneously solved. The first is that kriging is a process which honours the data (subject to the mesh size selected). This assumes that the sampling is repeatable, even when a nugget effect is present, which is not always valid. The use of a distance tolerance simulates possible error in sampling and results in non covariance between samples, which relaxes the criterion to honour the data. The second problem is that much of the data was scaled from plans and diagrams to provide AMG co-ordinates. This introduces a real error (assumed to be in the order of five metres), which can be accounted for by the data distance tolerance.

1.6 Validity checking

The resultant soil depth plot required a validity check in order to assess the end result in terms of the real data. For this purpose an isopleth plot was compared with plots of the distribution of sample points in each range (Figs 1 to 10, Appendix II). The comparison was good, as would be expected from the gridding method chosen.

A rigorous test was performed by comparing the resultant isopleth map with plots of sample locations which were not included in the data set, but for which minimum a depth of soil is known (Figs 11 to 17, Appendix II). These sampled locations are those where the borehole or test pit did not encounter rock, and so strictly were not originally included. The comparison was generally very good, although in one small area known soil depths were consistently deeper than that predicted by the numeric surface. In this area an adjustment was made by including bores with the known soil depth.

The resultant variogram was little changed (the sill was adjusted to 3.6 metres), and so the overall adjustment to the grid was minimal.

1.7 Numeric surface representation

Two numeric surfaces are produced by the kriging techniques described above.

1 Soil depth
The resultant grid of soil depth values is illustrated as a shade-colour plot in Figure 6. By using shade colour, the minor variations are not so obviously displayed as on the corresponding isopleth plot (Fig. 7). The overall impression of gradation is given, rather than the concept of distinct boundaries.

2 Confidence values
The second surface computed by the program represents the confidence placed on the soil depth values. This surface is illustrated by a shade colour plot in Figure 8. The confidence values are expressed in metres and depict the 'plus-or-minus' values that can be placed on the predicted soil depth values at any given place. As expected, the confidence is high in areas where the data is dense, and poor in areas where data is sparse (as modelled by the variograms).

The confidence values highlight the lack of data in some areas of the map, such as the southeast corner, where the soil depth can be read as one metre plus-or-minus three and a half metres. However, the philosophy behind the attempt to illustrate soil depth is that any information is better than none at all, and the confidence grid can be included as a reliability diagram.

1.8 Geological interpretation

The resultant isopleth map of soil depth illustrates some interesting trends which have geological implications.

The variation in depth of soil may be due to two main causes. Firstly, differential weathering, or weathering that occurs at different rates as a result of variations in composition and resistance of a rock, or differences in intensity of weathering due to topographic or climatic conditions. Secondly, differences in geologic age of various surfaces, resulting in the development of younger and older weathering profiles.

In the Melton development area, the variations in soil depth can be interpreted as being due to a combination of geologic age and differential weathering. For example, the area of Melton closest to Mt Cottrell (i.e. the south-eastern portion of the map) shows uniformly shallow soil. This implies that some lava flows from the past volcanic eruptions are represented here by their shallow weathering profiles. The edge of the flow is bounded by Toolern Creek, the other side of which the soils thicken. These thicker soils would be formed on older lava flows and so have had time to weather deeper at an accelerated rate due to the moisture provided by Toolern Creek.

Similarly, the shallow soils in the north-west area would represent the most recent lava flow from the eruption point at 'Melton Park'. The thicker soils south and east of this flow would represent the soils in the older flows.

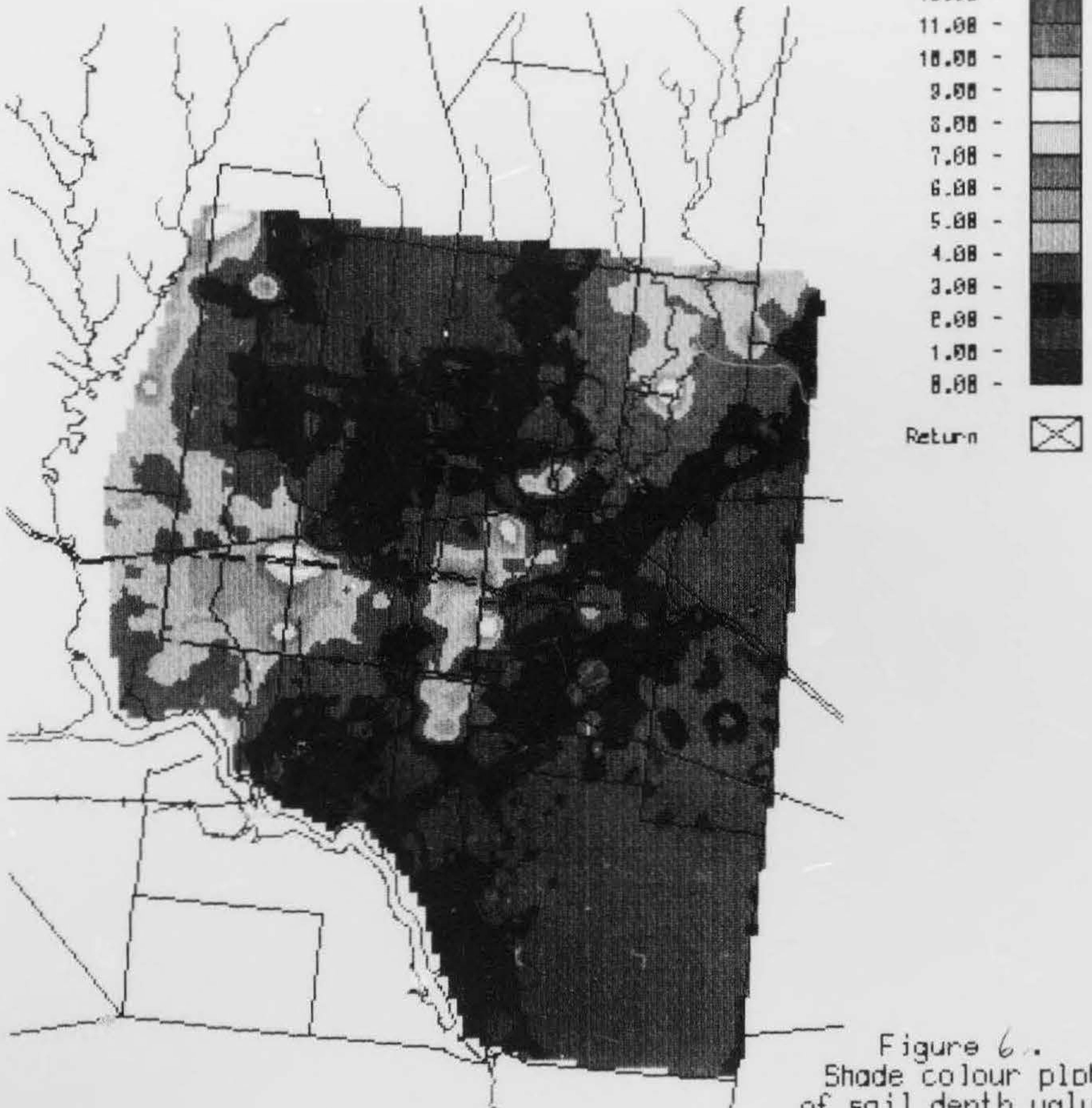
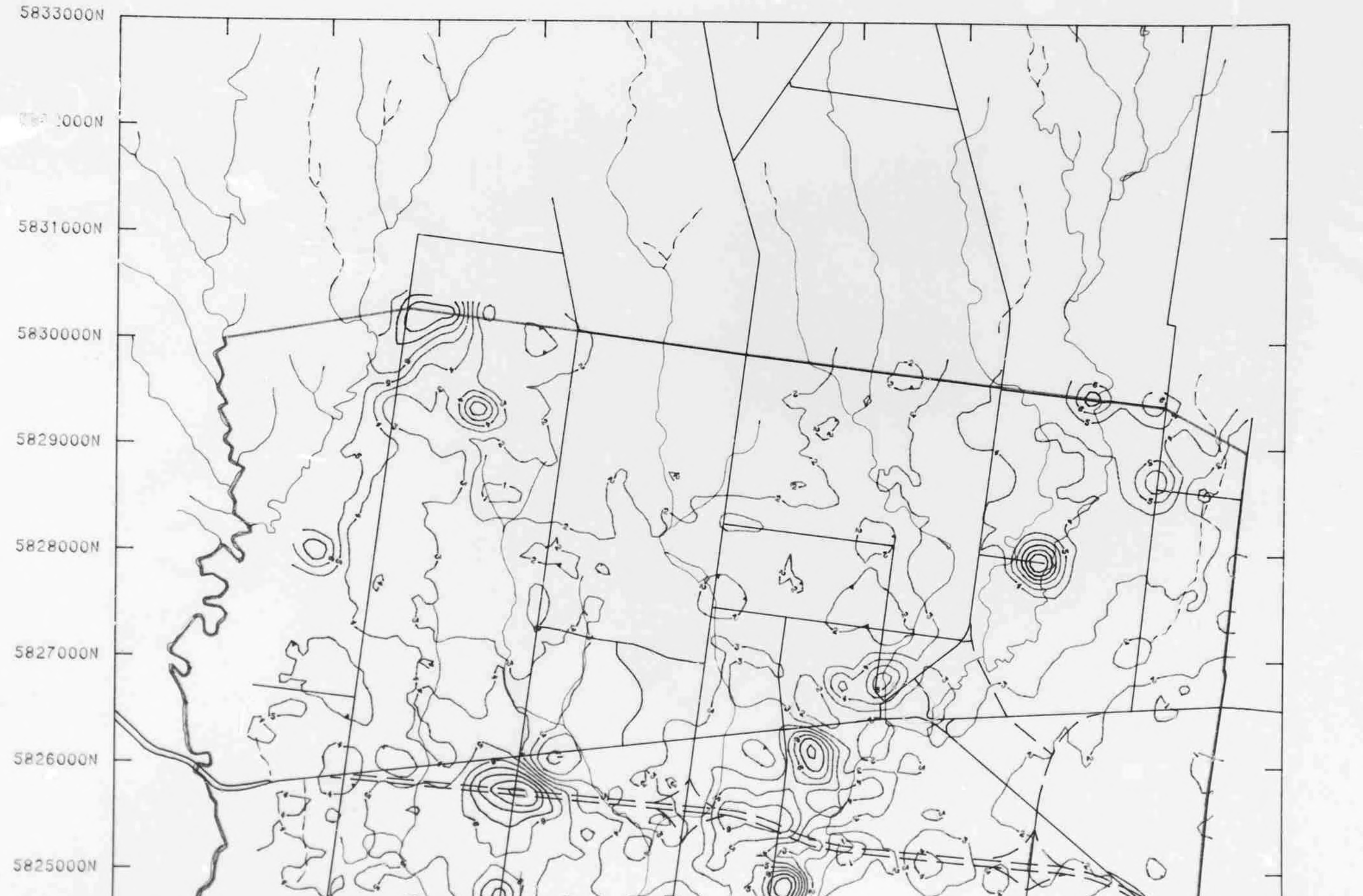


Figure 6.
Shade colour plot
of soil depth values



0 2500 5000metres

SCALE 1:50000

Numeric surface generated
using block kriging -
100 X 100 m. blocks.

Isopleth interval
- one metre
< 3 m. - red
3 - 6 m. - magenta
6 - 9 m. - purple
> 9 m. - blue

ENGINEERING GEOLOGICAL MAPPING
OF MELTON, VICT.

Figure 7.

SOIL DEPTH ISOPLETHS

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FIGURE 8. SOIL DEPTH MAP
CONFIDENCE VALUES

1.9 Geotechnical interpretation

The soil isopleths are a very useful guide to geotechnical parameters for decision making in development planning. The areas where soils are thin present problems for the servicing of developments with sewerage and storm-water drainage. In these areas excavation would often be carried out by expensively and laborously drilling and blasting.

Areas where soils are thick represent the problem areas for housing development, as the basalt soils have high shrink-swell potential, and maximum heave would occur on the thickest soils (section 4.2). Similar consequences relate to road making.

1.10 Further work

The upgrading of the soil isopleth map can be very easily carried out by adding new information to the data set and computing new numeric surfaces to plot. This would refine both the accuracy and confidence of new plots.

One spin-off from using kriging as a gridding technique is that sampling programs can be easily planned to achieve a required confidence value. For example, if soil depths to the nearest metre were required to be known in an certain area, then by perusing the variograms an optimal grid spacing could be chosen to achieve the desired result. This could be useful for the development of large subdivisions, where it is advantageous to know where shallow rock occurs.

2 Groundwater

The nature of the groundwater regime can be an important consideration in the engineering development of urban environments. Shallow groundwater tables can cause settlement of building foundations, as can over-extraction of a groundwater resource (Leggett, 1973). Large expanses of pavement increase runoff from storms and decreases the total amount of water recharging underground supplies. Similarly small residential lot size renders large areas impervious (Leopold, 1968).

Groundwater was not encountered at any time during the drilling program. Several open standpipes were installed in the deeper soil areas, and with one exception, groundwater was not encountered during one year of monitoring. The one exception (Djerriwarrh 102) was adjacent to the Toolern Creek, where water was encountered in gravel beds.

Table 2 documents the monitoring program.

Aspects of the occurrence and distribution of groundwater in the region have been documented by Kenley (1960 & 1977), Thompson (1972), Rhia (1975 & 1976), Plier-Malone (1977), and Williams (1983 & 1986). Essentially, the groundwater in the region occupies the two major sub-horizontal geological formations, the Newer Volcanics and the Werribee Formation (U.R. 1986/4).

Bore	Installation	17.11.'83	8.12.'83	29.3.'84	18.7.'84	16.8.'84	19.9.'84	17.10.'84	15.11.'84
		Date & Readings							
Djerriwarrh 100 Harkness Rd	10.11.'83 7.5 m Dry	7.5 m Dry	7.5 m Dry	7.5 m Dry	7.5 m Dry	7.54 m Dry	7.59 m Dry	7.57 m Dry	7.56 m Dry
Djerriwarrh 101 Hardy's Rd	11.11.'83 2.75 m Dry	2.7 m Dry	2.7 m Dry	2.7 m Dry	-	2.47 m Wet	Area flooded	1.98 m Wet	2.0 m Wet
Djerriwarrh 102 Shire Pound	14.11.'83 8.75 m Dry	7.21 m Wet	7.0 m Wet	7.15 m Wet	7.59 m Wet	7.42 m Wet	7.53 m Wet	7.17 m Wet	Destroyed
Djerriwarrh 103 Buckle Crt	15.11.'83 12.5 m Dry	12.4 m Dry	12.4 m Dry	12.4 m Dry	12.4 m Dry	11.94 m Dry	11.94 m Dry	11.96 m Dry	11.96 m Dry
Djerriwarrh 104 The Bullock Trk	16.11.'83 5.2 m Dry	5.1 m Dry	5.1 m Dry	5.1 m Wet	5.0 m Wet	4.83 m Dry	4.88 m Dry	4.89 m Dry	4.89 m Dry
Djerriwarrh 105 Cnr Harkness Rd & Porteous Rd	18.11.'83 12.5 m Dry	-	12.0 m Dry	12.0 m Dry	12.0 m Dry	11.94 m Dry	11.36 m Wet	11.38 m Dry	11.36 m Dry
Djerriwarrh 106 Bulmans Rd	21.11.'83 7.4 m Dry	-	7.0 m Dry	6.9 m Dry	6.9 m Dry	6.74 m Dry	6.79 m Wet	6.79 m Dry	6.79 m Dry
Djerriwarrh 107 Bulmans Rd	23.11.'83 8.0 m Dry	-	7.5 m Dry	7.5 m Dry	7.5 m Dry	7.29 m Dry	7.31 m Dry	7.32 m Dry	7.31 m Dry
Djerriwarrh 108 Bulmans Rd	23.11.'83 3.4 m Dry	-	3.3 m Dry	Destroyed					
Kororoit 67 Cnr Ryans Rd & Finchs Rd	25.11.'83 8.1 m Dry	-	8.0 m Dry	8.0 m Dry	8.0 m Dry	7.83 m Dry	7.86 m Dry	7.86 m Dry	7.86 m Dry

Table 2. Open standpipe readings.

Note. From 16.8.'84 the accuracy of recording was improved.
Groundwater encountered in bore Djerriwarrh 102.
Bore Djerriwarrh 101 was situated in low-lying area.

The Newer Volcanics consist of layered sequences of basalt flows and interbedded soil layers of low permeability. The basalts are differentially cracked by large numbers of thermal contraction joints which represent about 0.5% by volume of the rock and serve both as the main space for water storage and the channelways for water movement. They generally contain at least two separate aquifers - upper and lower - which in places are vertically interconnected and operate as a two aquifer system (Kenley, 1977).

The upper basalt aquifer is an unconfined water table aquifer which is recharged by direct slow infiltration of rain or stream water. The lower basalt aquifer is a confined low pressure aquifer. Water enters this aquifer in areas where it locally outcrops and partly also by vertical leakage from the upper aquifer in places where the low permeability interbeds are lacking.

The Werribee Formation contains a number of porous sand and gravel layers each of which behaves as a confined aquifer under considerable hydrostatic head. The groundwater in this formation occupies the pore spaces between the sand grains which may represent up to 25% by volume of the sand. These sands do not outcrop in the map area, and recharge may be from the north and west where the Formation outcrops, or from downward leakage from the basalt aquifers.

Information on the groundwater in the Melton Development Area is generally lacking. Only 36 of the researched bores had standing water level measurements, while 45 had groundwater quality information. Two numeric surfaces were computed from the data.

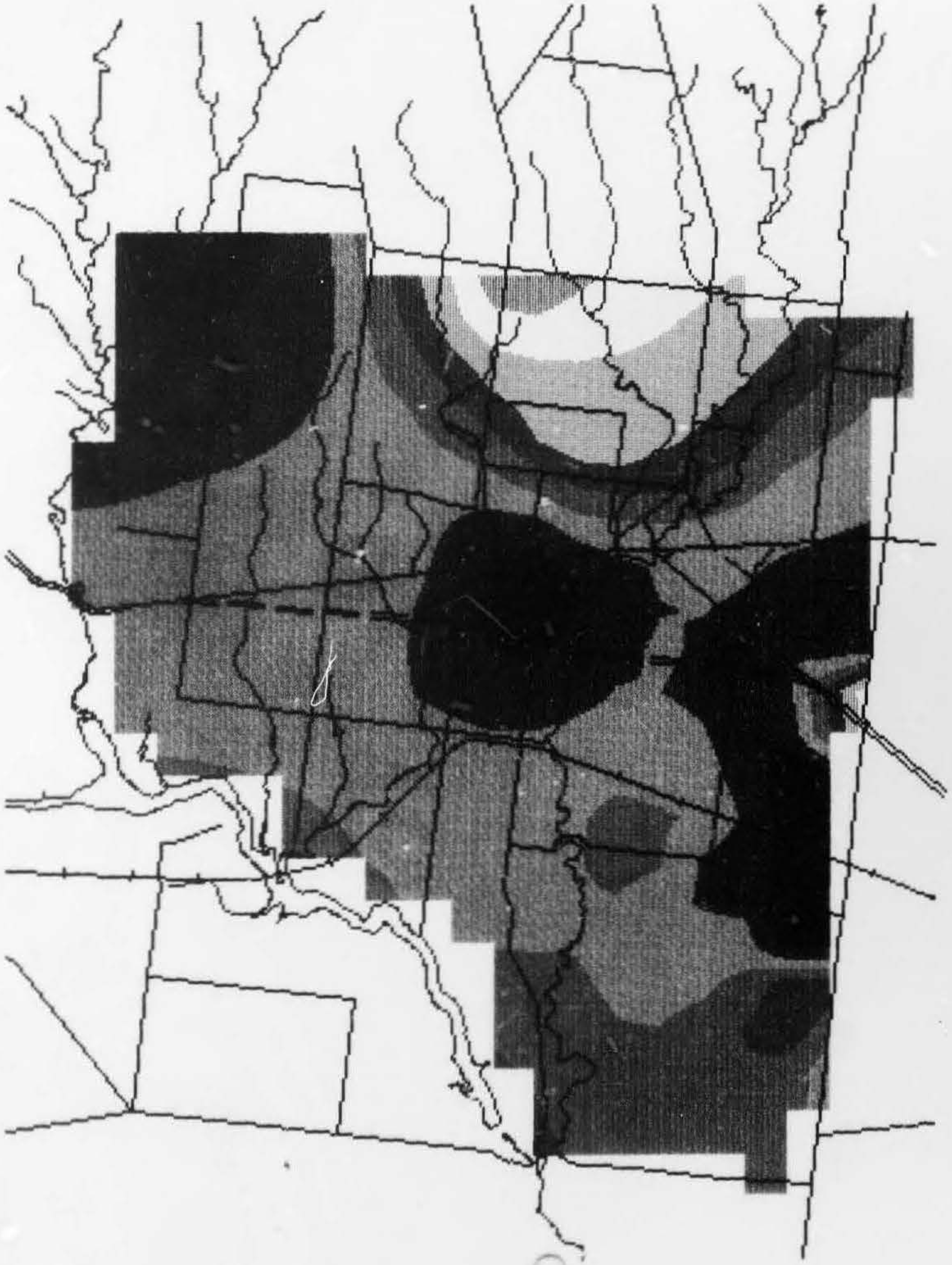
The first - a grid of the standing water levels - was computed using a general purpose gridding program (GPCGRD; ECS, 1986). The second - a grid of the height above the AHD of the potentiometric surface - was computed by subtracting the standing water level grid from a grid of the topographic surface. These surfaces are illustrated in Figures 9 & 10.

2.1 Water Quality

Table 3 details the quality of the groundwater within the map area.

In general terms, the groundwater in the Melton map area is of the sodium-chloride type with some magnesium sulphate. There are, however, some differences both in concentration and percentage.

A major study of the groundwater conditions of a proposed quarry site (bounded by Ferris Rd, Mt Cottrell Rd and the North-Western Railway) found that two water masses, one in the upper and one in the lower aquifer, can be identified from water quality (Williams, 1986). Figure 11 illustrates this difference where the Stiff diagrams for the upper and lower aquifers from test pump well samples and two farm bores have been compared. The difference is due to a greater percentage of magnesium chloride in the lower aquifer and sodium bicarbonate in



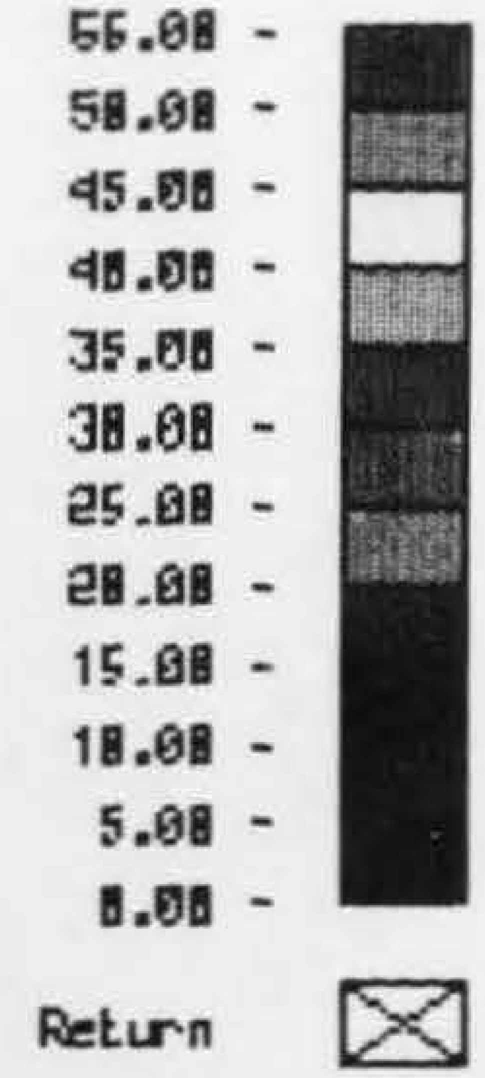


FIGURE 9 . STANDING WATER LEVELS

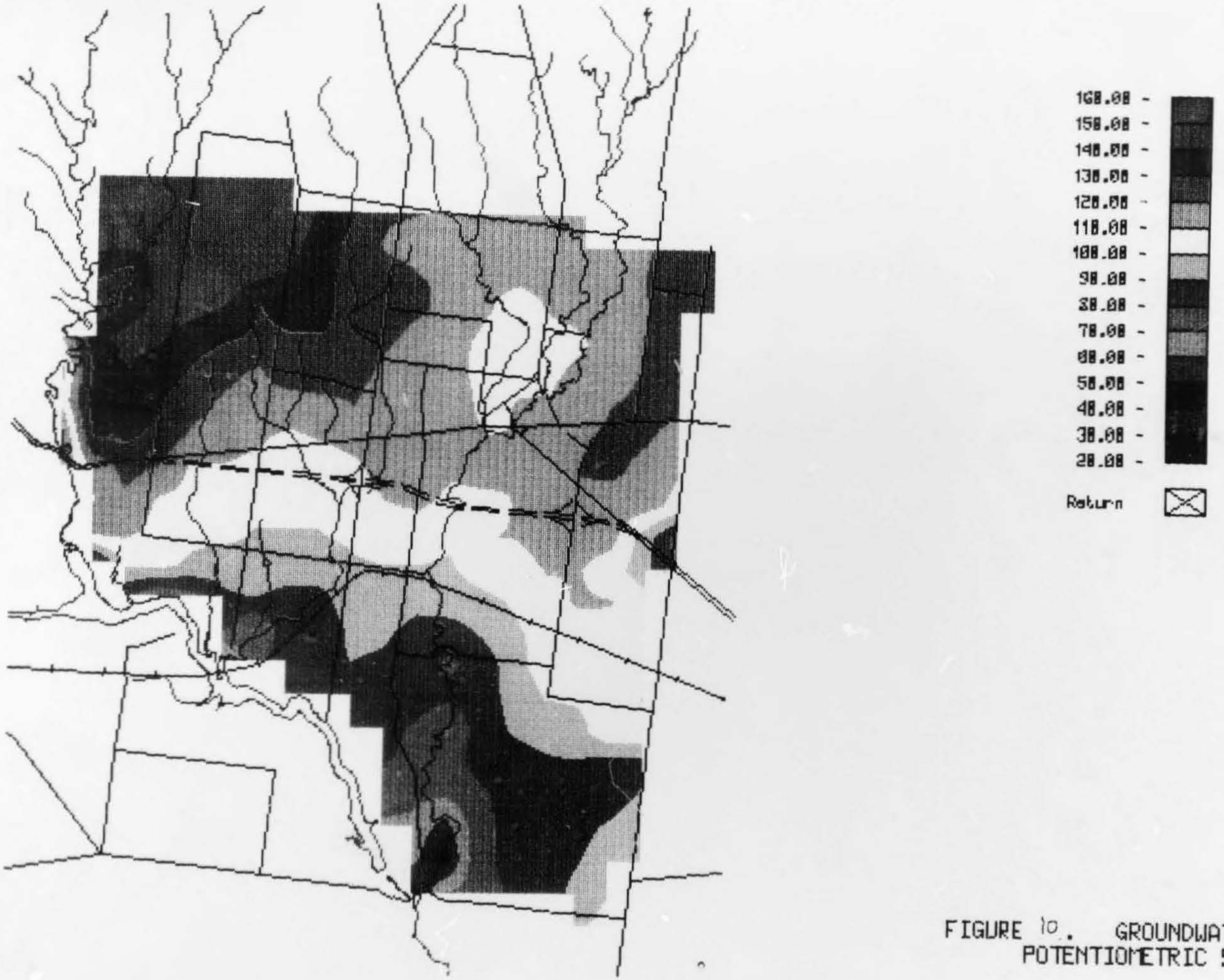


FIGURE 10. GROUNDWATER POTENTIOMETRIC SURFACE

ANALYSIS

BORE	pH	TDS mg/l	$CaCO_3$	Fe soluble	Fe total	Na	K	Ca	Mg	SiO_2	Cl	SO_4	CO_3	HCO_3	Cond. omhs
D8001	7.5	5110									2250			774	
D8003	7.6	613	171	.01	.36			21	29	11	169	65		187	
D8003	7.6	584									194			208	
D8004	7.8	5930									2956		6	452	
D8005		7260									3320			1244	
D8006		5040									2400		33	790	
D8007		10080									5070			447	
D8008	7.8	4080									2300				
D8009	8.1	3400									1508		30	408	
D8010	7.8	4880									2446			530	
D8011	8.1	1826	625	.04	.13			67	111	30	832	77		311	
D8012	8.2	3270									1615			303	
D8013	8.0	3240									1645			335	
D8014	7.8	10380									5330			1007	
D10001	8.1	2061	348		.20	539	13	32	65	72	742	105	0	493	3200
K16	8.4	3016	840	.00	.20	850	15	42	83	76	1116	158	30	632	4500
K8001	8.0	6600	1940	.03	.07	1340	29	104	410	23	2570	640	0	290	
K8002	8.2	3580	1020	.02	.09	880	12	56	210	39	1500	270	0	680	
K8006	7.6	2420									1130			391	
K8007	8.0	1740									807			362	
K8008	8.0	3900	960	.03	.46	940	20	68	190	65	1700	240	0	460	
K8035	7.9	2396	656	.00	.40	573	20	25	144	72	1078	154	0	316	3840
K8036	7.8	3500	880	.06	.18	870	16	68	170	67	1520	250	0	500	
K8053	8.2	5746	1544	.00	.02	1530	38	58	340	59	2960	418	30	309	9300
K8070	7.9	5090	990	.03	.09	1420	39	76	190	43	2310	440	0	390	
K8071	7.9	2450	380	.02	.35	740	24	48	63	40	1010	210	0	440	
K8072	8.0	3300	990	.08	.09	790	32	96	180	56	1300	430	0	410	
K8086	7.8	2060	190	.02	.49	690	14	22	33	50	830	170	0	460	
K8087	7.8	2800	590	.02	.49	730	20	66	103	21	1230	270	0	290	
K10009	8.2	2884	846		1.50	677	15	70	163	55	1280	192	12	419	4710
K10009	8.1	2480	250	.02	.02	850	15	32	41	54	1050	240	0	460	
K10011	7.9	2061	156	.01	4.80	621	9	18	27	64	639	142	0	507	2995
K10011	7.8	2020	180	.02	.02	690	11	24	29	67	790	150	0	500	
K10012	8.1	3803	612	.01	2.20	1100	22	59	113	59	1550	318	0	553	5950
K10015	7.9	5752	895	.01	.00	1750	32	108	152	44	2730	608	0	317	9000
K10019	8.5	2300	550		.09	560	17	64	97	51	890	170	0	488	
P8003	8.4	1817									647		60	406	
P8004	8.5	1807	200	.02	.84			13	41	19	635	72	65	409	
P10001	7.8	8347	2680					207	526		3980			100	12884
P10010	8.2	1972	306	.01	.70	542	11	17	64	31	739	85	0	481	3175
Y8001	8.2	7260									3830		6	251	
Y8002	7.8	3240									1603			243	
Y8003	8.0	2340									1080		6	240	
Y8004	8.3	3500									1584			308	
Y8006	7.8	3030									1331		18	511	
Y8007	8.1	3460									1513		24	631	
Y8009	7.9	2540									1240		12	396	
Y10004	7.8	7510									3730			407	12000

Key to bores: D - Djerriwarrh; K - Kororoit; P - Pywheitjorrk; Y - Yangardook.
(u) - upper aquifer (l) - lower aquifer

Table 3. Water quality.

the upper aquifer. These differences are probably caused by the residence time being less in the upper aquifer. The analyses also show a clear cut salinity gradient with depth. In the upper aquifer total dissolved solids (TDS) values of 2020 - 2480 mg/l are recorded while in the lower aquifer the values ranged from 2800 - 6600 mg/l.

With the exception of one bore (Djerriwarrh 8003), all the groundwater recorded has levels of total dissolved solids higher than generally recommended for human consumption. The World Health Organisation (WHO) sets a desirable limit of 1500 mg/l with an objective limit of 500 mg/l. The sodium content is well in excess of the taste threshold recommended by the WHO of 150 mg/l. Generally, the iron content is such that it would cause staining of bathroom fixtures and could stain clothes washed with the water.

The groundwater would be suitable for some stock watering, the limits for TDS and magnesium in drinking water of livestock are given in Table 4.

The agressiveness of water is a more complicated quality to determine, involving many variables related to both the water chemistry and flow rate and the nature of the material under attack. The State Chemistry Laboratory consider that, in general, water with a TDS of 3000 mg/l or greater is probably aggressive toward metal.

	Total Soluble Salts mg/l	Magnesium mg/l
Poultry	3500	-
Pigs	4500	-
Horses	6000	250
Cows in milk	6000	250
Ewes with lambs	6000	250
Beef cattle	10000	400
Adult sheep on dry feed	14000	500

Table 4. Limits for total soluble salts and magnesium in drinking water of livestock.

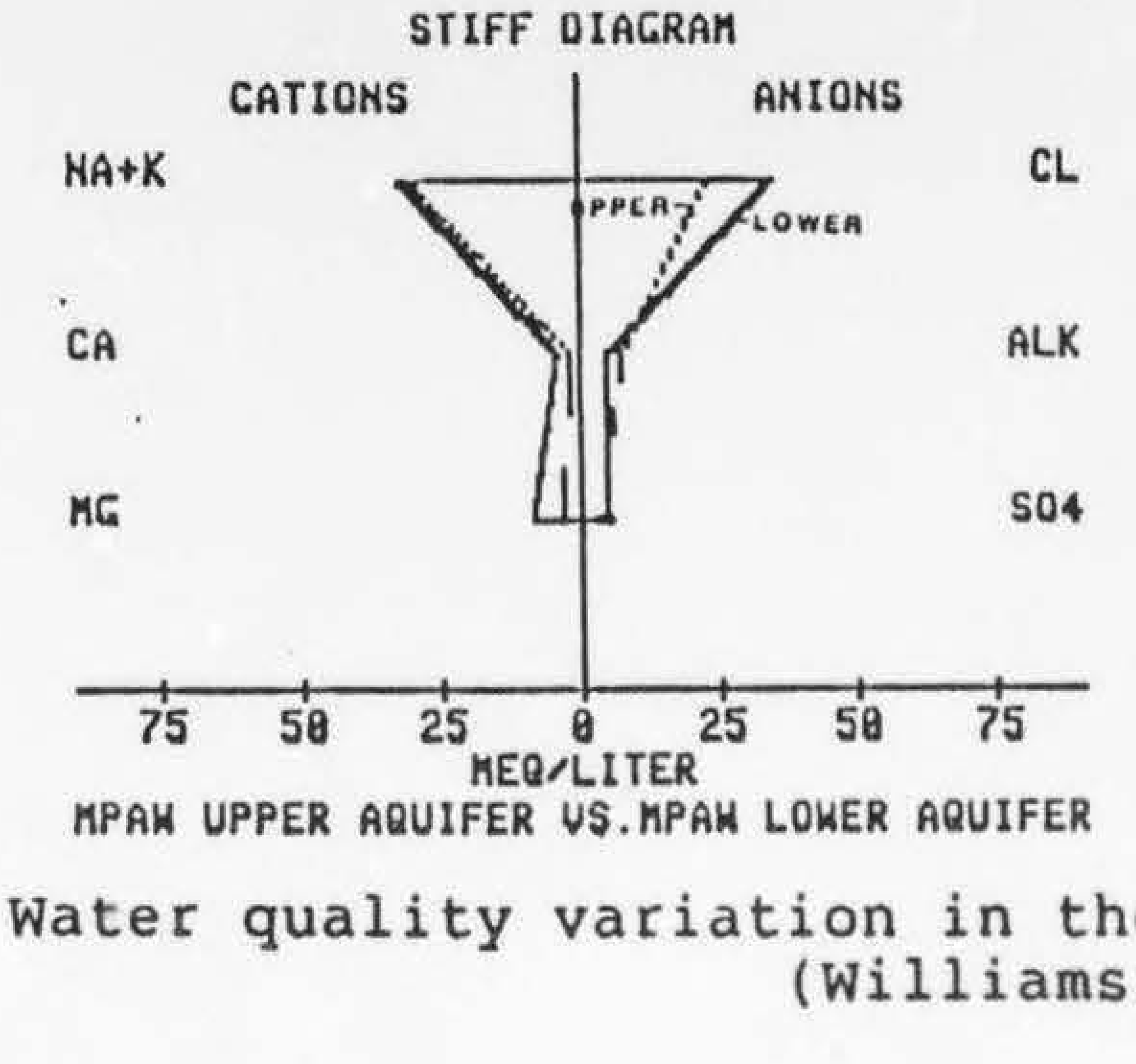


Figure 11. Water quality variation in the aquifers. (Williams,1986)

One problem with the quantity of test data is that there are insufficient to form accurate numeric surfaces. Gridding the data and producing isopleth maps is not feasable because of the extreme variability in the data over short distances (in three dimensions), which makes prediction speculative.

The distribution of soil samples tested to determine the various engineering properties is illustrated in Figure 12. The data set is essentially a scatter of points (in three dimensional space) which require graphical representation to study their relationship in any configuration. These graphical pictures are useful to confirm or contradict previous concepts, and may reveal new ideas in a dramatic way.

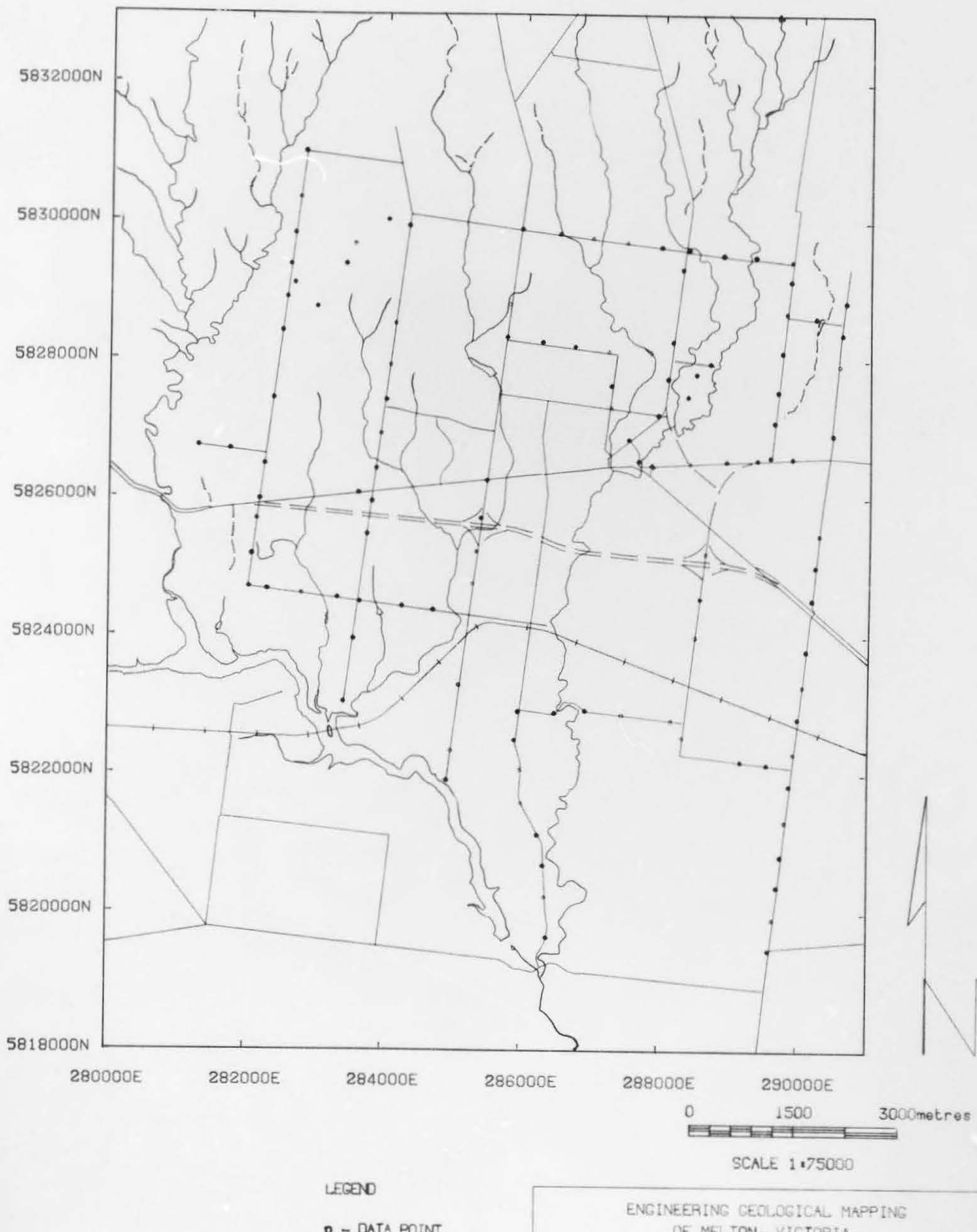
There are many of these graphical techniques available. The simplest code a single numerical value into a simple character (Fig. 13). Others code single values of two or more variables into one compound character (Fig. 14).

When designing or choosing compound character scales, consideration must be given to whether the scales are separable (i.e. whether one can easily shift attention from one coded aspect to another), and whether the coded aspects are individually value-mergeable into impressions of regional trends.

Of the compound character scales shown in Figure 14, the most unusual and versatile is the Chernoff face (Chernoff, 1973). A revised version of the face by Davis (Bruckner, 1978) allows the coding of up to 20 variables (Fig. 15 & Table 5). Much has been written on the merits and demerits of the use of Chernoff's faces (Wang, 1978).

The major difficulty in trying to represent the Melton soil test data is that the data has variation in all three dimensions (i.e. there are multiple 'z' values at any x,y location). Representation of this foliation or layering at quite different levels of 'z' has not been solved. One suggestion is the procedure of locating the 'most imposing gap' in the collection of 'z' values for nearby (x,y) points followed by smoothing (Tukey & Tukey, 1980). Then to study the foliated structure several kinds of plots can be generated, such as smoothed gap location values, display the original points coded in some way to indicate which layer they are in, or make separate displays for the points in each layer. All these methods, however, are inappropriate, since the variables still require 'layering' into intervals.

In representing geochemical data, the use of 'flag maps' can overcome the difficulty of three dimensional representation (Farrelly, pers. com., 1984). Figure 16 shows the liquid limit values illustrated as a flag map. From this it can be seen that the geographic variation and the variation with depth is not clearly represented. In general, the area to the north and west have higher liquid limit values than the areas to the south and east.



LEGEND

o - DATA POINT

ENGINEERING GEOLOGICAL MAPPING OF MELTON, VICTORIA.

Figure 12.
DISTRIBUTION OF SOIL TEST DATA

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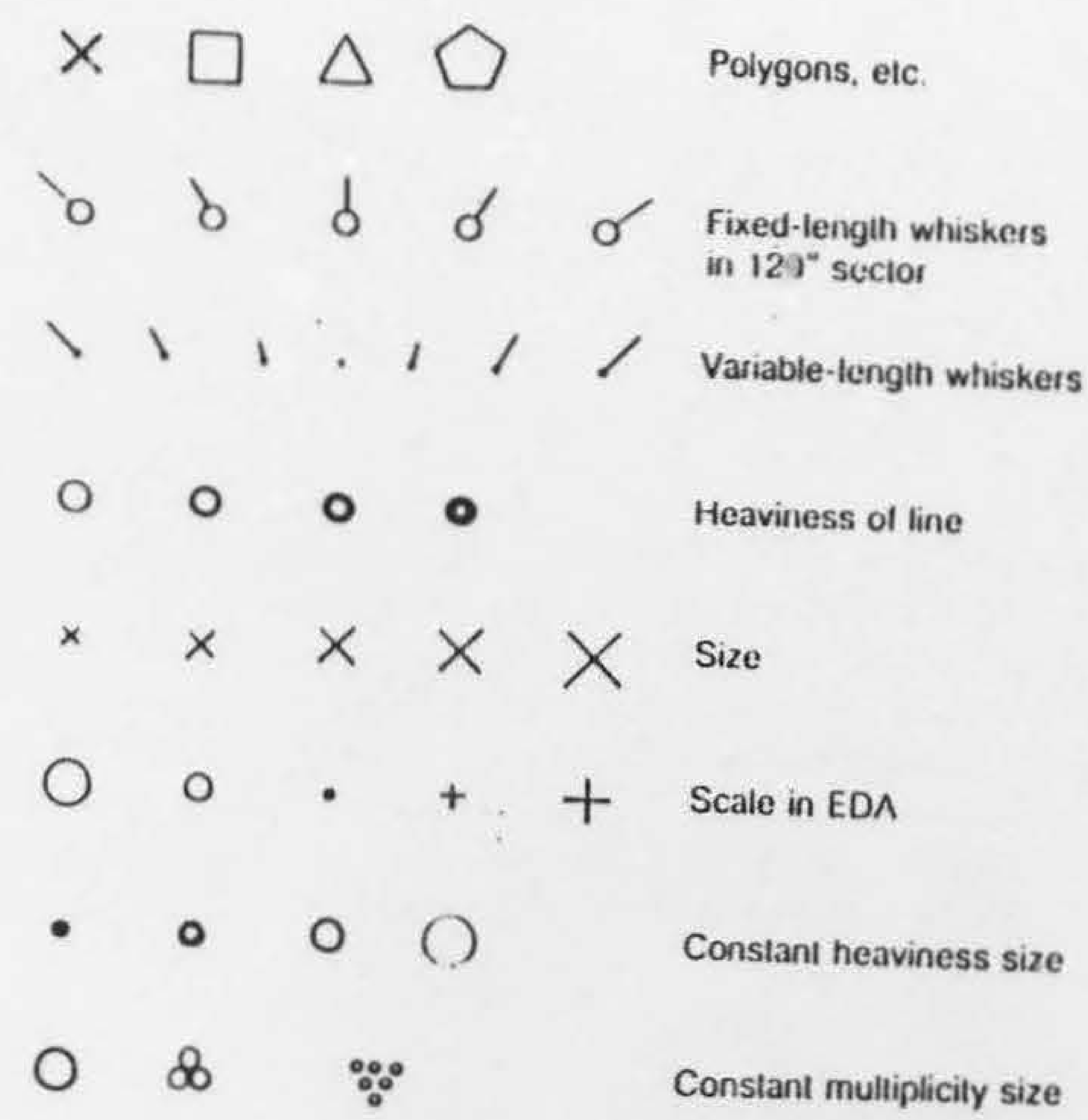


Figure 13. Several possible individual-value simple character scales. (Tukey & Tukey, 1980)

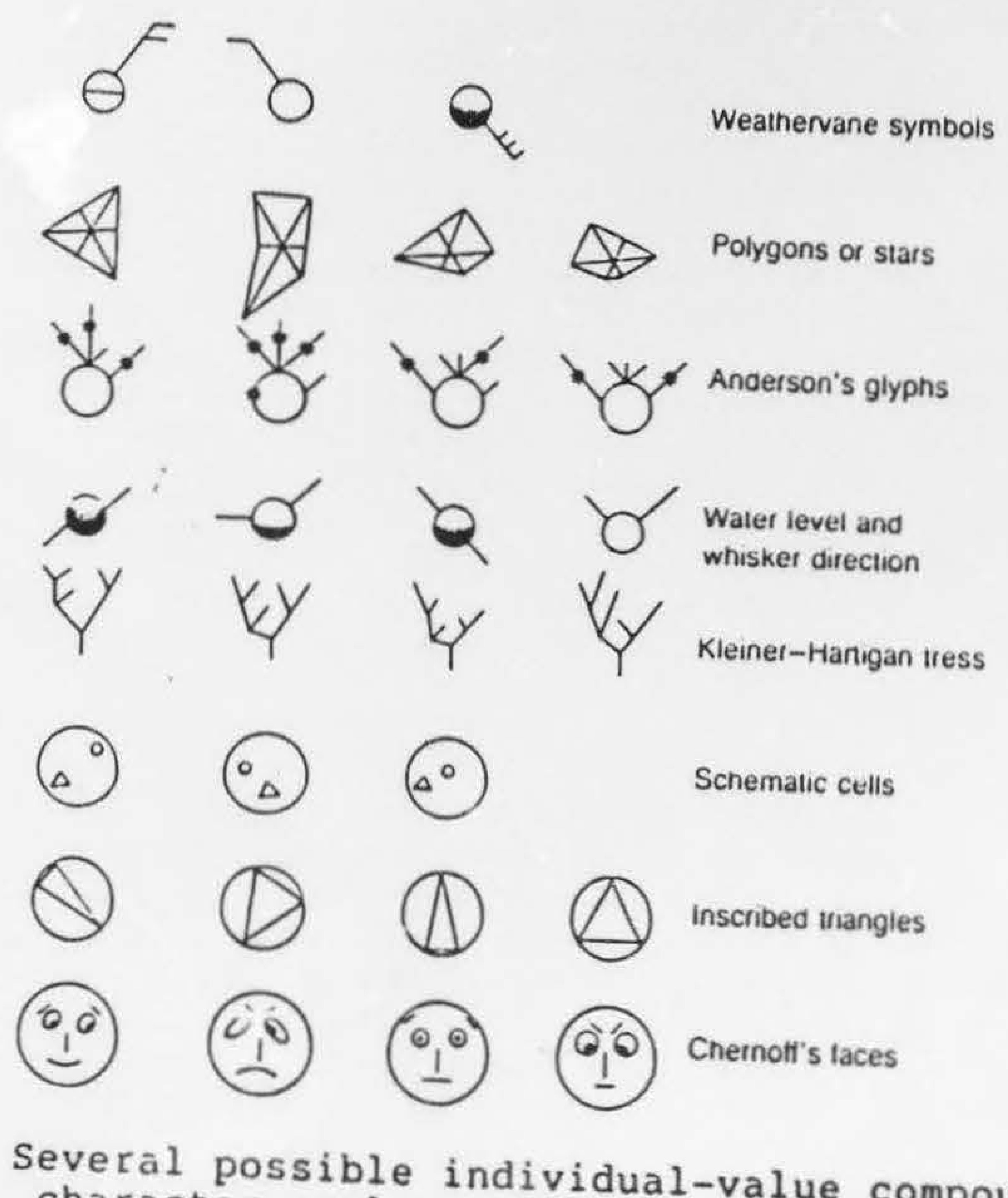


Figure 14. Several possible individual-value compound character scales. (Tukey & Tukey, 1980).

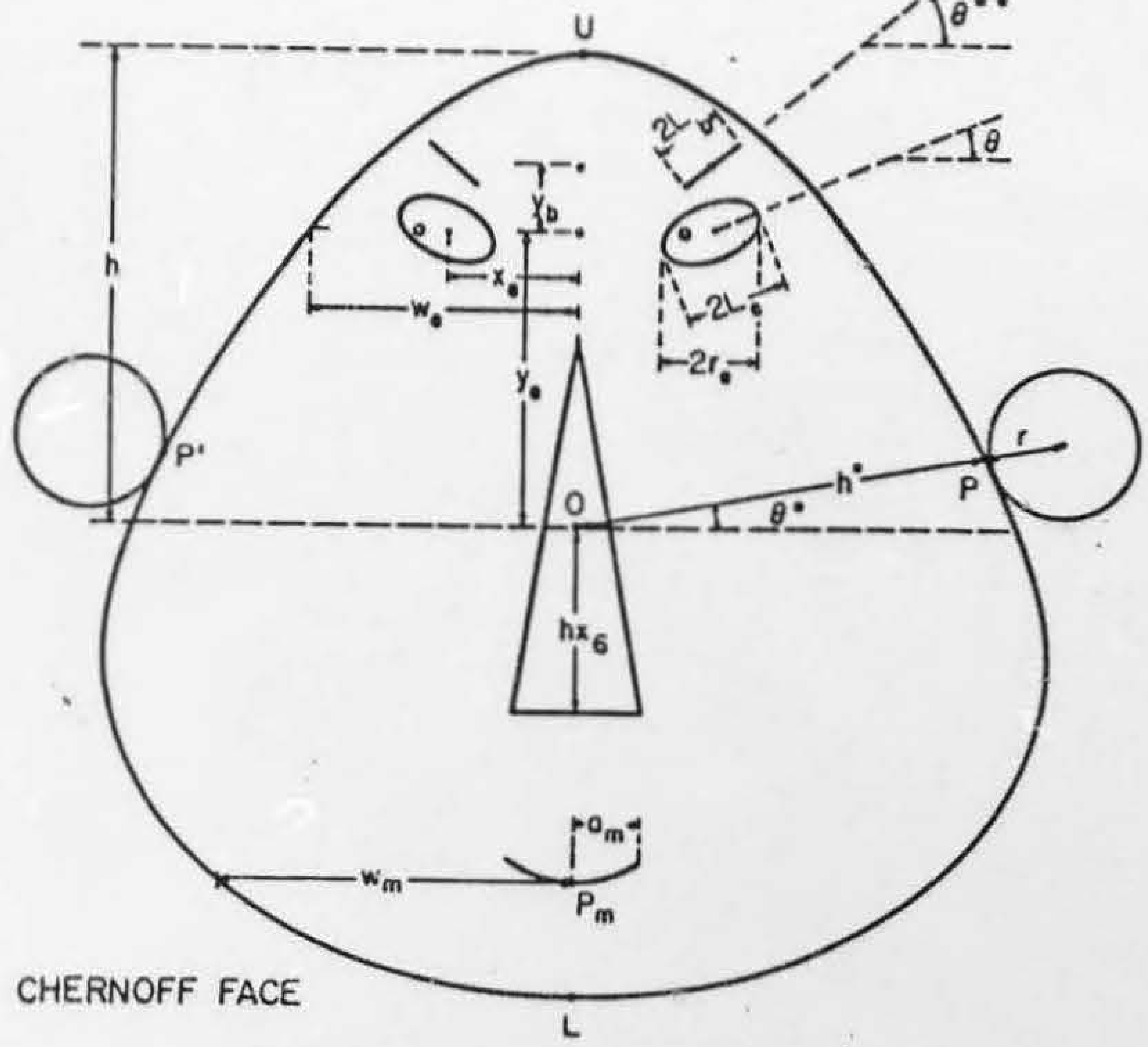


Figure 16.
Chernoff Face.
(Bruckner, 1978)

Variable		Facial Feature	Default Value	Range	
x_1	controls h*	face width	.60	.20	.70
x_2	controls θ*	ear level	.50	.35	.65
x_3	controls h	half-face height	.50	.50	1.00
x_4	is	eccentricity of upper ellipse of face	.50	.50	1.00
x_5	is	eccentricity of lower ellipse of face	1.00	.50	1.00
x_6	controls	length of nose	.25	.15	.40
x_7	controls p_m	position of center of mouth	.50	.20	.40
x_8	controls	curvature of mouth	0.00	4.00	4.00
x_9	controls	length of mouth	.50	.30	1.00
x_{10}	controls y_e	height of center of eyes	.10	0.00	.30
x_{11}	controls x_e	separation of of eyes	.70	.30	.80
x_{12}	controls θ	slant of eyes	.50	.20	.60
x_{13}	is	eccentricity of eyes	.60	.40	.80
x_{14}	controls L_e	half-length of eye	.50	.20	1.00
x_{15}	controls	position of pupils	.50	.20	.80
x_{16}	controls y_b	height of eyebrow	.80	.60	1.00
x_{17}	controls θ** −θ	angle of brow	.50	.00	1.00
x_{18}	controls	length of brow	.50	.30	1.00
x_{19}	controls r	radius of ear	.50	.10	1.00
x_{20}	controls	nose width	.10	.10	.20

Table 5. Description of facial features and ranges.
(Bruckner, 1978)

The correlation of high Atterberg limit values, LS values and FS values with areas of deeper soil is quite good (Fig. 17) These areas with high swell potential and deep soils constitute the worst possible conditions for development. In these areas the soil will exhibit large shrink-swell values and have the maximum depth (therefore the maximum volume) for heave (section 4.2)

4 Suitability for Development

4.1 Past assessments

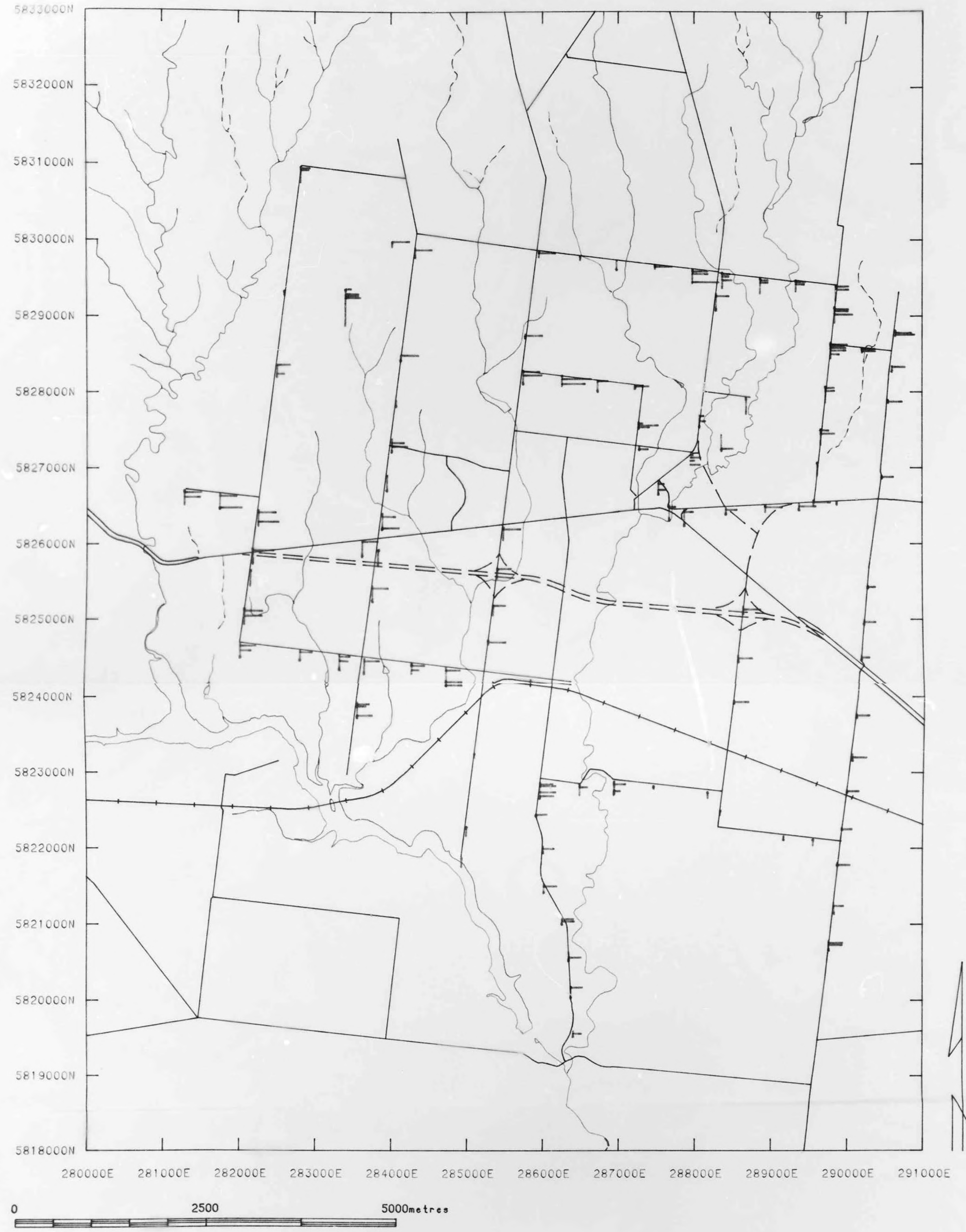
In their assessment of the existing environment at Melton, Clarke Gazzard Planners Pty. Ltd. found:

> "Foundation conditions experienced in Melton are similar to those in Melbourne's Western Suburbs.
> The material underlying most of the study area is of basaltic clay type. It is moderately expansive but does not present serious problems nor impose significant cost penalties in normal types of single and double storey domestic and commercial construction.
> Design of services and foundations needs to compensate for some seasonal fluctuations.
> Minor areas such as that in the south east near Greigs Road where Rockbank Association soil occurs could present problems in road construction. Special measures may need to be taken to combat erosion in the zones of alluvium associated with Toolern Creek and most extensive in the north east and near the reservoir in the south west of the Designated Area."
> (Clarke Gazzard, 1976)

This assessment was based on a CSIRO terrain classification map (Grant, 1972) produced at a scale of 1:250000, and enlarged to approximately 1:55555 for inclusion in their report. The Terrain Patterns of this map compare remarkably closely to the 1:250000 Melbourne geology map (1972). Both these maps are erroneous.

On the matter of soil classification (presumably in the engineering sense), Clarke Gazzard Planners Pty Ltd concluded:

> "The major proportion of the study area is underlain by a basaltic clay type material. This material is similar to that which occurs in the western suburbs of Melbourne, in summer being very hard with considerable surface cracking whilst in winter it is moist and puggy. It does not present any serious problems in regard to conventional domestic or commercial types of building up to two storeys in height when properly designed to account for the seasonal movements and the expansive properties of the soil. Excavations for footing or services is somewhat more difficult because of the presence of rock floaters which vary considerably in size and may require the use of explosives to achieve the desired shape. Such difficulties in excavation are reflected in higher earthwork costs than would apply in other areas.
> Based on the preliminary information available soil conditions in the study area do not represent a planning constraint."
> (Clarke Gazzard, 1976)



Bore depth shown in brown
Scale 1:1000

Liquid limit value shown in green
Scale 1:20

ENGINEERING GEOLOGICAL MAPPING OF MELTON. VICT.	
Figure 16. LIQUID LIMIT VALUES	
PROJ NO. GE 14	DATE: 16-NOV-87

Numeric surface generated
using block kriging -
100 X 100 m. blocks.

Isopleth interval
- one metre
< 3 m. - red
3 - 6 m. - magenta
6 - 9 m. - purple
> 9 m. - blue

Bore depth shown
in brown
Scale 1:1000

Liquid limit value
shown in green
Scale 1:20

ENGINEERING GEOLOGICAL MAPPING
OF MELTON, VICT.

Figure 17.
SOIL DEPTH ISOPLETHS
AND LIQUID LIMIT VALUES

PROJ NO.	GE 14	DATE:	16-NOV-87

1986/5

Following recommendations made in the abovementioned report, the DVA was commissioned to examine the requirements for drainage and flood mitigation at Melton. As a part of this study the DVA requested the SCA to report on the land capability of the area, part of which examined the suitability of particular areas for subdivision and septic effluent disposal.

The investigation identified 12 separate land units, based on topography, drainage line entrenchment and soil types. (Fig. 18) The capabilities for the various units for urban subdivision are presented in Table 6 (SCA, 1978).

This study highlights the benefits of an initial rapid assessment, for planning purposes. The important aspect is that certain land was recognised as being poorly suited to subdivision.

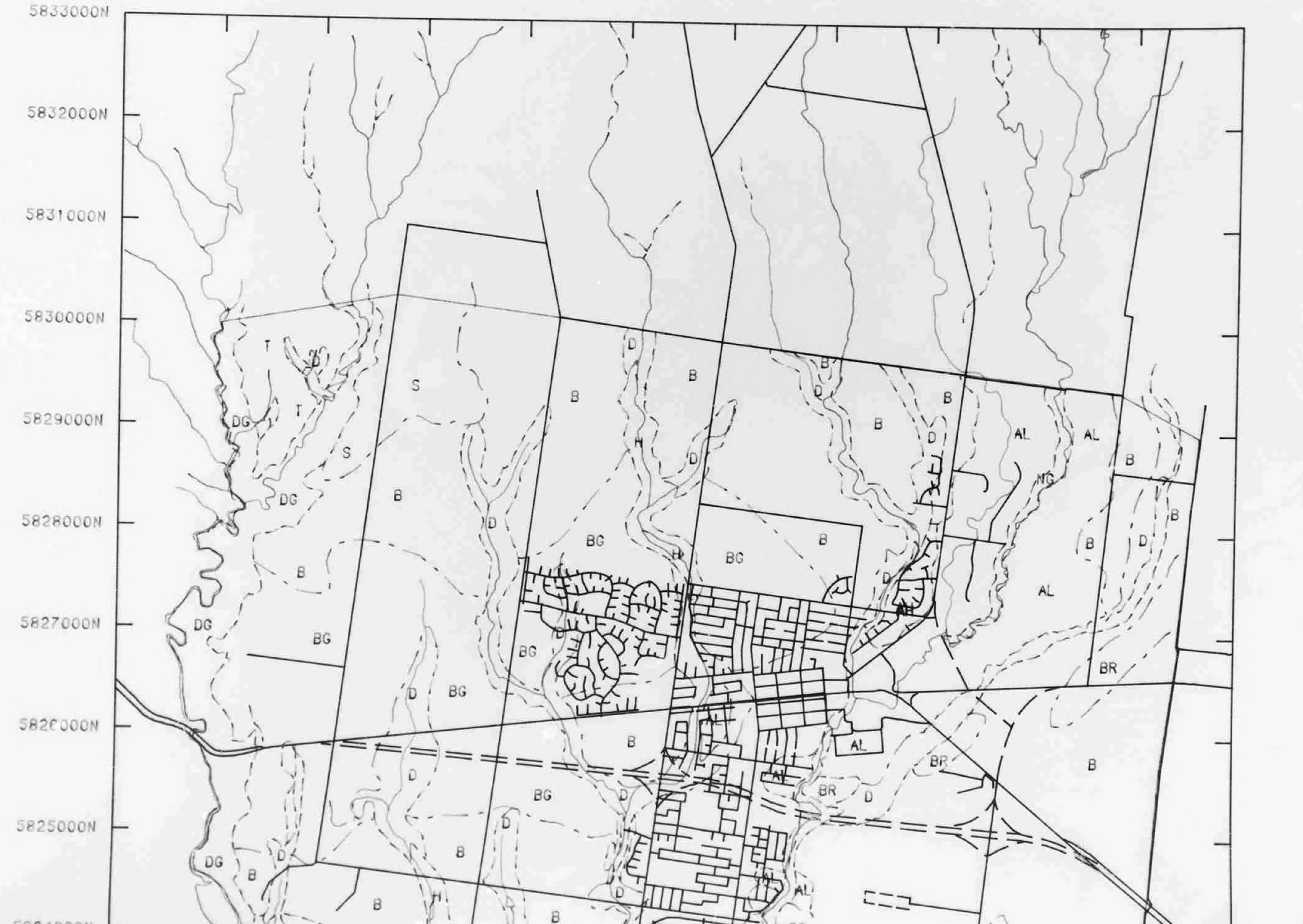
A further study by White and Kelyneck (1985) delineated 32 map units describing a specific topographic element and associated soil type. The capability of the land was then assessed for various land utilisations (viz. secondary roads, septic tank absorption fields, building foundations, farm dams, shallow excavations, rural subdivisions, and urban developments).

This study emphasizes the pitfalls in rapidly producing maps which imply that a detailed study of the area has been made. Examples of poor assessments are easily found - the area affected by gilgai and subsidence 'sinkholes' (U.R. 1986/4) is rated as "good" for building foundations while the area between Toolern Creek and Gisborne-Melton Road (which is here considered good) is rated by the assessment as being "very poor".

4.2 Building Foundations

The geological conditions beneath urban areas provide the ultimate support for all structures in that city. The relevance of the engineering properties of the geological foundation materials has therefore been studied for centuries. Since the evolution of cities in society, the construction of buildings has been subject to regulatory control, often including rules regarding foundation conditions. The Code of Hammurabi (2067-2025 BC) is thought to be the first set of building regulations ever recorded (Leggett, 1973).

In Victoria, the building regulations first incorporated strict control of the foundations for building construction in 1980, when an amendment to the Uniform Building Regulations (UBR) was introduced in response to pressure from the building and insurance industries to decrease the incidence of distress in domestic housing. There were several reasons for this. Firstly, the trend in housing from the 1950's had been toward single-leaf masonry veneer construction ('brick veneer'). This form of construction is less tolerant to movement (i.e. less flexible) than the 'weatherboard', or 'fibro' houses of the pre-1950's. In addition, the growing desire for quality and the increased



NON CREEK ASSOCIATED UNITS

B - Basalt Plain
BG - Gilgaid Basalt Plain
BR - Stony Rises on Basalt
AH - Higher Alluvial Terrace
S - Sinkhole Plain
T - Tertiary Plateau

CREEK ASSOCIATED UNITS

D - Dished Drainage Lines
H - Hilly Sided Drainage Lines
G - Stony Gorge
DG - Deep Gorge
NG - Northern Toolern Creek Gorge
AL - Low Level Alluvium

ENGINEERING GEOLOGICAL MAPPING
OF MELTON, VICT.

Figure 18.
SCA LAND UNITS
(SCA, 1978)

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35

Land Unit and Map Symbol		
Non creek associated units	Capability	Factors causing limitations
Basalt Plain (B)	Fair	Shrink-swell, drainage, rock, erosion during development
Gligaied Basalt Plain (BG)	Fair to Poor	Shrink-swell, drainage, rock, erosion during development
Stony Rises on Basalt (BR)	Fair to Poor	Depth to rock
Higher Alluvial Terrace (AH)	Fair	Shrink-swell, drainage, erosion during development
Sink-hole Plain (S)	Poor	Subsidences
Tertiary Plateau (T)	Fair	Erodibility
Creek Associated Units		
Dished Drainage Lines (D)	Fair	Lower areas flood, shrink-swell, erosion during development
Hilly-sided Drainage Lines (H)	Poor	Slope, depth to rock, flooding on low level terraces
Stony Gorge (G)	Very Poor	Slope, rock outcrop
Deep Gorge (DG)	Very Poor	Steep, unstable banks
Northern Toolern Creek Gorge (NG)	Very Poor	Unstable, steep banks

Table 6. SCA Land Units - Development limitations.

awareness of consumer rights meant that house owners were less forgiving toward signs of distress. Less geologically suitable land was being subdivided as Melbourne's western and south-western suburbs grew. A trend toward tree planting in home gardens, particularly native gardens, also increased the incidence of distress, since most of the problems were with seasonal movement of expansive soils. This same ammendment to the regulations also required that a builder guarantee his/her work for six years.

In 1983 the Victoria Building Regulations (VBR) were introduced to bring Victoria into general compliance with regulations in other states. The relevant section of these regulations covering footings and foundations is included as Appendix III. Essentially these regulations state that foundations are to meet three requirements:

i) Assessment of adequacy - (regulation 32.2)

The adequacy of foundations shall be based on approved

a) well established and relevant local knowledge and experience of foundation conditions in the vicinity of the proposed building; or

b) tests on the foundation materials.

ii) Allowable bearing pressures - (regulation 32.4)

the bearing pressure on the foundation of a building shall not exceed the values given in the regulations unless-

a) an investigation of the foundations has been conducted and the building surveyor is satisfied in the light of the report on that investigation, that higher bearing pressures are justified; or

b) an investigation of the site has been conducted under AS1726 and the bearing pressures are based on the information obtained from that investigation.

iii) Foundation classification (sub-regulations 3 & 4, regulation 33.4)

The classification of the foundation of any site on which footings are to be constructed shall be in accordance with the "Classification of Expansive Behaviour of Melbourne Soils for Domestic Construction" published by CSIRO and AEBIRA (Australian Engineering and Building Industry Research Association). (sub-regulation 3)

In areas outside the Metropolitan area (those shown in figure 3 of the publication referred to in sub-regulation (3)) the site on which the footings are proposed to be constructed shall be similarly classified by adopting, where practicable, the principles stated in that publication and taking into account experience or knowledge of local or traditional building construction practices. (sub-regulation 4)

The last of these three requirements has special relevance at Melton. The classification system relates the expected expansive behaviour of the foundation to the performance of the minimum standard footing design recommendations. Three categories of movement are used, viz. stable, intermediate and unstable.

The soils of Melbourne are subdivided primarily in terms of their geological origin. A simplified map of the major soil types referred to in the CSIRO and AEBIRA publication is reproduced as Figure 19. Soils of the one geological origin are then further subdivided on the basis of their typical soil profile. Table 7 reproduces the classification.

The requirements for the classification of sites and the design and construction of residential slabs and footings are now covered by Australian Standard AS2870-1986 (Residential Slabs and Footings). The standard was prepared in response to an Australia-wide need for guidance on the design of slabs and footings for houses, and although a wide range of conditions is covered, the standard places particular emphasis on the design for reactive clay sites susceptible to significant ground movement due to moisture changes. The standard may be used to satisfy the requirement that the structural design of footings and floor slabs shall take account of the following:

a) Swelling and shrinkage movements of reactive clay soils due to moisture changes.

b) Settlement of compressible soils or fills

c) Distribution to the subgrade of the applied loads

d) Tolerance of the superstructure to movement

The standard sets out the requirements for:

a) the classification of a site; and

b) the design and construction of a footing system, including slab supported on the ground, strip and pad footings or a piled or piered system, which supports a masonry or framed one or two storey house, extension or outbuilding.

The sections of the standard relevant to site classification are included in Appendix IV.

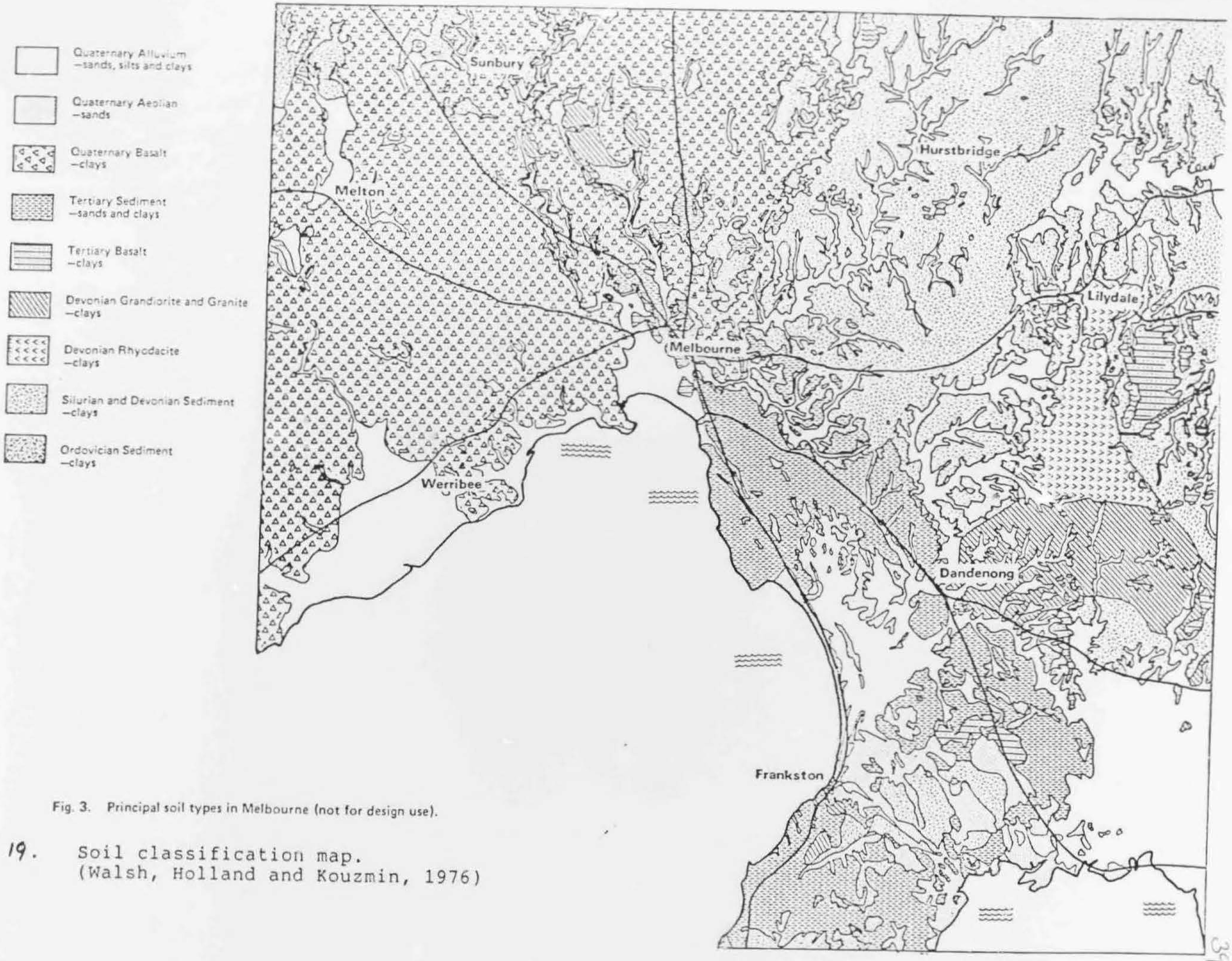


Fig. 3. Principal soil types in Melbourne (not for design use).

Figure 19. Soil classification map.
(Walsh, Holland and Kouzmin, 1976)

Geological description of soil	Approximate depth (mm)	Typical soil profile: Description	Classification: Strip & Stump Footings	Classification: Slabs or Footing Slabs
Quaternary Alluvium – sand, silts and clays		Werribee Delta		
		Geol. Map 7822, Ref. Opw Grant(7) Ref. 52010–01/2		
	0–100	Red brown clayey silt	Stable	Stable
	100+	Red brown silty clay to clay		
		Carrum Swamp		
		Geol. Map 849 7922, Ref. Qvm, 859 Ref Q5. Grant Ref. 52010–00/3		
	0–300	Dark grey sandy topsoil	Stable	Stable
	300–2000+	Grey and brown sandy clay with occasional gravel layers		
	0–600	Black clay topsoil	Stable	Stable
	600–2000+	Grey and brown sand to clayey sand with very occasional sandy clay layers		
	0–300	Black silty sand or clay topsoil	Stable	Stable
	300–2000+	Brown grey yellow sand, silty sand or clayey sand with occasional sandy clay layers.		
		The following profile is uncommon but may occur near creeks and rivers.		
	0–200	Black clay topsoil	Intermediate	Intermediate
	200–2000+	Grey and brown clay		
		Port Melbourne – South Melbourne Area		
		Geol. Map Qrs		
	0–600	Black stratified silty clay	Stable	Stable
	600+	Sand silt or silty clay		
		Geol. Map Qrp		
	0–400	Dark grey sandy topsoil	Stable	Stable
	400–2000	Grey sand		
Quaternary Aeolian – sands	0–150	Dark grey brown silty sand topsoil	Stable	Stable
	150–300+	Light yellow grey sand		
	300+	Various clays		
	0–2000+	Uniform grey to dark grey sand	Stable	Stable
Quaternary Basalts – clays	0–100	Brown to black clay topsoil		
	100–rock	Brown to black highly plastic clay may contain floaters		
		(a) Deeper clay soils (> 1 m) or soils for which local knowledge indicates past problems.	Unstable	Intermediate
		(b) Shallower clay soils or soils for which local knowledge indicates satisfactory past performance.	Intermediate	Intermediate
		(c) Very shallow soils (< 200 mm clay) where edge beams of slabs or footings may be founded on rock.	Stable	Stable
	0–150	Brown to light brown silty clay topsoil	Intermediate	Intermediate
	150–rock	Red to red brown clay		
Tertiary Sediments – sands and clays	0–1000+	Deep uniform grey sand	Stable	Stable
	0–200	Grey silty topsoil	Stable	Stable
	200–500	Grey to yellow sand or clayey sand		
	500–rock	Light grey to yellow sandy or silty clay		
	0–300	Dark brown or grey sandy topsoil	Stable	Stable
	300–800	Loose brown or grey sand		
	800+	Brown clay to sandy clay generally becoming more sandy with depth		
	0–300	Brown sandy topsoil	Stable	Stable
	300–1000	Loose brown sand		
	1000+	Medium dense red brown, or brown sand with occasional gravel layers		
	0–600	Black sandy topsoil	Stable	Stable
	600+	Yellow brown and grey sandy clay, becoming more sandy with depth.		
Tertiary Basalts – clays	0–400	Dark grey to reddish clayey topsoil	Intermediate	Intermediate
	400–rock	Dark grey or reddish brown to brown highly plastic clay		
Devonian Granodiorite and Granite – clays	0–200	Grey or brown sandy to silty topsoil	Stable	Stable
	200–600	Grey and brown silty sand to clayey silt		
	600–rock	Mottled red, brown and grey clay		
Devonian Rhyodacite – clays	0–200	Grey to brown silty topsoil	Stable	Stable
	200–700	Grey brown, yellow brown or red brown silty or sandy clay, mottled		
	700–rock	Mottled red brown and grey yellow brown or orange clay generally stiff to very stiff, may contain silt, sand or gravel		
Ordovician, Silurian and Devonian Sediments – clays	0–100	Grey or grey brown silty topsoil	Stable	Stable
	100–400	Grey, grey brown or yellowish silt to silty clay. Soft when wet, hard if dry.		
	400–rock	Mottled yellow grey or reddish brown clay		

* This table may be applied to solid masonry construction if the internal walls are articulated (for example, by the use of full height door openings) otherwise the standard designs may be inadequate.

SOILS CLASSIFICATION FROM GEOLOGICAL ORIGIN AND TYPICAL SOIL PROFILE FOR MASONRY VENEER OR TIMBER CONSTRUCTION

Table 7. Soil classification table. (Walsh, Holland and Kouzmin, 1976)

Essentially, the site classes are designated as follows:

Foundation	Character	Class
Sand and rock	Stable	A
Silt and some clay		S
Moderately reactive clay	Reactive	M
Highly reactive clay		H
Extremely reactive clay		E
Sand	Controlled fill	A
Material other than sand		A to P
Mine subsidence	Problem	P
Uncontrolled fill		
Landslip		
Soft		
Collapsing soil		

Table 8. Site Classes (AS2870-1986).

All site classifications for Victoria are based on one or more of the following:

a) Assumption of a soil type without any site investigation from a classification map or from well established local knowledge provided that soils are known to be consistent over large areas. The soil type and site conditions shall be checked by a site visit before construction.

b) Site investigation to identify soil profile using one or more boreholes or test pits in the site or a number distributed over a subdivision.

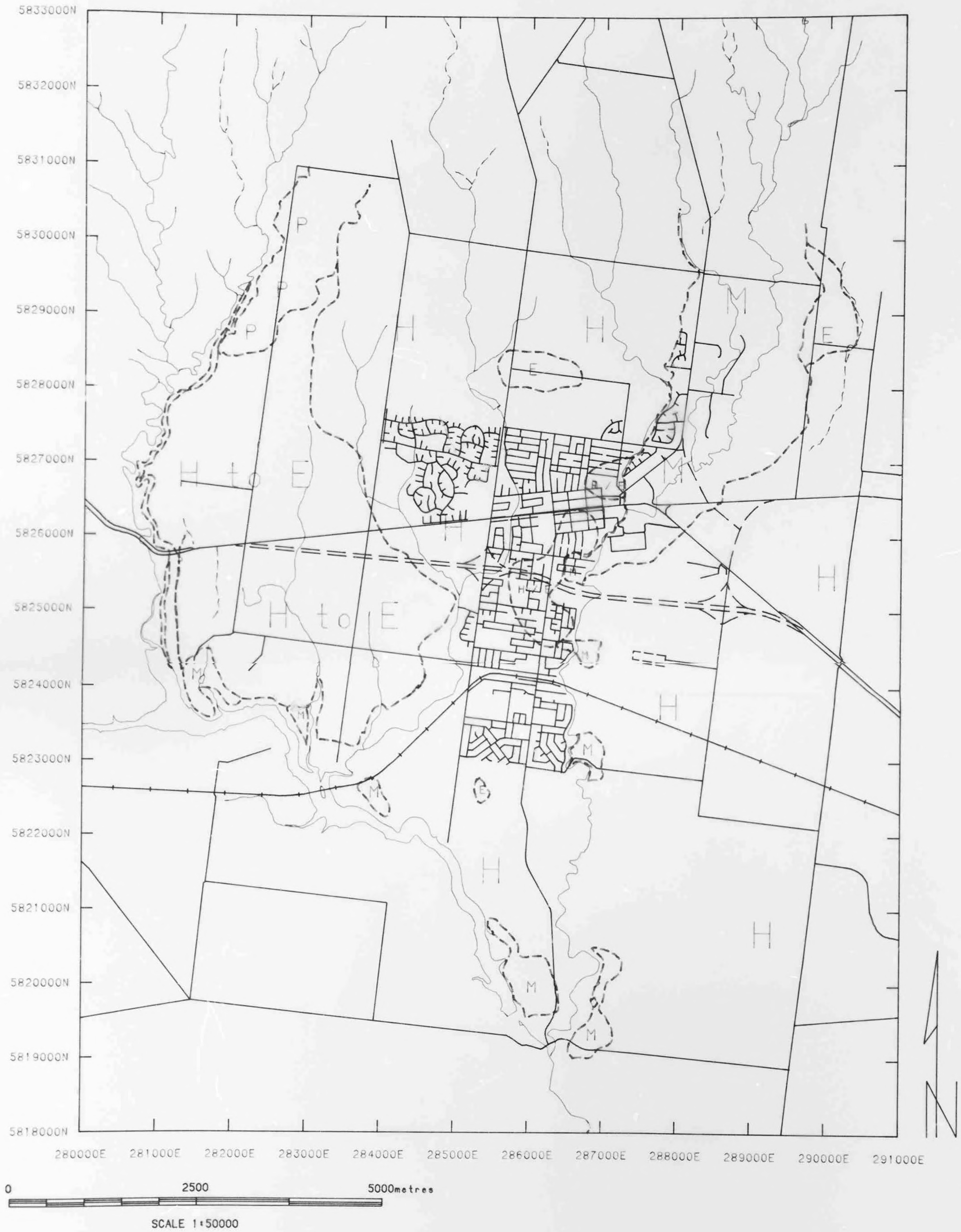
c) Site investigation using a penetrometer, for sand sites.

d) Site investigation including soil sampling and appropriate tests.

Where a Building Authority has designated a presumed site classification or simplified system based on a map of site classifications, this may be used but shall not preclude the adoption of a less severe classification if supported by a site investigation and a classification in accordance with the standard.

This standard is not referred to in the VBR at present, however it is intended to be incorporated if possible (E. Carroll, pers. comm., 1986).

An attempt to map the site classifications referred to in AS2870 is illustrated in Figure 20. This map is derived from a combination of soil depth, swell potential and soil genesis. The site classification method, then, is by soil profile identification (Appendix C, AS2870) rather than by surface movement calculation.



SITE CLASSIFICATION -
AUSTRALIAN STANDARD AS2870

Foundation	Character	Class
Moderately reactive clay	Reactive	M
Highly reactive clay		H
Extremely reactive clay		E
Subsidence area or uncontrolled fill	Problem	P

ENGINEERING GEOLOGICAL MAPPING
OF MELTON, VICT.

Figure 20.

SITE CLASSIFICATION

PROJ NO. GE 14 | DATE: 5-JAN-88

4.2.1 Class P (problem) sites

The area described in Unpub. Rept. 1986/4 - the 'sinkhole plain' - is the largest of the problem sites identified within the Melton development area. The most obvious problem for development of this site is the active subsidence. Roads, housing, and sewerage systems would be seriously damaged by sinkhole development. The unpredictability of the sinkholes makes the siting of any structures hazardous. It is likely however, that areas of the site could be found where the potential for subsidence would be lower.

The soil subsidence occurring in this area would classify the site as a class P (problem) site according to AS2870. The presence of deep basaltic clay classifies the site as "unstable" for strip and stump footings and "intermediate" for slabs or footing slabs according to the Victoria Building Regulations (VBR). The regulations set out the minimum dimensions for either type of footing, which are not condidered adequate for this site. The depth of highly expansive clay and the potential for sinkhole development create unusual foundation conditions which require special engineering design for footings.

Buildings founded on expansive soil need careful attention paid to building design and maintenance, in order to mitigate or control structural damage. Properly engineered foundations, segmented interior design, flexible connections to utility lines, and carefully designed lot drainage and landscaping are required for satisfactory building performance.

Selection of building sites in areas where the soil is thinnest and removal of all trees surrounding buildings would lessen the risk of subsidence occurring, although not entirely rule it out. Placement of the footings on the rock (by designing pier and beam footings) would ensure that the building would not subside, even though the soil may. Chen (1975) warns that pier and beam design does not always work in expansive soils, since the swelling and shrinking can produce considerable lateral and frictional forces on the piers.

An alternative solution would be to replace the foundation soils with non-swelling granular soils. Chen (1975) suggests at least 1.5m under the footings and 3m beyond the building line Soil replacement will lessen the chances of building distress considerably since it would overcome the effects of the expansive soils and cushion the effect of any subsidence. The possibility of subsidence occuring still remains, although the effects would be less dramatic on the surface due to the compensatory movement of the granular soil.

Other problem sites identified are very small areas where farm dams seen on aerial photographs taken in 1943 and 1967 have been infilled.

4.3 Sewerage

An investigation of the usefulness and limitations of various methods of treating or disposing of domestic waste waters was carried out by the Environment Protection Authority (EPA) and reported on in 1975. The report found that reticulated sewerage is the only really satisfactory method of dealing with domestic waste water discharges on long-term basis. However, in non-sewered areas several alternatives were available for waste disposal (Table 9). For environmental reasons only the "all-waste" treatment systems are considered suitable, as the previous practice of sullage disposal by direct discharge to creeks or drains is no longer acceptable.

Of the all-waste disposal units the septic tank with soil absorption has some specific geological requirement. Although in principle, surface irrigation with effluent is possible, normal domestic waste disposal do not use this method for both aesthetic and health reasons. Nearly all septic tanks using soil absorption of effluent use absorption trenches. In the ground absorption process, the soil factors which determine the rate of absorption of water are:

- infiltrative capacity of the liquid soil interface
- percolative capacity of the soil itself
- effective soil particle size
- trench loading

The first two factors determine the rate at which liquid enters the soil and can percolate away, and therefore play the major part in absorption process. Treated effluent however is considerably different in composition to pure water, and for the case where a septic tank effluent is being applied the infiltration rate is always less than the percolation rate due to clogging of the interface with suspended matter and biological growths, as well as a swelling of hydrated soil particles and deflocculation by added sodium or potassium ions. (EPA, 1975a)

Measurement of infiltrative capacity is difficult and it is usual to measure percolation rates or soil permeability (hydraulic conductivity) instead. Both the Victorian Health Commission and the EPA have used percolation tests to assess infiltrative capacity. The EPA has developed a standard test to measure the percolation rate of soils in relation to septic effluent absorption (EPA, 1975b). This procedure is based on that of the U.S. Public Health Service, and simply entails excavating a standard hole in the ground, soaking the soil in the hole for a minimum time, and then measuring the percolation rate of the soil as a drop in water level in the hole over a standard time.

Research of three methods of measuring soil permeability (Winneberger, 1974) shows poor reproducibility when using the percolation test method. Past experience in performing many of these tests has led to a disregard for the usefulness of the test. The results vary markedly according to the soil fabric, season and site specific location.

WC only

* Septic tank plus: soil absorption
  sand filter
  transpiration bed
  chlorinated discharge

* Incenerator systems

* Humus toilets

* Storage in holding tank - periodic removal

* Chemical stabilisation - storage and removal

* Cesspits

* Pan service

All Waste

* Septic tank plus: soil absorption
  sand filter
  transpiration bed
  lagooning
  chlorinated discharge

* Small treatment plant plus: chlorinated discharge
  lagooning
  transpiration bed
  soil absorption
  sand filter

Sullage Only

* Septic tank plus: soil absorption

* Soakage pit

Table 9. Current domestic waste water disposal methods.

The soil requirements for absorption of septic tank effluent are as follows:

- moderate to high permeability (above 10-5 cm/sec)
- percolation rate greater than 2.5 cm/hr.
- low clay content
- low shrink/swell potential
- not subject to flooding
- no shallow impermeable horizons
- ground water-table at least 1m below the trench bottom (EPA ,1975c)

For the development area, it is intended that most of the dwellings will be conected to a reticulated sewerage system. However, in the low density areas, some septic tank systems may be installed. For these reasons no regional assessment has been made of suitability of effluent absorption. In general, clay soils are not very good because of their expansive nature and very low permeability.

The area in the north west corner (the 'sinkhole plain') presents a difficulty for sewage disposal. Because the site is isolated from the Melton City by a proposed regional cemetery, is was not intended by the ULA to service the site with reticulated sewerage.

Alternative sewage disposal would be limited to above ground methods (e.g. composting, chemical or incinerating toilets, "grey water" irrigation, etc.), since the permeability of the clay is too low to provide adequate effluent absorption. Even in areas where septic system absorption lines could be located in the gravel/sand/silt layer, the localised addition of moisture to the underlying expansive clay would cause excessive swell and distress in the sewerage system. Similarly, sand filters or other in-ground disposal would be ultimately unsatisfactory.

4.4 Roadmaking

In Victoria the common practice in roadmaking has been to use flexible pavements for most highway and suburban roads. The PCA have published guidelines for the design of flexible pavements which are used by road engineers in Victoria (CRB, 1980a).

In the process of arriving at a pavement thickness and composition it is necessary to consider many factors. These may be classified into five broad categories:

(i) Function of the road
(ii) Traffic loading
(iii) Subgrade conditions
(iv) Properties of the available pavement materials
(v) Drainage conditions

The conditions of the subgrade is the most relevant to the engineering geological mapping in the Melton development area. In general, the support provided by the subgrade is the most important factor in determining pavement design thickness, composition and performance (CRB, 1980a). The subgrade should be prepared and compacted so that its long term bearing strength is as uniform and as high as possible. In situ strengths during construction may differ greatly from the strengths ultimately developed at the equilibrium moisture content.

The long term strength of the subgrade is governed by:

- the type of material
- the subgrade moisture regime
- the sensitivity of the subgrade to moisture changes

Of these factors, the last is of particular importance at Melton. The extensive expansive clay soils have the potential to appreciably change volume with changes in moisture. This swelling or shrinkage of expansive clays is rarely uniform and the resulting distortion can severely damage an otherwise sound pavement.

Volumetric changes can be minimised by:

1. Minimising changes in the moisture content, eg. compacting the subgrade at a moisture content close to the equilibrium moisture content, and maintaining it at this level until covered by the pavement.

2. Placing sufficient weight of material over the subgrade to counteract the swelling pressure.

3. Modifying the subgrade to reduce its sensitivity to moisture by the addition of a stabilising agent such as lime.

The RCA have extensively tested the Melton soils for the construction of the Western Freeway Melton By-pass and opted for lime stabilization as a suitable soil treatment. The addition of lime (approx. 4%) to the subgrade material greatly improves the roadmaking properties.

The pavement thickness design procedure described by the RCA is based on an empirical relationship between:

- the strength of the subgrade in terms of its CBR (section 3.11.3), and

- the pavement thickness required over the subgrade to carry the predicted traffic loading at the desired level of performance.

APPENDIX I

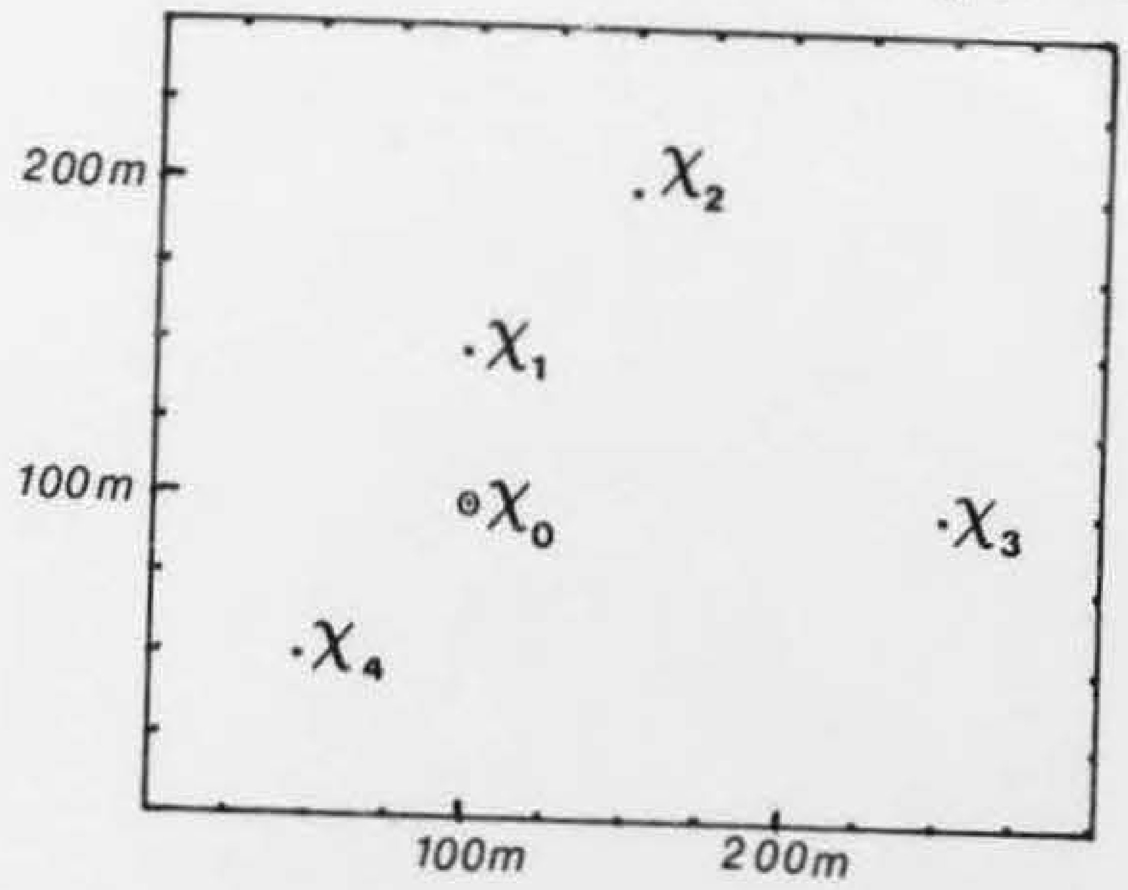
SIMPLE KRIGING EXAMPLE

(Modified from Farrelly, 1985)

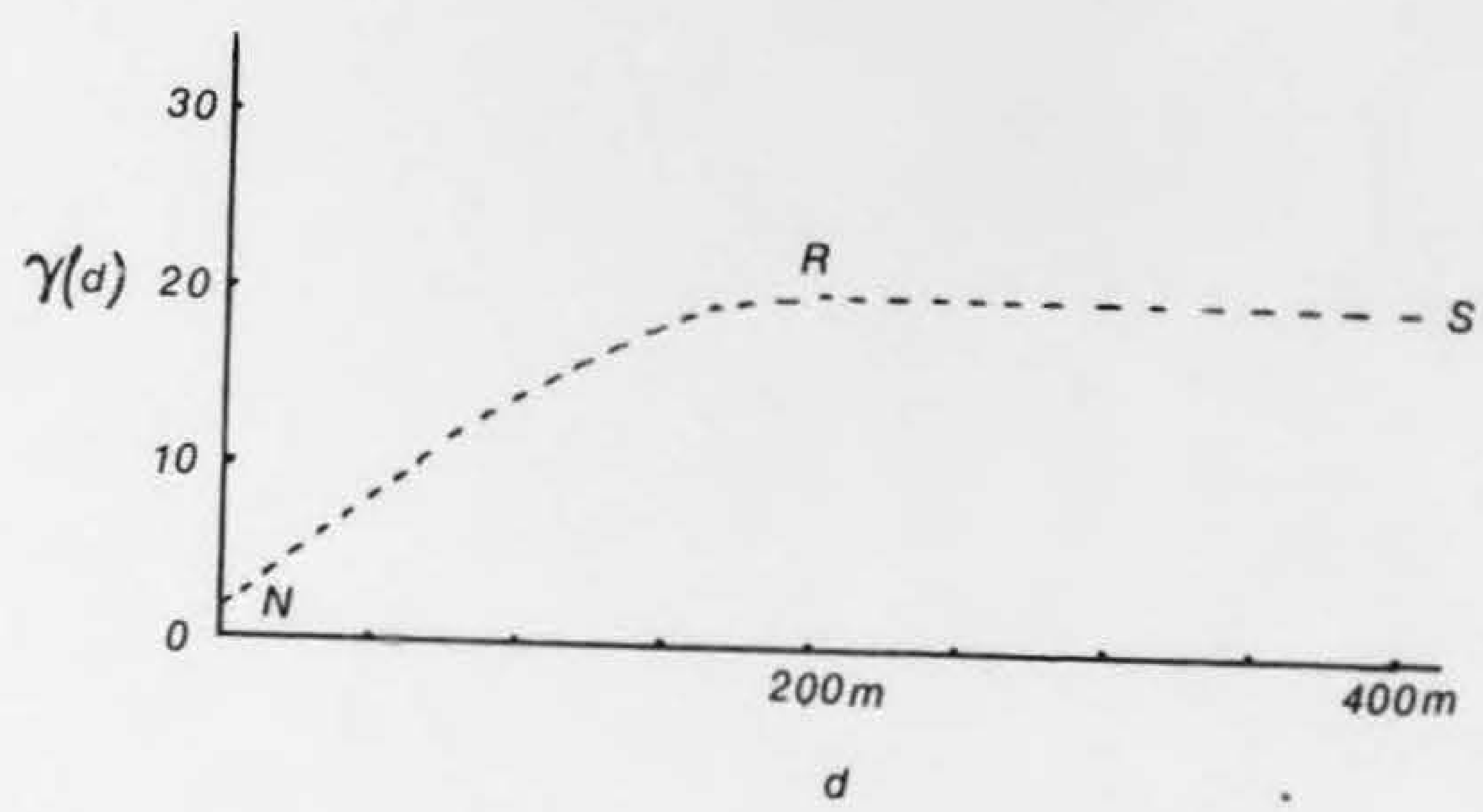
SIMPLE KRIGING EXAMPLE

Assuming isotropic and stationary semi-variances, all that is required to calculate the kriging weights is the semi-variogram model and the arrangement of the data points with respect to the point, area or volume being estimated.

Take the following area, with data points distributed as shown :



Given the semi-variogram for the whole area :



$$\gamma(d) = 18\left[\frac{3}{2}\frac{d}{200} - \frac{1}{2}\frac{d^3}{200^3}\right] + 2 ; \quad \text{for} \quad d \leqslant R$$

$$\gamma(d) = 20 ; \quad \text{for} \quad d > R$$

The point χ_0 is estimated using the weighted average :

$$\widehat{\chi}_0 = \lambda_1 \chi_1 + \lambda_2 \chi_2 + \lambda_3 \chi_3 + \lambda_4 \chi_4$$

where $\lambda_1 + \lambda_2 + \lambda_3 + \lambda_4 = 1$

In matrix form, the solution to the set of kriging equations is written:

$$\begin{bmatrix} \lambda_1 \\ \lambda_1 \\ \lambda_1 \\ \lambda_1 \\ \mu \end{bmatrix} = \begin{bmatrix} \gamma_{1,1} & \gamma_{1,2} & \gamma_{1,3} & \gamma_{1,4} & 1 \\ \gamma_{2,1} & \gamma_{2,2} & \gamma_{2,3} & \gamma_{2,4} & 1 \\ \gamma_{3,1} & \gamma_{3,2} & \gamma_{3,3} & \gamma_{3,4} & 1 \\ \gamma_{4,1} & \gamma_{4,2} & \gamma_{4,3} & \gamma_{4,4} & 1 \\ 1 & 1 & 1 & 1 & 0 \end{bmatrix}^{-1} \times \begin{bmatrix} \gamma_{0,1} \\ \gamma_{0,2} \\ \gamma_{0,3} \\ \gamma_{0,4} \\ 1 \end{bmatrix}$$

where

$$\gamma_{1,1} = \gamma(d_{1,1}) = \gamma(0) = 0$$
$$\gamma_{1,2} = \gamma(d_{1,2}) = \gamma(70{\cdot}7) = 11{\cdot}1$$
$$\gamma_{1,3} = \gamma(d_{1,3}) = \gamma(50) = 8{\cdot}6$$

etc. etc.

The solution to the example is $\begin{bmatrix} .51 \\ .03 \\ .09 \\ .37 \\ .87 \end{bmatrix}$

Thus our estimate is :

$$\chi_0 = .51\chi_1 + .03\chi_2 + {\cdot}09\chi_3 + {\cdot}37\chi_4 + \epsilon$$

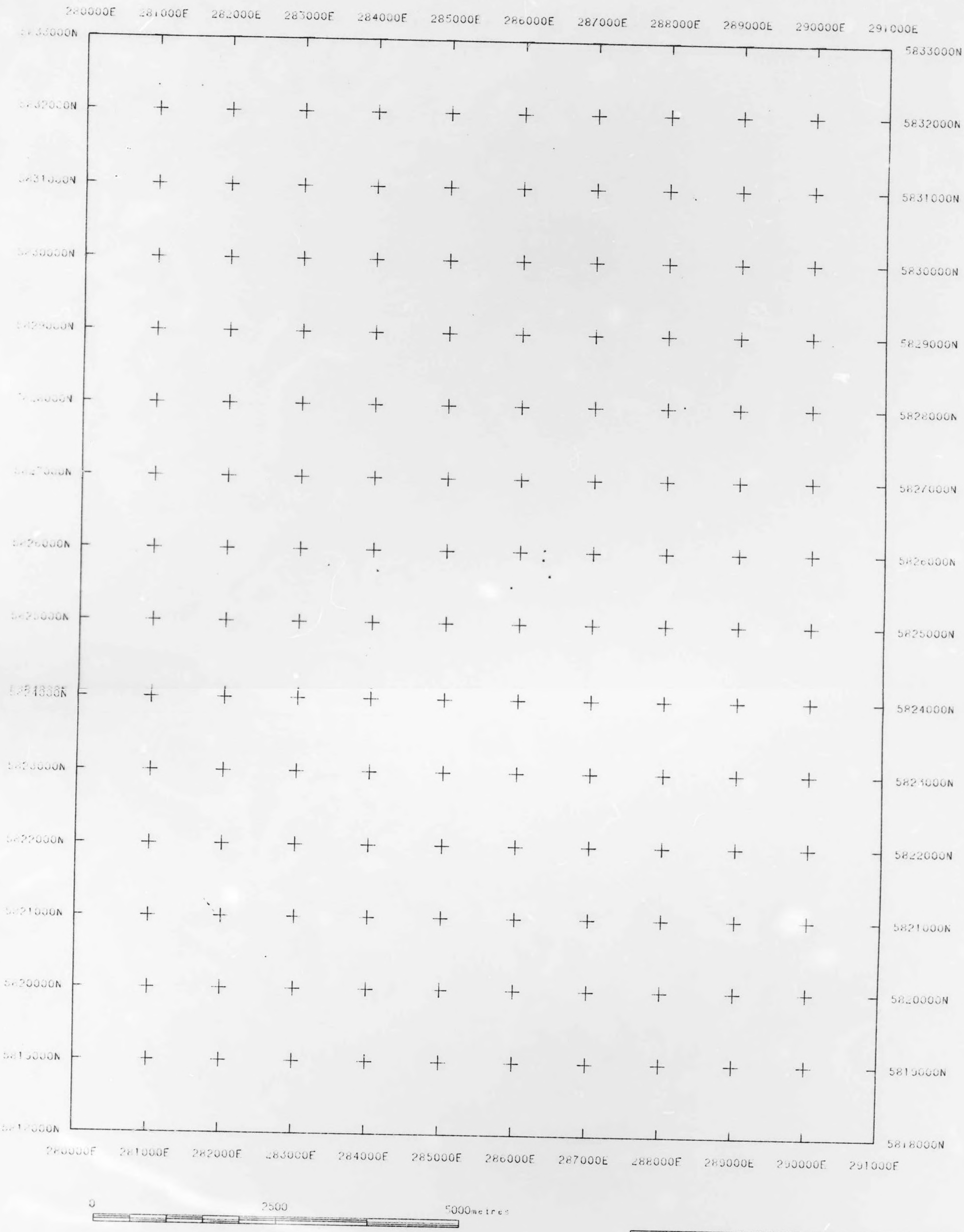
with the variance of ϵ , the kriging variance, being :

$$\sigma_k^2 = \mu + \sum_{i=1}^{n} \lambda_i \gamma_{i,o} = 11{\cdot}5$$

This is a measure of estimation error, and the ability to derive such a measure is one of the advantages of kriging. Another advantage is the automatic down-weighting of samples in a direction in which we already have information. This 'screen effect' can be seen in our example where χ_2 has a lower weighting than χ_3, even though it is closer to χ_0.

APPENDIX II

CHECK PLOTS FOR SOIL DEPTH



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LEGEND

• DATA POINT

N

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ENGINEERING GEOLOGICAL MAPPING
OF MELTON, VICTORIA.

ROCK MORE THAN 7m DEEP

PROJ NO.

DATE: 17-SEP-85

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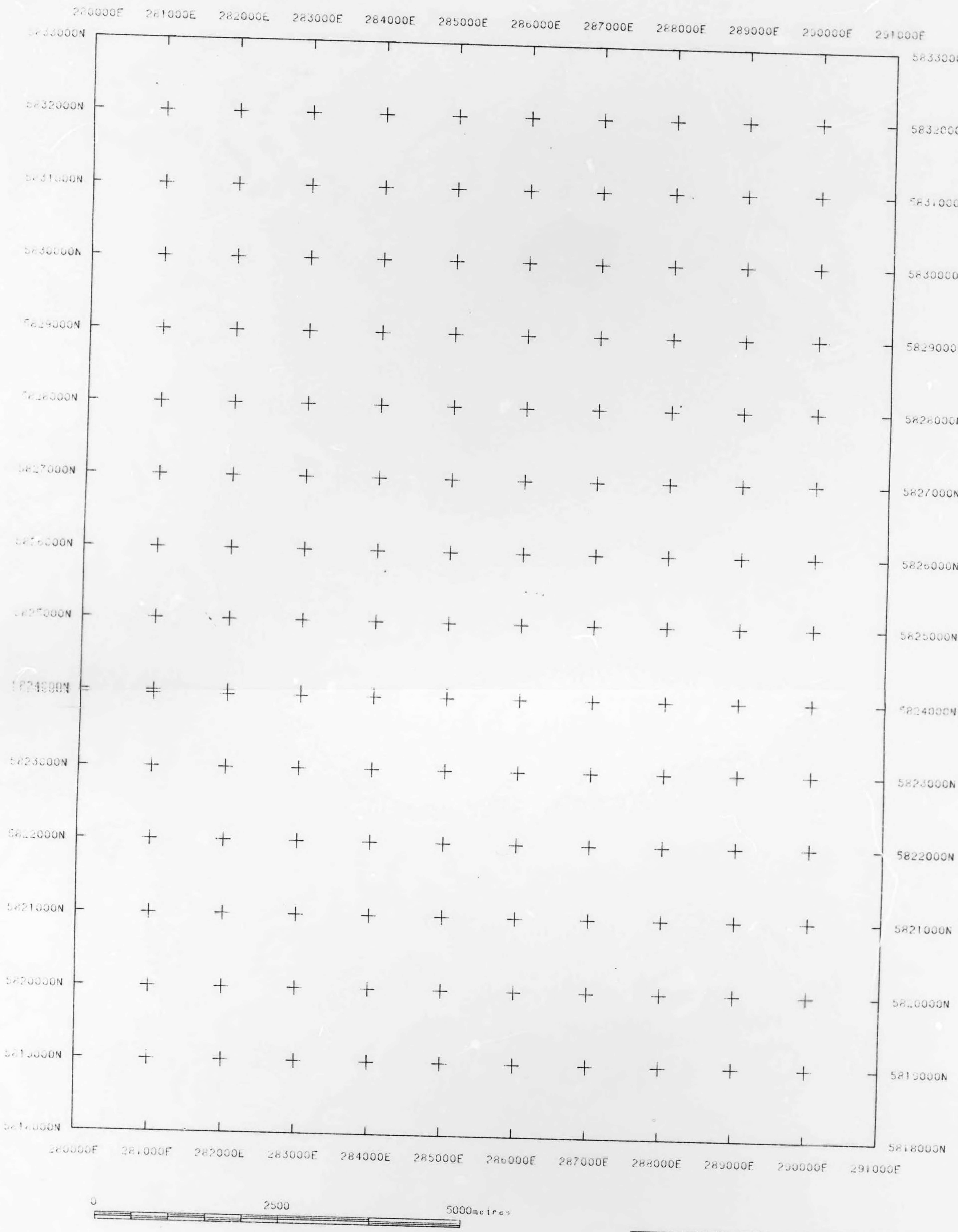

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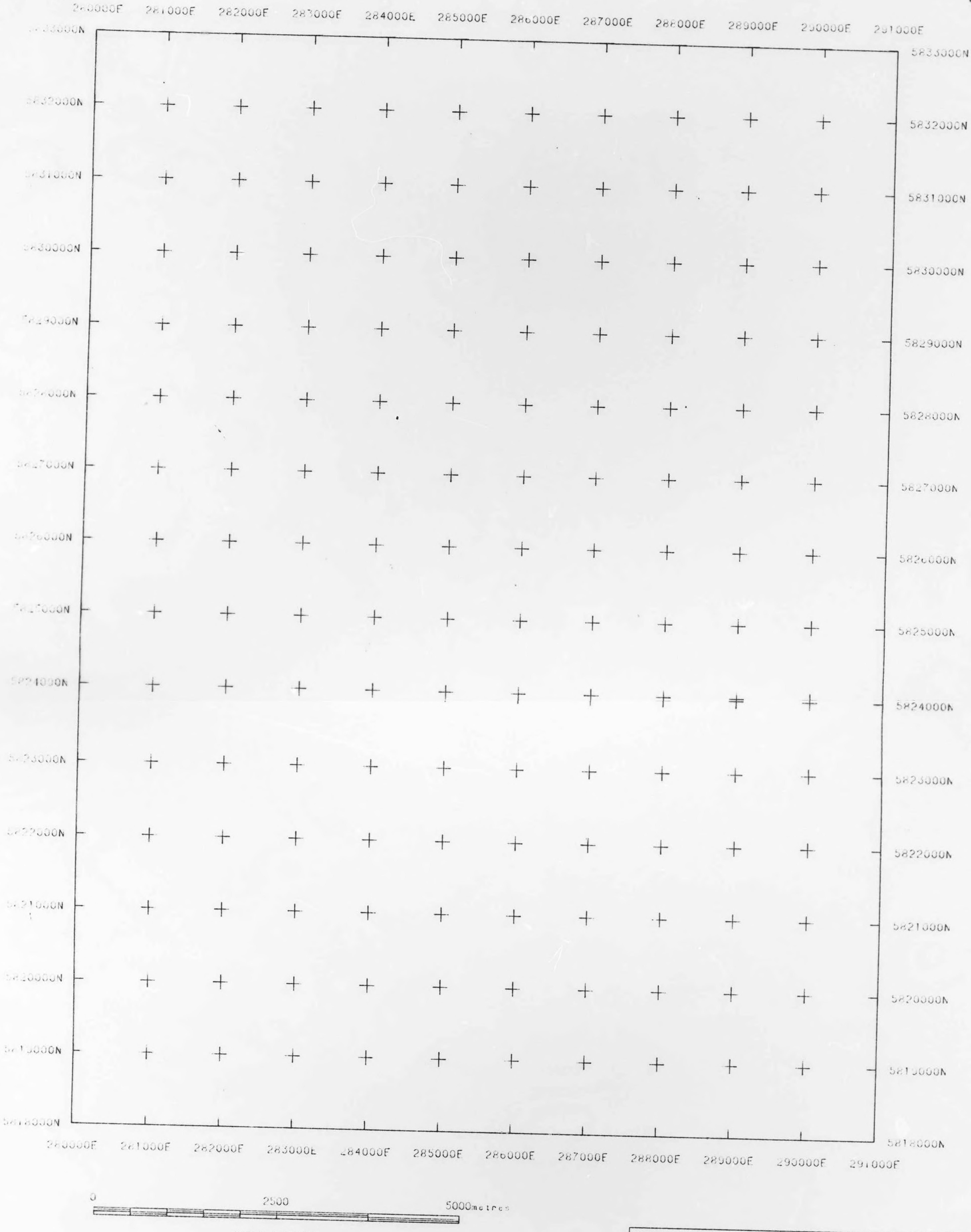
SOIL DEPTH ROCK MORE THAN 6m DEEP	
PROJ NO.	DATE: 17-SEP-85



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LEGEND

• DATA POINT

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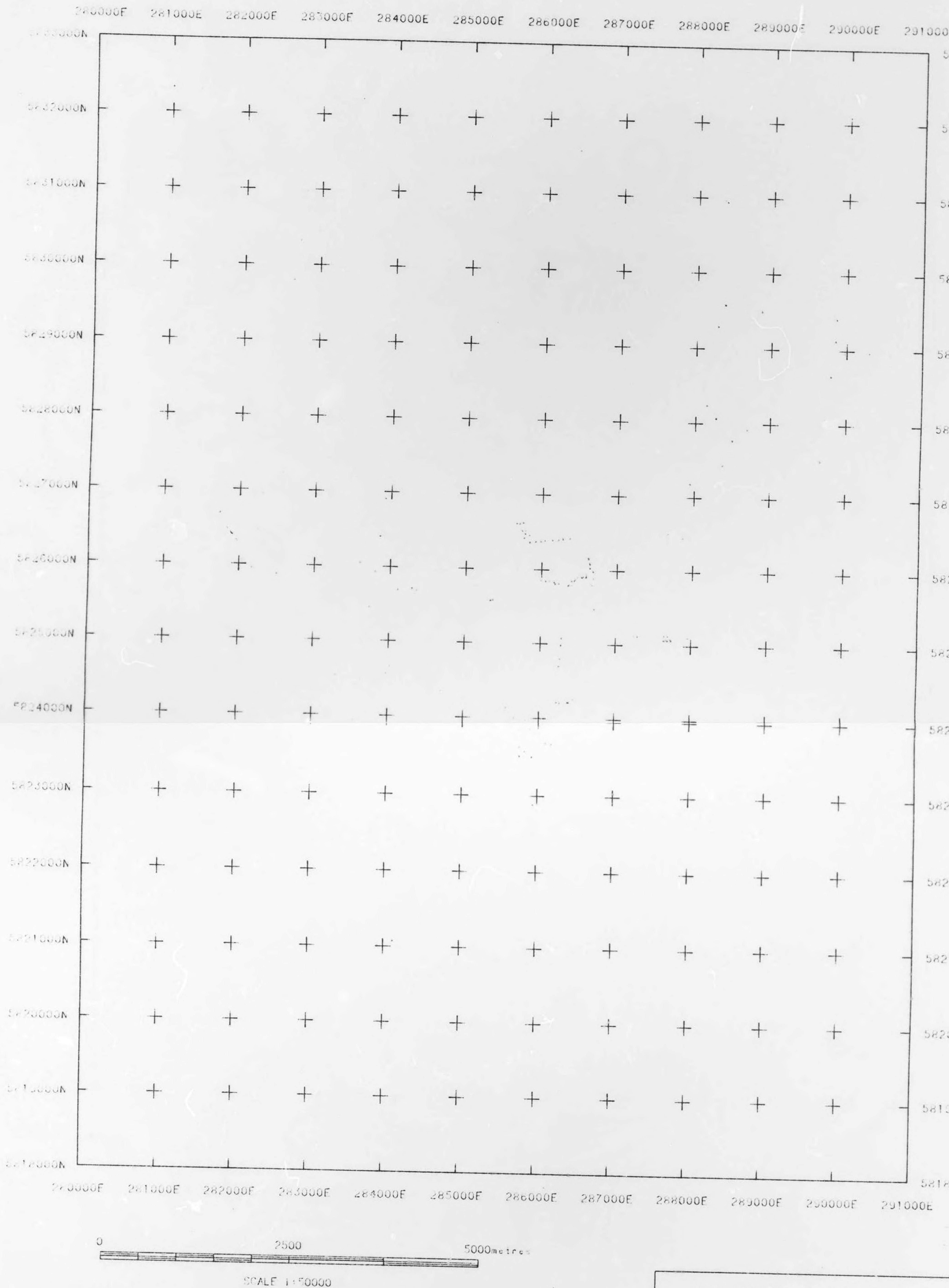
GEOLOGICAL SURVEY OF VICTORIA

ENGINEERING GEOLOGICAL MAPPING
OF MELTON, VICTORIA.

ROCK MORE THAN 5m DEEP

PROJ NO.	DATE: 17-SEP-85



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• DATA POINT

N

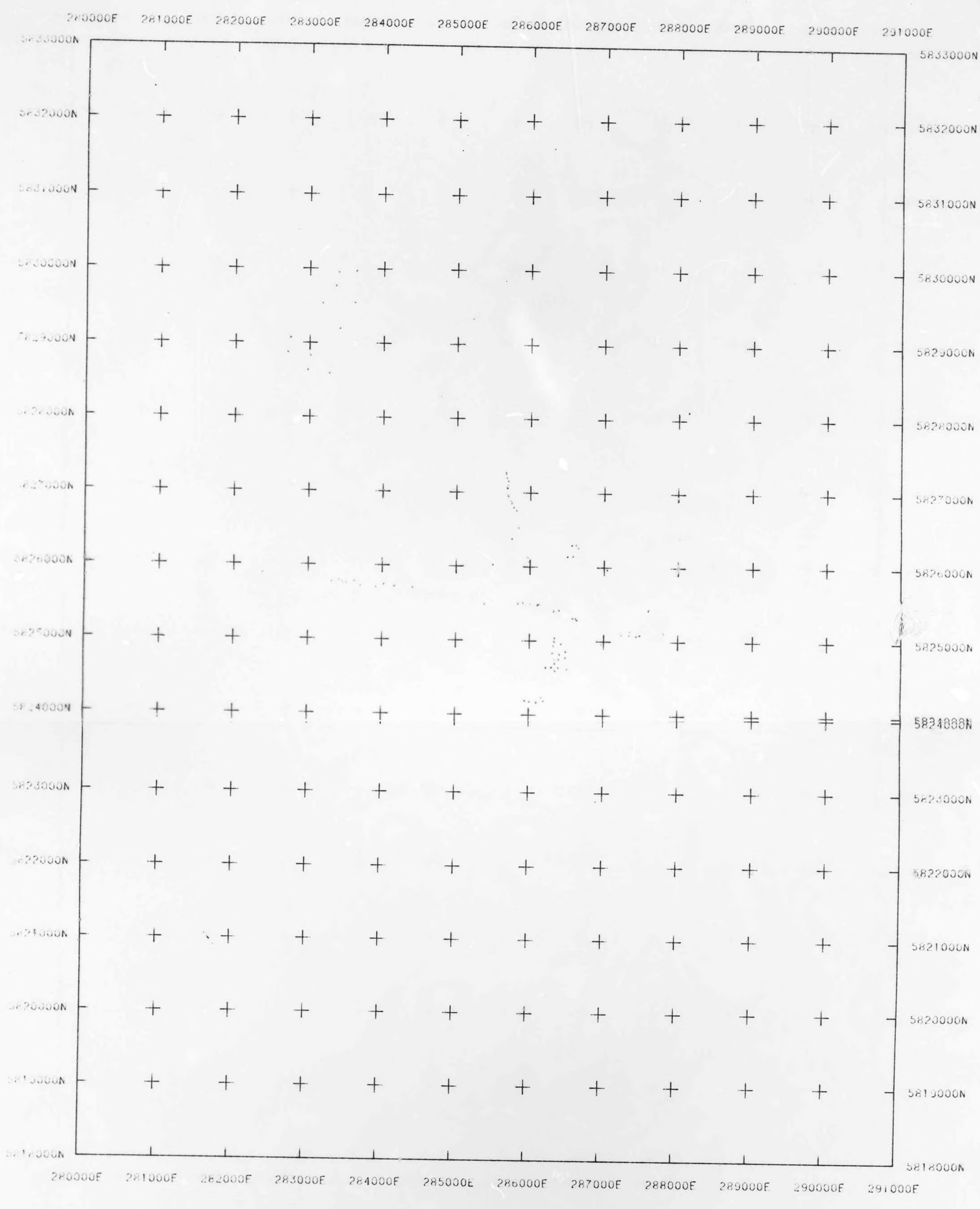
GEOLOGICAL SURVEY OF VICTORIA

ENGINEERING GEOLOGICAL MAPPING
OF MELTON, VICTORIA.

SOIL DEPTH

ROCK MORE THAN 4m DEEP

PROJ NO.	DATE: 17-SEP-85



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SCALE 1:50000

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• DATA POINT

N

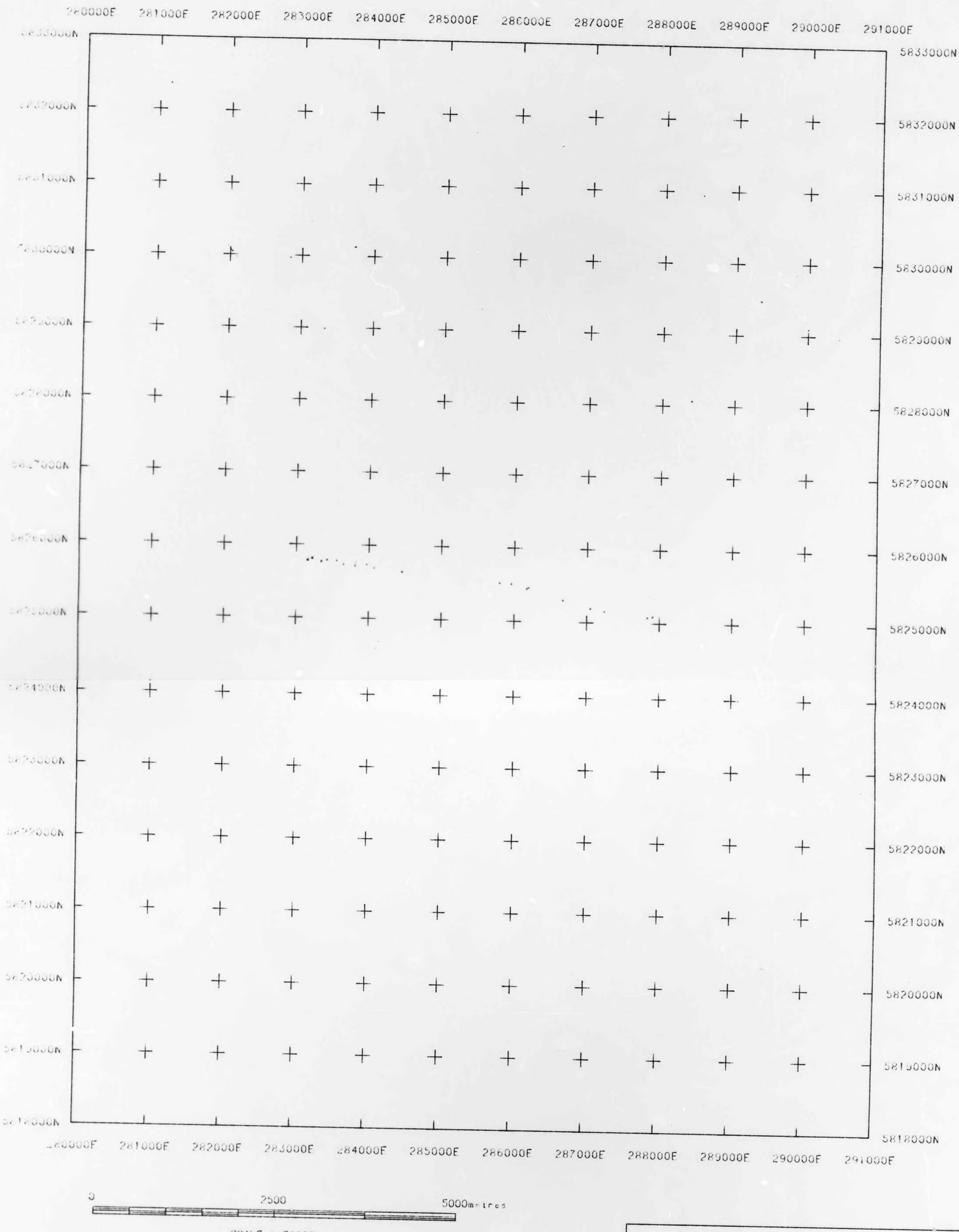
GEOLOGICAL SURVEY OF VICTORIA

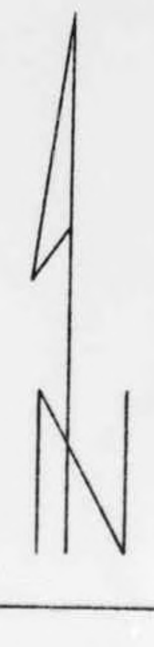
ENGINEERING GEOLOGICAL MAPPING OF MELTON, VICTORIA.

SOIL DEPTH

ROCK MORE THAN 3m DEEP

PROJ NO.	DATE: 17-SEP-85





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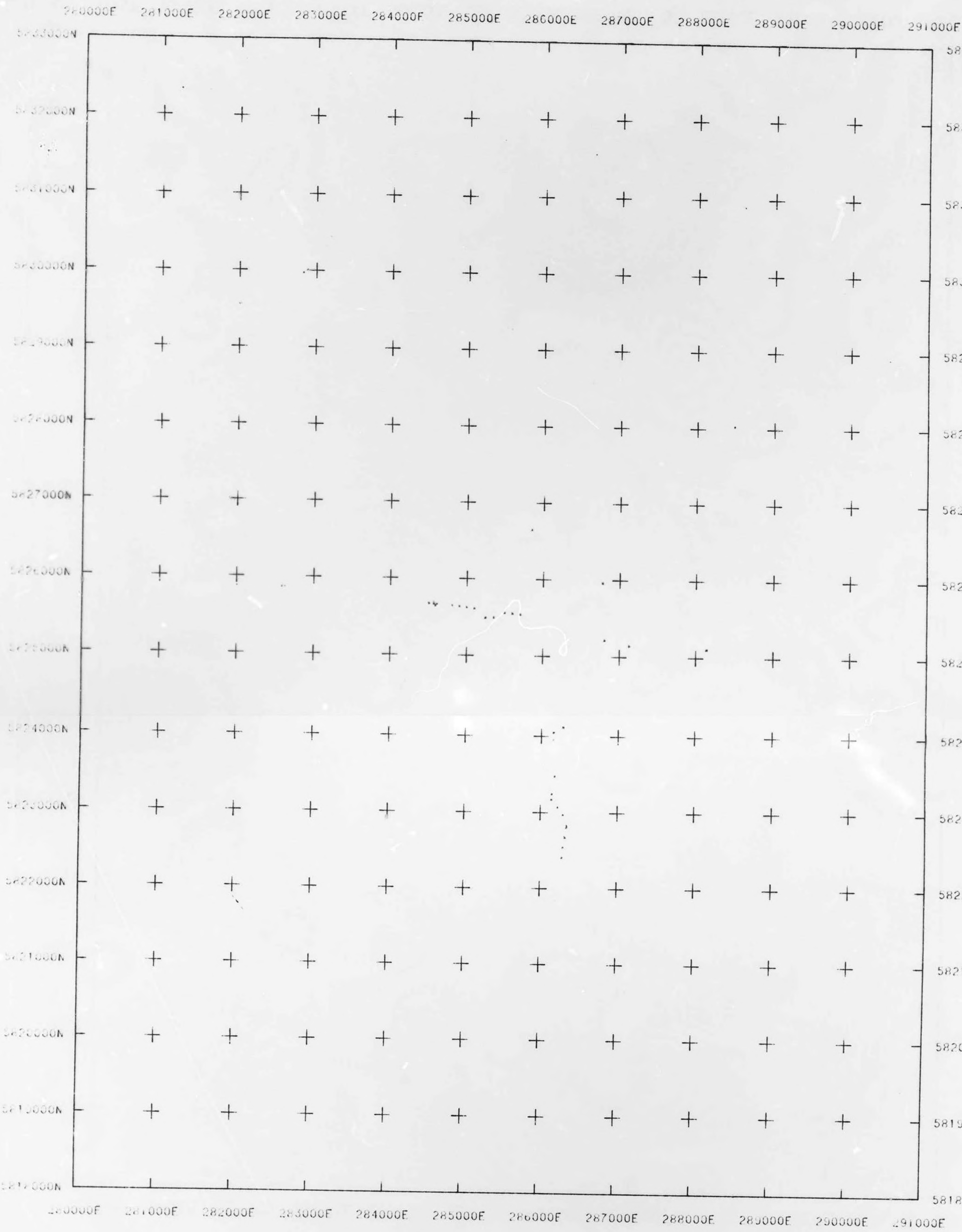
SOIL DEPTH ROCK MORE THAN 2m DEEP

PROJ NO.	DATE: 17-SEP-85



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• DATA POINT

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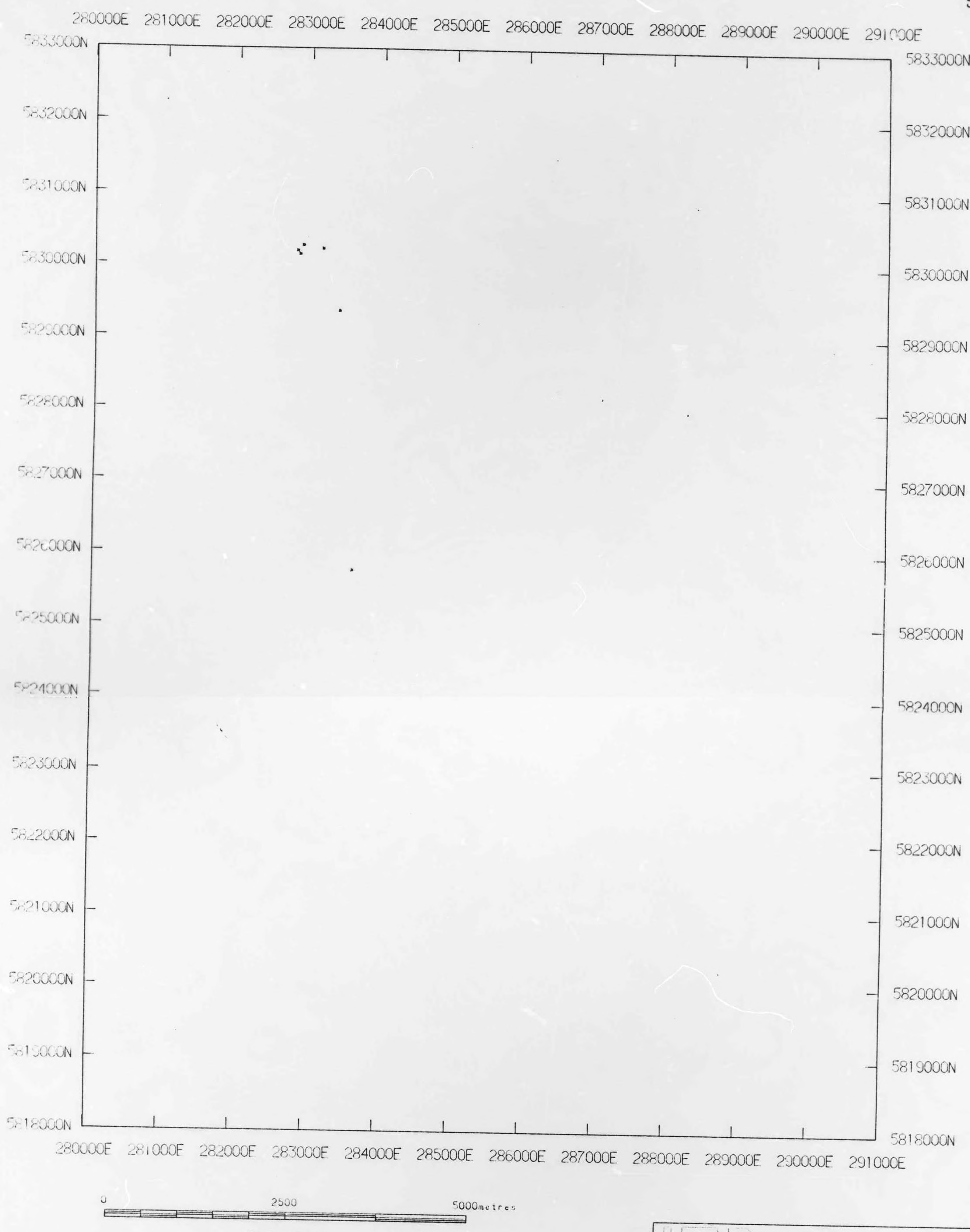
ENGINEERING GEOLOGICAL MAPPING OF MELTON, VICTORIA.

ROCK MORE THAN 1m DEEP

PROJ NO.	DATE: 17-SEP-85



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LEGEND

• SOIL DEPTH DATA POINT WHERE DEPTH IS BETWEEN NINE AND TEN METRES

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ENGINEERING GEOLOGICAL MAPPING OF MELTON, VICTORIA.

SOIL DEPTH

PROJ NO.	DATE: 15-JAN-86

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LEGEND

• SOIL DEPTH DATA POINT WHERE DEPTH IS BETWEEN EIGHT AND NINE METRES

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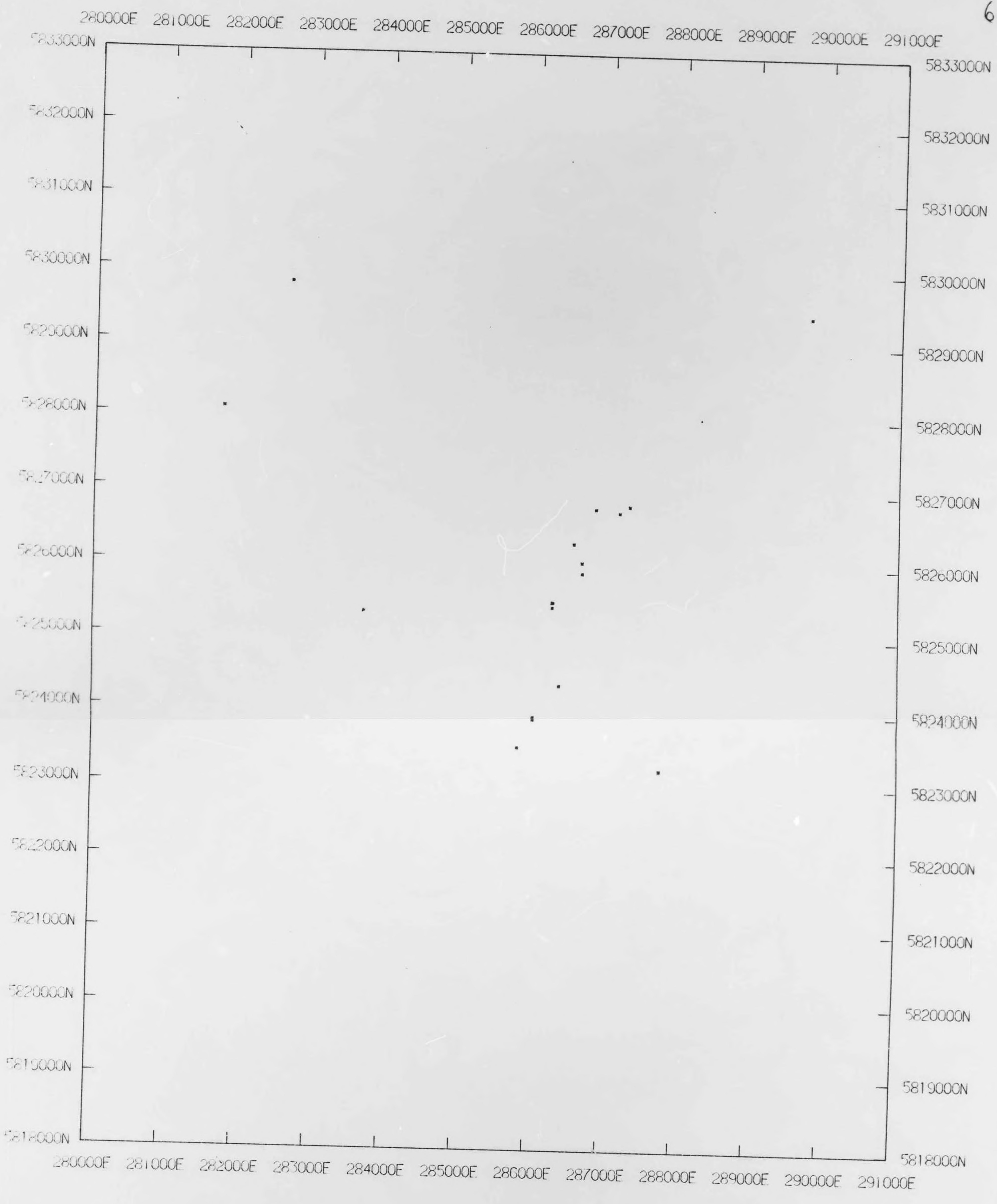
ENGINEERING GEOLOGICAL MAPPING OF MELTON, VICTORIA.

SOIL DEPTH

PROJ NO.

DATE: 15-JAN-86



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• SOIL DEPTH DATA POINT WHERE DEPTH IS BETWEEN SEVEN AND EIGHT METRES

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SOIL DEPTH

PROJ NO.

DATE: 15-JAN-86

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• SOIL DEPTH DATA POINT WHERE DEPTH IS BETWEEN SIX AND SEVEN METRES

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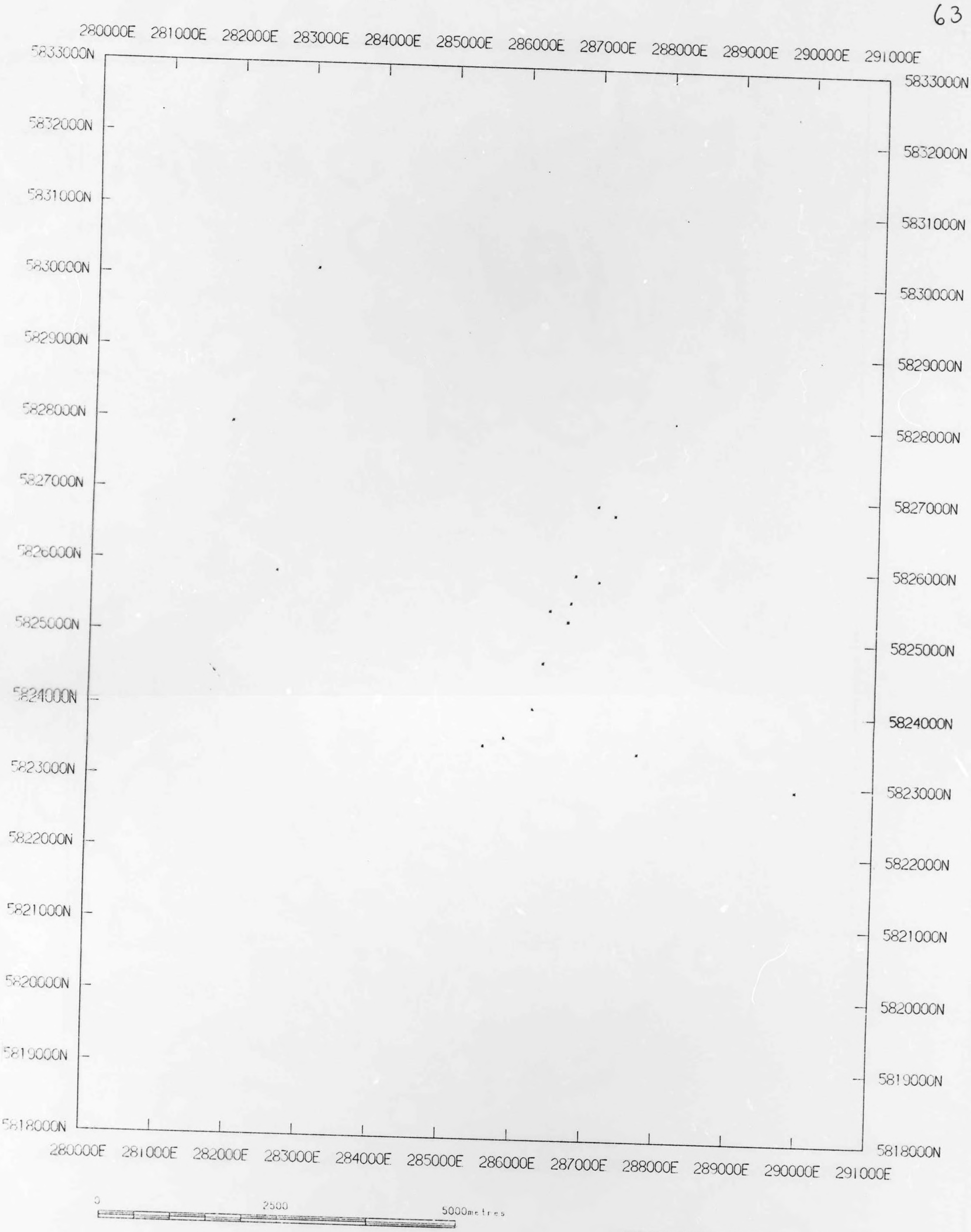

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• SOIL DEPTH DATA POINT WHERE DEPTH IS BETWEEN FIVE AND SIX METRES

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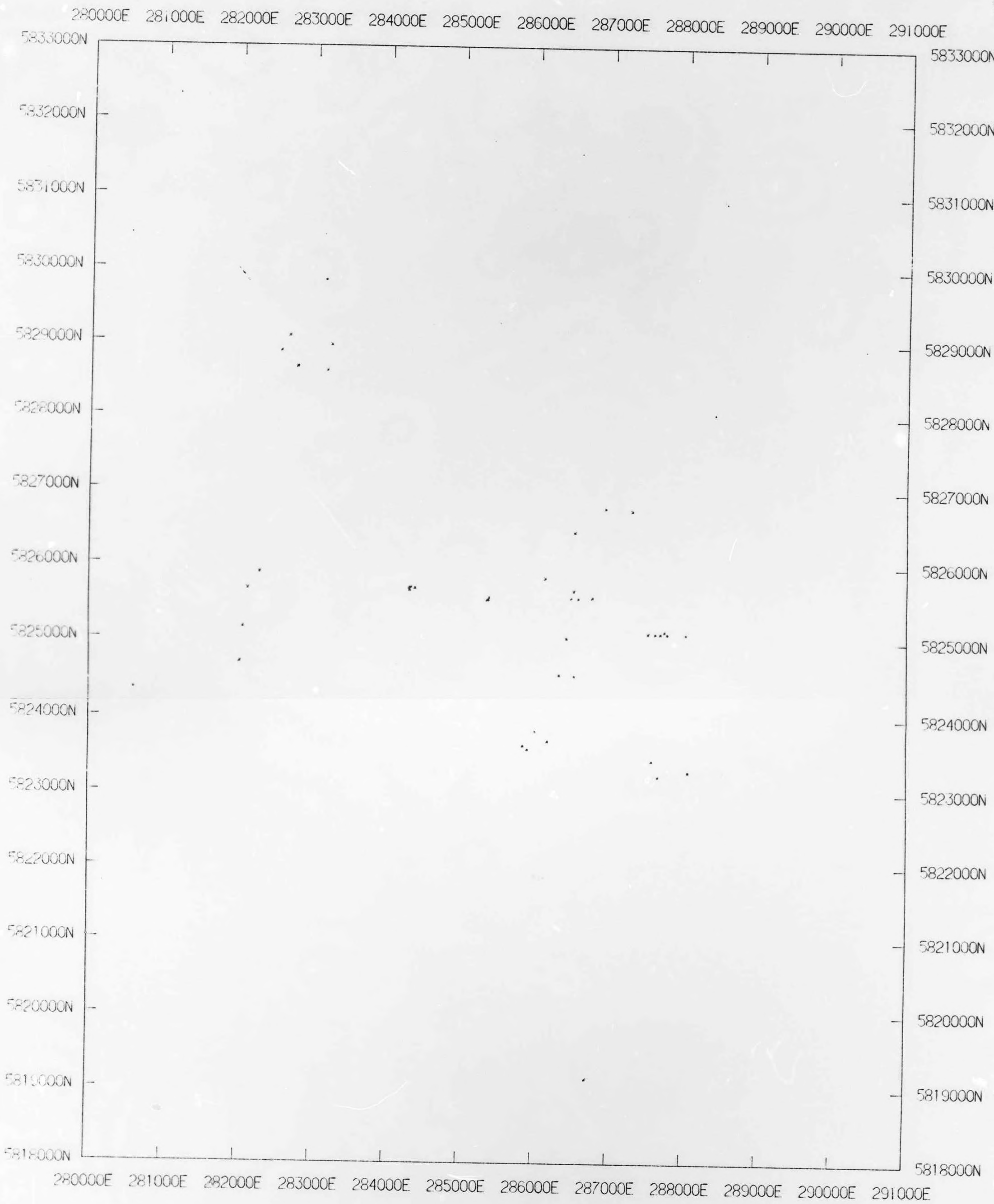
SOIL DEPTH

PROJ NO.

DATE: 15-JAN-86



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• SOIL DEPTH DATA POINT WHERE DEPTH IS BETWEEN FOUR AND FIVE METRES

N

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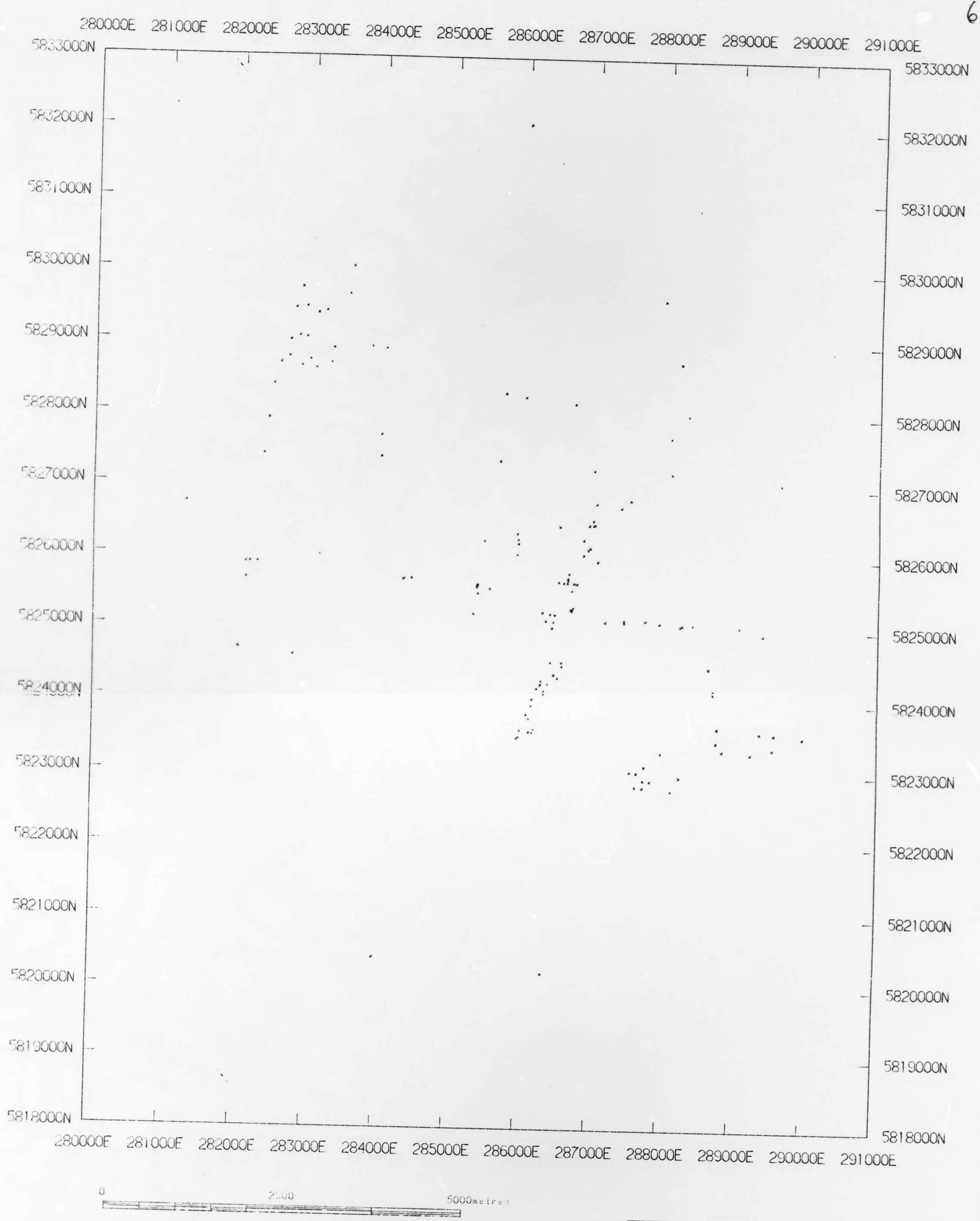

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LEGEND

• SOIL DEPTH DATA POINT WHERE DEPTH IS BETWEEN THREE AND FOUR METRES

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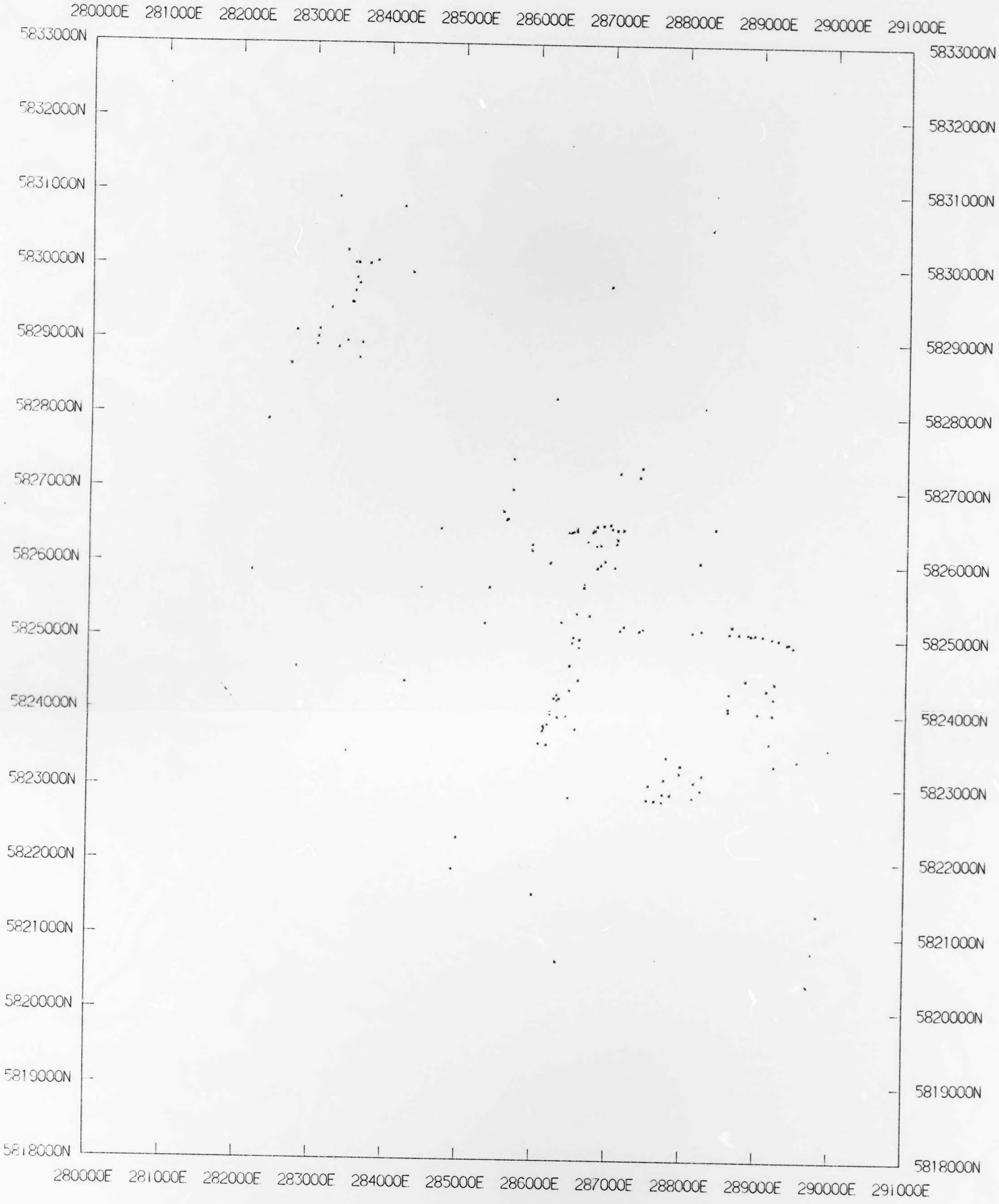
DATE: 15-JAN-86



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• SOIL DEPTH DATA POINT WHERE DEPTH IS BETWEEN TWO AND THREE METRES

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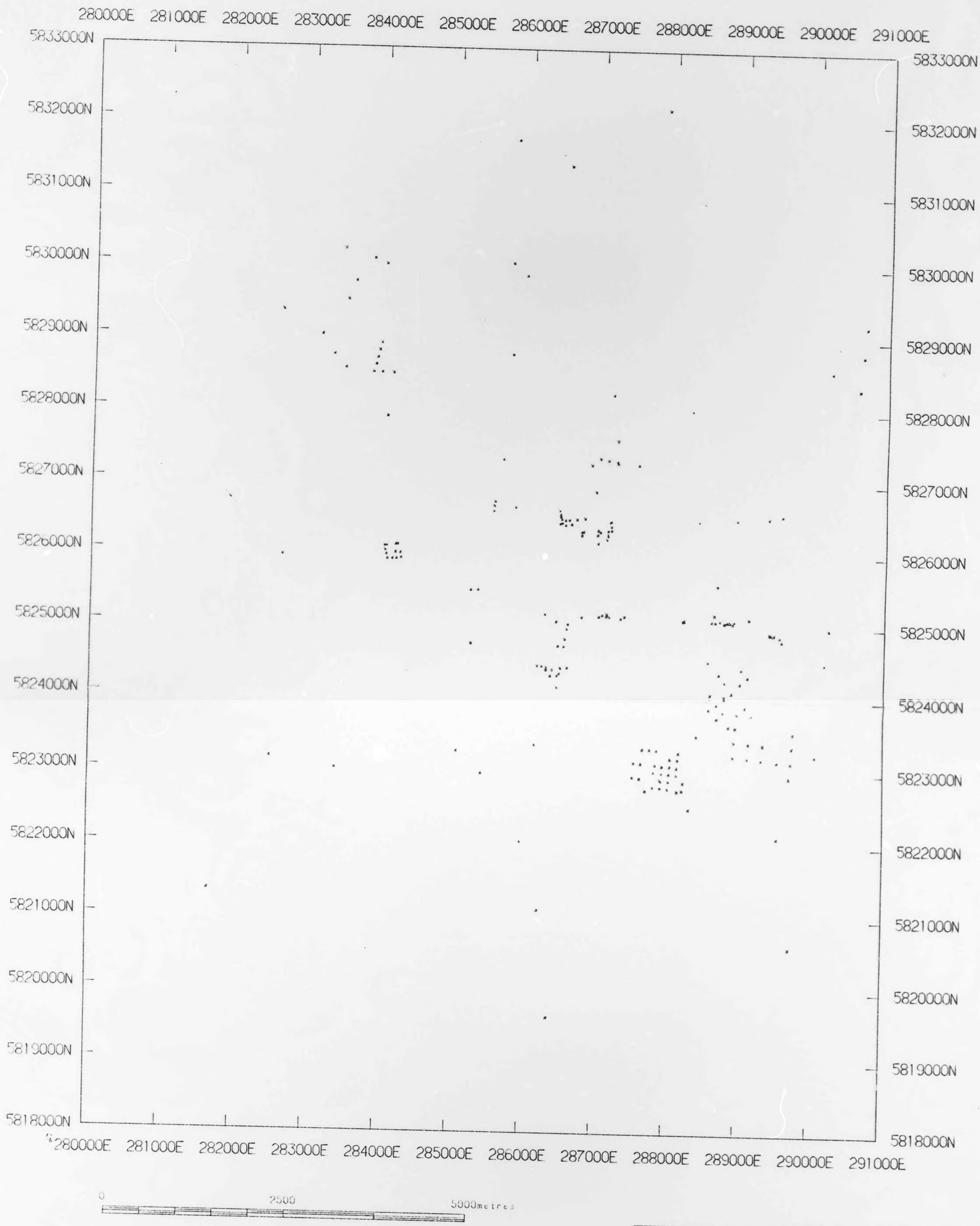

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• SOIL DEPTH DATA POINT WHERE DEPTH IS BETWEEN ONE AND TWO METRES

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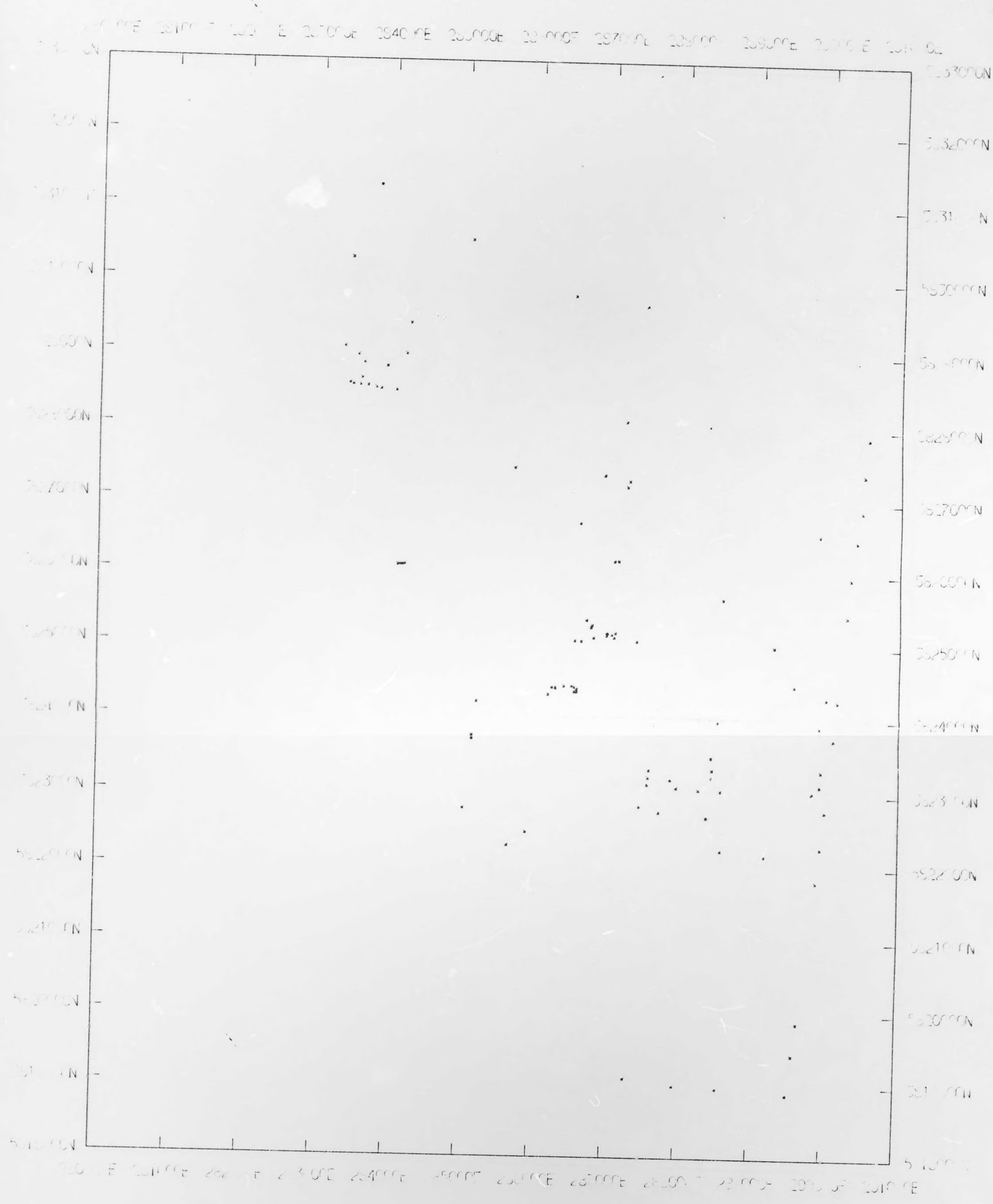
DATE: 15-JAN-86



0 2500 5000metres

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• SOIL DEPTH DATA POINT WHERE DEPTH IS ONE METRE OR LESS

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SOIL DEPTH	
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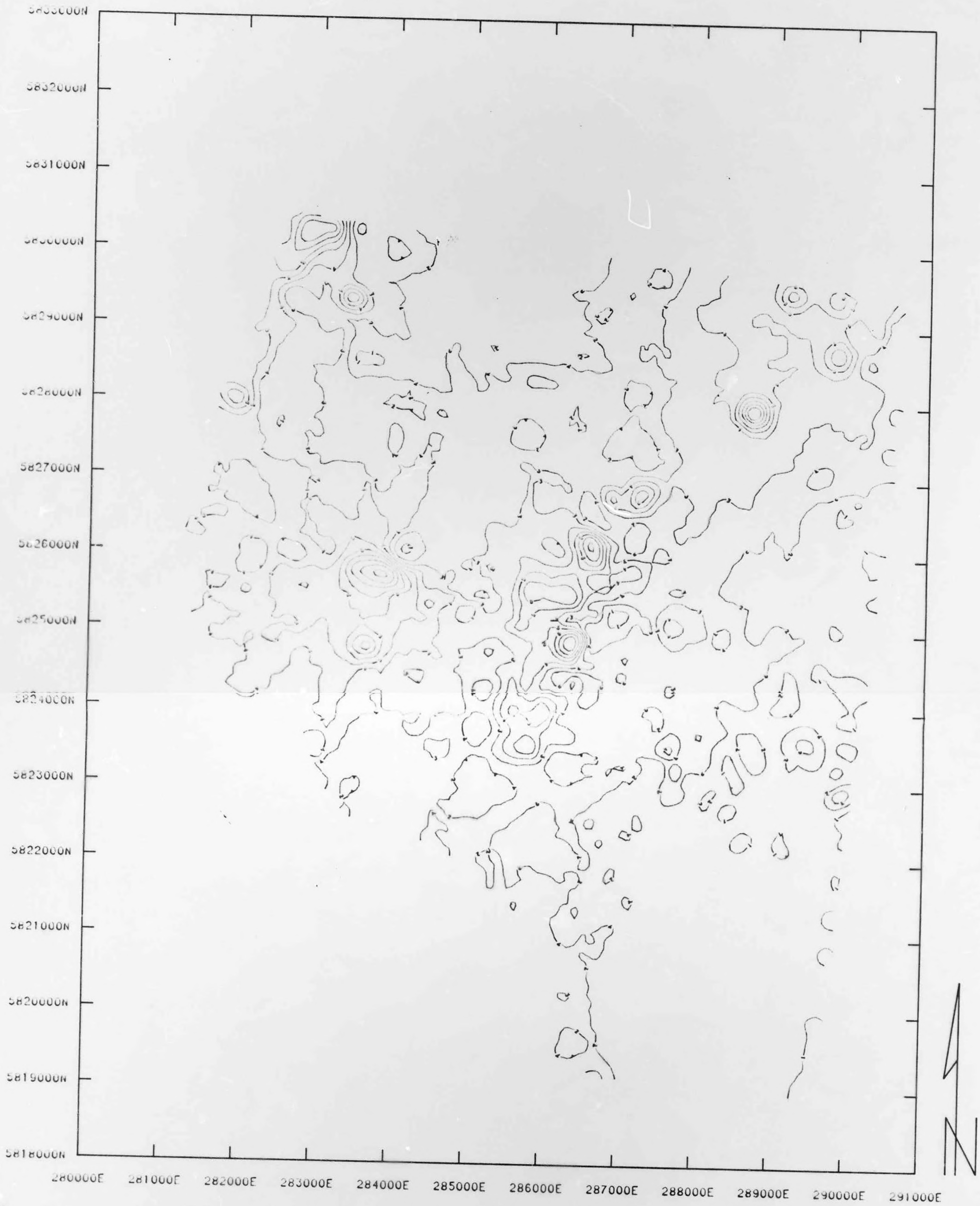

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SCALE 1:50000

Numeric surface generated
using block kriging -
100 X 100 m. blocks.

ENGINEERING GEOLOGICAL MAPPING OF MELTON. VICT.			
SOIL DEPTH ISOPLETHS			
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APPENDIX III

BUILDING REGULATIONS FOR FOOTINGS AND FOUNDATIONS
EXTRACT FROM VBR (1983)

GROUP VI—STRUCTURAL PROVISIONS

PART 32—FOUNDATIONS

32.1 * * * * *

FOUNDATIONS: ASSESSMENT OF ADEQUACY

32.2 The adequacy of *foundations* shall be based on *approved*—

(*a*) well established and relevant local knowledge and experience of *foundation* conditions in the vicinity of the proposed building; or

(*b*) tests on the *foundation* materials.

32.3 * * * * *

ALLOWABLE BEARING PRESSURES—GENERAL

32.4 The bearing pressure on the *foundation* of a building shall not exceed the values given in Regulation 32.5 unless—

(*a*) an investigation of the *foundations* has been conducted and the *building surveyor* is satisfied in the light of the report on that investigation, that higher bearing pressures are justified; or

(*b*) an investigation of the *site* has been conducted under AS 1726 and the bearing pressures are based on the information obtained from that investigation.

ALLOWABLE BEARING PRESSURES

Application of Regulation

32.5 (1) This Regulation shall only apply where the class and description of the soil or rock adopted for the purposes of this Regulation and the allowable bearing pressures adopted for the purposes of this Regulation are stated on the plans submitted for a *building approval.*

Reference to Tables

(2) The allowable bearing pressures for use pursuant to this Regulation shall be those prescribed in—

(*a*) Table 32.5A;

(*b*) Table 32.5B; or

(*c*) Table 32.5C—

as *required* to be construed in accordance with the Notes to those Tables.

Determination of soil description

(3) In determining an appropriate soil description for use pursuant to this Regulation, the designer shall take account of seasonal moisture conditions.

Pad or strip footings near boundaries

(4) Where any pad or strip footing is on or within 1 m of the boundary of the *allotment* other than a *street alignment*, the allowable bearing pressure shall be two-thirds of the value otherwise prescribed in this Regulation.

TABLE 32.5A

FOOTINGS ON COHESIVE SOIL

Description	*Maximum Allowable Bearing Pressures for Footings at Ground Surface (kPa)*	
	Strip Footings	*Pad Footings (Square or Circular)*
(1)	(2)	(3)
Very soft clay and silt	20	30
Soft clay and silt	40	60
Firm clay	95	110
Stiff clay	180	210
Very stiff clay	350	430
Hard clay	520	650

Notes:

A. Rectangular footings with width to length proportions in the ratio 1 : 5 or greater shall be deemed to be strip footings.

B. For rectangular footings with a width to length ratio between 1 : 1 and 1 : 5 the allowable bearing pressure may be interpolated between those prescribed for strip footings and pad footings.

C. Where a footing is located below ground surface the allowable bearing pressure may be increased by 5 kPa for each 300 mm in distance which the base of the footing is below the ground surface.

D. (1) For the purposes of this Table the following interpretations shall apply:

(*a*) 'Very soft clay and very soft silt' means soil which may be readily penetrated to a depth of 100 mm by the clenched fist.

(*b*) 'Soft clay and soft silt' means soil which may be easily penetrated to a depth of 50 mm by the thumb.

(*c*) 'Firm clay' means soil which may with moderate effort be penetrated to a depth of 50 mm by the thumb.

(*d*) 'Stiff clay' means soil which may readily be indented by the thumb, but penetrated by the thumb only with great effort.

(*e*) 'Very stiff clay' means soil which may be readily indented by the thumbnail.

(*f*) 'Hard clay' means soil which may be indented by the thumbnail but only with great difficulty.

(2) For the purposes of these interpretations clay shall include silty or sandy clays.

TABLE 32 5B

FOOTINGS ON NON-COHESIVE SOILS

Description	*Allowable Bearing Pressure in kPa for a Footing located at Ground Surface*	*Increase in Allowable Bearing Pressure in kPa for every 300 mm of Depth of Base of Footing Below Ground Surface*	*Maximum Allowable Bearing Pressure in kPa Under any Conditions*
(1)	(2)	(3)	(4)
Loose sand or gravel	50w	15	100
Medium sand or gravel	150w	40	250
Dense sand or gravel	350w	100	550
Very dense sand or gravel	600w	150	700

Notes:

A. For the purpose of this Table, w is the least plan dimension of the footing in metres.

B. If, in the opinion of the building surveyor, the water table is likely to rise to a level the distance of which below the base of the footing is not more than w, the allowable bearing pressure and maximum allowable bearing pressure shall be one half of that otherwise prescribed.

C. For the purposes of this Table, the following interpretations shall apply:

(*a*) 'Loose sand or gravel' means sand deposits readily removable by shovelling only and into which a sharp pointed wooden post 50 mm square can easily be driven with a hammer not exceeding 5 kg.

(*b*) 'Medium sand or gravel' means sand or gravel deposits removable by vigorous shovelling and into which a sharp pointed wooden post 50 mm square can be driven with a hammer not exceeding 5 kg with some difficulty.

(*c*) 'Dense sand or gravel' means sand or gravel deposits requiring picking for removal, and offering high resistance to penetration by excavating tools.

(*d*) 'Very dense sand or gravel' means gravel deposits requiring hard picking for removal, and offering hard resistance to disturbance by excavating tools.

TABLE 32 5C

FOOTINGS ON ROCK

Description	*Maximum Allowable Bearing Pressures for Rock Foundations in Various Conditions of Weathering in kPa*		
	Highly Weathered	*Moderately Weathered*	*Fresh to Slightly Weathered*
(1)	(2)	(3)	(4)
Soft limestone and similar porous rocks	100 to 400	300 to 1000	800 to 1500
Sandstone, mudstone and similar sedimentary rocks	200 to 600	500 to 1500	1200 to 2000
Slate, schist and similar metamorphic rocks	200 to 600	600 to 2000	1500 to 3000
Basalt, granite and similar igneous rocks	200 to 600	500 to 2000	1500 to 4500

Notes:

A. (*a*) The lower end of each range shall be used for rock foundations of the category to which the range applies and which are highly jointed or contain obvious defects.

(*b*) The upper end of each range shall be used for massive and consistent rock foundations of the category to which the range applies.

B. A bearing pressure greater than 600 kPa shall not be imposed by a footing resting on a basalt rock foundation unless a qualified engineer—

(*a*) establishes the condition of the basalt rock foundation to a depth of not less than 1½ times the width or diameter of the footings; and

(*b*) decides in accordance with good engineering practice and the condition of the basalt rock foundation that it is safe to rest the footing on that foundation.

C. For the purposes of this Table:

(*a*) 'Highly Weathered Rock' means rock of predominantly earthy colours (particularly yellows, reds and browns) with numerous clay seams and pieces of which can generally be broken by hand.

(*b*) 'Moderately Weathered Rock' means rock showing some earthy colour predominantly surrounding the joints with some clay seams and pieces of which can generally be broken by hand;

(*c*) 'Fresh to Slightly Weathered Rock' means rock predominantly of a mineral colour or lustre with only minor discolouration adjacent to joints and pieces of which can only be broken with difficulty using hand tools.

D. Where rock is in an extremely weathered form so that, although rock texture and appearance are mainly preserved, the rock substance has the strength properties of soil and may be readily disintegrated by gentle agitation in water, it shall be deemed to be soil for the purposes of this Regulation and classified in accordance with interpretations contained in Tables 32 5A and 32 5B.

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GROUP VI—STRUCTURAL PROVISIONS

PART 33—FOOTINGS NOT ON PILING OR CAISSONS

PROVISION OF FOOTINGS

33.1 Suitable footings shall be provided where necessary to reduce the intensity of the pressure of the building on the *foundations*.

DESIGN OF FOOTINGS

33.2 Footings, including slab-on-ground footings, shall be designed and *constructed* so that any relative movements of separate footings and of different parts of any one footing under loading, or of a footing and any other element of the substructure will not impair the stability of or cause significant structural damage to the superstructure.

DEEMED TO COMPLY

Scope

33.3 (1) This Regulation shall apply to concrete strip or pad footings where—

(*a*) the building does not contain more than 4 *storeys*;

(*b*) the area of any *storey* of the building is not greater than 600 m²; and

(*c*) the bearing pressure exerted by the footing does not exceed the value prescribed in Regulation 32.4.

Construction and proportions of footings designed by prescribed allowable bearing pressure

(2) A footing designed and *constructed* in accordance with this Regulation shall be deemed to comply with Regulation 33.2 if it—

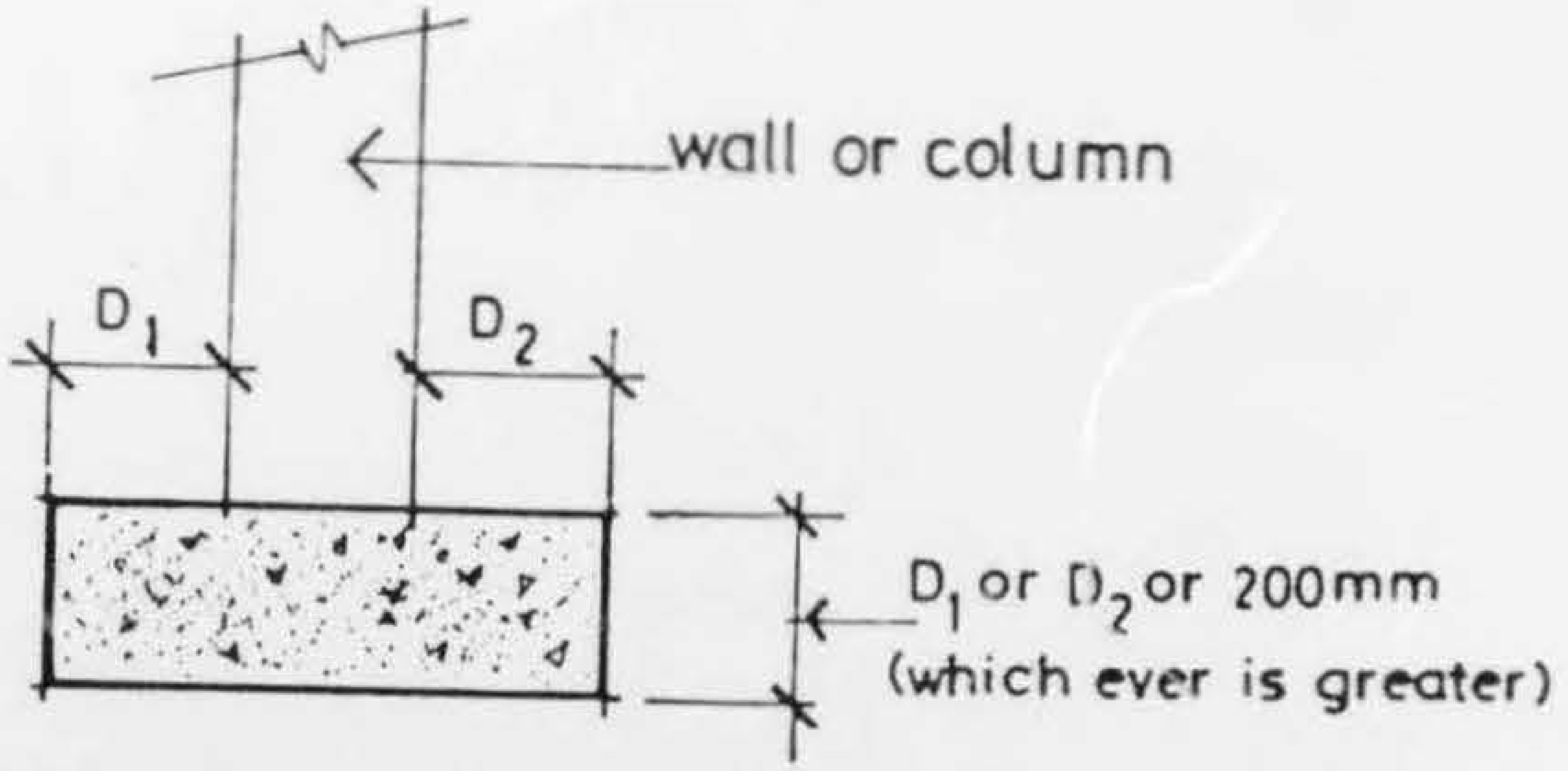
(*a*) is of reinforced concrete *constructed* having a compressive strength at 28 days of not less than 20 MPa, as determined in accordance with AS 1480;

(*b*) has a depth of not less than—

(i) the horizontal projection of the footing at right angles to the face of the wall or the column it supports (as illustrated in Figure 33.3); or

(ii) 200 mm—

whichever is the greater;



Horizontal Projection of footing

FIGURE 33.3

(*c*) in the case of a pad footing, contains not less than 0·15 per cent of the cross-sectional area of the footing, as designed, as reinforcement in each direction near the bottom face of the footing, with a minimum cover of 50 mm;

(*d*) in the case of a strip footing, contains not less than 0·15 per cent of the cross sectional area of the footing as designed as longitudinal reinforcement with a minimum cover of 50 mm, half of which shall be placed in the top third and half in the bottom third of the footing; and

(*e*) has reinforcement in strip footings and pad footings lapped for continuity—

(i) at splices—for a distance of not less than 500 mm;

(ii) at T intersections—for the full width of the layer;

(iii) at corners where fabric strips are used as reinforcement—for the full width of the fabric layer; and

(iv) at corners where bars are used as reinforcement—by a bent lap bar of 500 mm each leg, placed in each layer of reinforcement near the outer face of the corner.

DEEMED TO COMPLY

Application of Regulation

33.4 (1) Any footing of a Class I building or a building of another class which has a uniformly distributed live load not exceeding 3 kPa when calculated in accordance with AS 1170 which is *constructed* in accordance with the relevant provisions of this Regulation and

Regulations. 33.5 to 33.9 shall be deemed to comply with the requirements of Regulation 33.2 except where—

(*a*) by reason of—

(i) the nature of the *foundation*; or

(ii) the design of the building; or

(iii) any other relevant considerations—

the *building surveyor* is of the opinion that any such footing would not be adequate in the particular case; or

(*b*) the building—

(i) contains more than 2 *storeys*; or

(ii) has a wall, which, excluding any gable, exceeds 7·2 m in height; or

(iii) will contain a concrete floor other than a slab-on-ground.

Concrete strength

(2) Concrete used in footings shall have a compressive strength at 28 days of not less than 20 MPa, determined in accordance with AS 1480.

Foundation classification

(3) The classification of the *foundation* of any *site* on which footings are to be *constructed* shall be in accordance with the "Classification of Expansive Behaviour of Melbourne Soils for Domestic Construction" published by CSIRO and AEBIRA.

Areas not covered by publication

(4) In areas outside those shown in Figure 3 of the publication referred to in sub-regulation (3) the *site* on which footings are proposed to be *constructed* shall be similarly classified by adopting, where practicable, the principles stated in that publication and taking into account experience or knowledge of local or traditional building construction practices.

Drawings to include foundation classification

(5) The drawings referred to in Regulation 8.2 (2) shall include the *foundation* classification adopted pursuant to this Regulation and shall be confirmed to the satisfaction of the *building surveyor* on the *site* on which the footings are proposed to be *constructed*.

'FOOTINGS FOR STUMPS

General requirements

33.5 (1) Every footing for stumps shall comply with the following:

(*a*) The size of concrete footings for stumps shall be in accordance with AS 1684.

(*b*) The bearing areas of footings for stumps which support a wall sheathed with cement-sand facing tiles, shall be double those prescribed for a timber-framed wall in AS 1684.

(*c*) The footings shall be founded at a depth of—

(i) in the case of sites classified as stable in accordance with Regulation 33.3, not less than 450 mm; or

(ii) in the case of sites classified as intermediate in accordance with Regulation 33.3, not less than 700 mm; or

(iii) in the case of sites classified as unstable in accordance with Regulation 33.3, not less than 1 m.

Concession

(2) Notwithstanding paragraph (*c*) of sub-regulation (1), the *building surveyor* may permit the footings for stumps to be founded at a depth of less than 450 mm if he is satisfied by reason of experience or local knowledge that such a depth would be adequate for the structural stability of the building in the case of—

(i) the re-stumping of or *alterations* to an existing building;

(ii) rock *foundations*; or

(iii) a building in which walls of stud-framed and sheeted construction are supported on stumps.

Excavations

(3) Excavations for footings for stumps shall be—

(*a*) properly backfilled with *approved* material; and

(*b*) compacted in an *approved* manner.

STRIP FOOTINGS

Reinforcement

33.6 (1) Reinforcement in strip footings shall—

(*a*) be equally distributed in two layers, one near the top and one near the bottom of the footing;

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(*b*) have a concrete cover of not less than 50 mm at any part; and

(*c*) be laid continuously, each layer being lapped—

(i) at intersections—for its full width;

(ii) at splices—for not less than 500 mm;

(iii) at corners where fabric strips are used as reinforcement—for the full width of the fabric layer;

(iv) at corners where bars are used as reinforcement—by a bent lap bar of 500 mm each leg, placed in each layer of reinforcement near the outer face of the corner; and

(v) at steppings, as shown in Figure 33.6.

Design generally

(2) Strip footings constructed of concrete shall—

(*a*) have a width and depth in accordance with Table 33.6;

(*b*) if stepping is necessary, comply with one or more of the methods shown in Figure 33.6 and have level bottoms between steppings;

(*c*) be reinforced in accordance with Table 33.6 and Figure 33.6; and

(*d*) be founded on soil or rock having an allowable bearing pressure of not less than 100 kPa.

Reduced footing depth permissible

(3) Where a strip footing designed and *constructed* pursuant to this Regulation is to rest wholly or partly on a floater or rock outcrop, the depth of the strip footing in the vicinity of the floater or rock outcrop may, subject to sub-regulation (4) be reduced to not less than two-thirds of the depth otherwise prescribed by this Regulation.

Reinforcement in reduced footing depth

(4) Where the depth of a strip footing is reduced pursuant to sub-regulation (3), the reinforcement in the section of the strip footing of reduced depth—

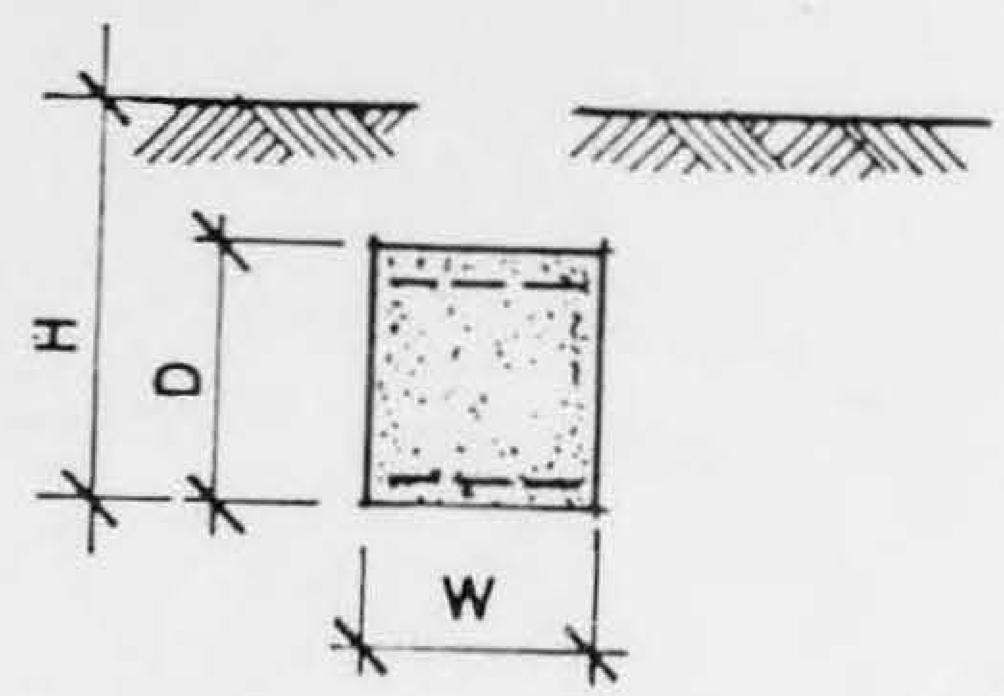
(*a*) shall be double the amount of that prescribed by Table 33.6; and

(*b*) shall extend at least 500 mm beyond the section of strip footing of reduced depth.

TABLE 33.6
MINIMUM DIMENSIONS AND REINFORCEMENT FOR STRIP FOOTINGS FOR DIFFERENT TYPES OF CONSTRUCTION

Foundation Classification	*Number of Storeys*	*Footings*		*Excavated Depth to Base of Footings Below Ground Surface*	*Alternative Reinforcement*	
		Width	*Depth*		*Number of C12, S16 or Y12 Bars or Main Wires of F11TM at Top and Bottom*	*Number of Main Wires of F8TM at Top and Bottom*
		W (mm)	*D (mm)*	*H (mm)*		
(1)	(2)	(3)	(4)	(5)	(6)	(7)
MASONRY VENEER, TIMBER FRAMED, METAL FRAMED OR SINGLE LEAF MASONRY WALLS						
Stable	One	300	375	450	2	3
	Two	375	375	450	3	4
Intermediate	One	300	525	600	3	6*
	Two	375	525	600	3	6*
Unstable	One	300	675	750	3	6*
	Two	375	675	750	3	6*
CAVITY OR DOUBLE LEAF MASONRY WALLS						
Stable	One	350	375	450	2	3
	Two	450	375	450	3	6*
Intermediate	One	350	525	600	3	6*
	Two	450	525	600	4	8*
Unstable	One	350	675	750	3	6*
	Two	450	675	750	4	8*

*Reinforcement is to be provided in two equal layers.



STRIP FOOTING
ILLUSTRATION TO TABLE 33.6

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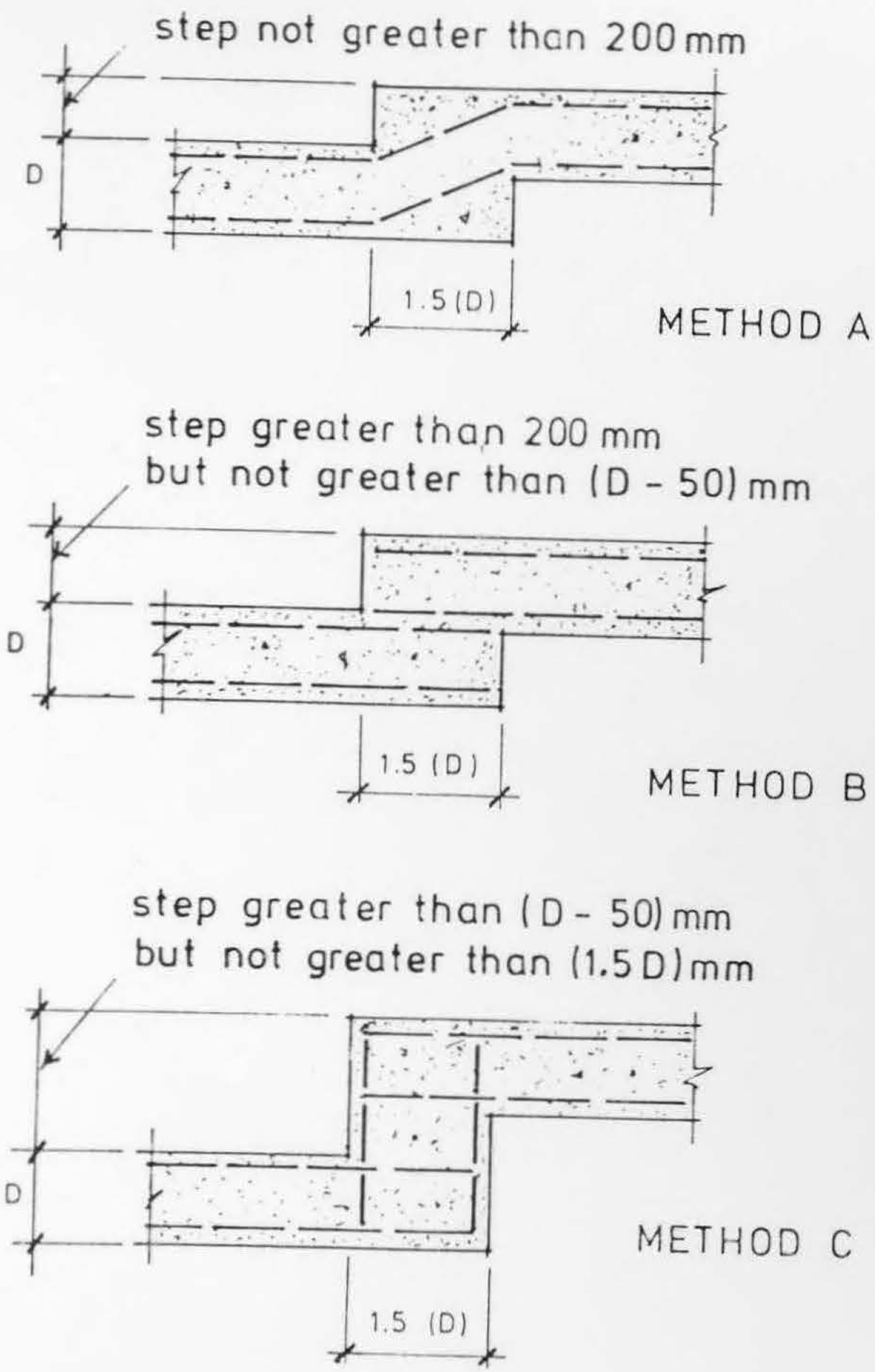


FIGURE 33 6 STEPPED FOOTINGS

SLAB-ON-GROUND

Reinforcement in the slab-on-ground beams

33.7 (1) Reinforcement in the beams in slab-on-ground shall—

(*a*) be placed near the bottom of each beam;

(*b*) have a concrete cover of not less than 50 mm at any part; and

(*c*) be lapped—

(i) at intersections—for its full width;

(ii) at splices—for not less than 500 mm;

(iii) at corners where fabric strips are used as reinforcement—for the full width of the fabric layer;

(iv) at corners where bars are used as reinforcement—by a bent lap bar of 500 mm each leg, placed in each layer of reinforcement near the outer face of the corner; and

(v) at steppings—as shown in Figure 33 6.

Reinforcement fabric in slab-on-ground

(2) Reinforcement fabric in slab-on-ground shall—

(*a*) be placed in the upper half of the slab, with a concrete cover of not less than 25 mm at any part;

(*b*) be lapped for a distance of not less than 225 mm; and

(*c*) be supported by bar chairs at spacings of not more than 1·2 m in either direction.

Requirements generally

(3) Every slab-on-ground shall comply with the following:

(*a*) Top-soil containing significant amounts of organic matter shall be removed from the area on which the slab is to rest.

(*b*) Edge beams of the slab shall be founded on soil or rock having an allowable bearing pressure of not less than 100 kPa.

(*c*) The slab shall be founded on soil or rock having an allowable bearing pressure of not less than 30 kPa.

(*d*) The slab shall be provided with a vapour barrier which shall—

(i) consist of a sheet of polyethylene not less than 0·2 mm in thickness;

(ii) be placed beneath the slab so that the bottom surface of the slab is entirely underlaid;

(iii) be continued around the edge beams to at least ground level or to the bottom of the edge recess, whichever is the lower;

(iv) be lapped at all joints for a distance of not less than 200 mm; and

(v) be taped around pipes which penetrate the slab.

(*e*) The dimensions and reinforcement of the edge beams shall be not less than those prescribed in Table 33.7A and as illustrated in Figure 33.7A.

(*f*) Edge recesses shall be provided for a *masonry* cavity wall or *masonry* veneer construction and shall—

(i) have a depth of not less than 50 mm and any part of the edge beam below any such recess shall have a depth of not less than 150 mm; and

(ii) be *constructed* in the manner illustrated in Figure 33.7A.

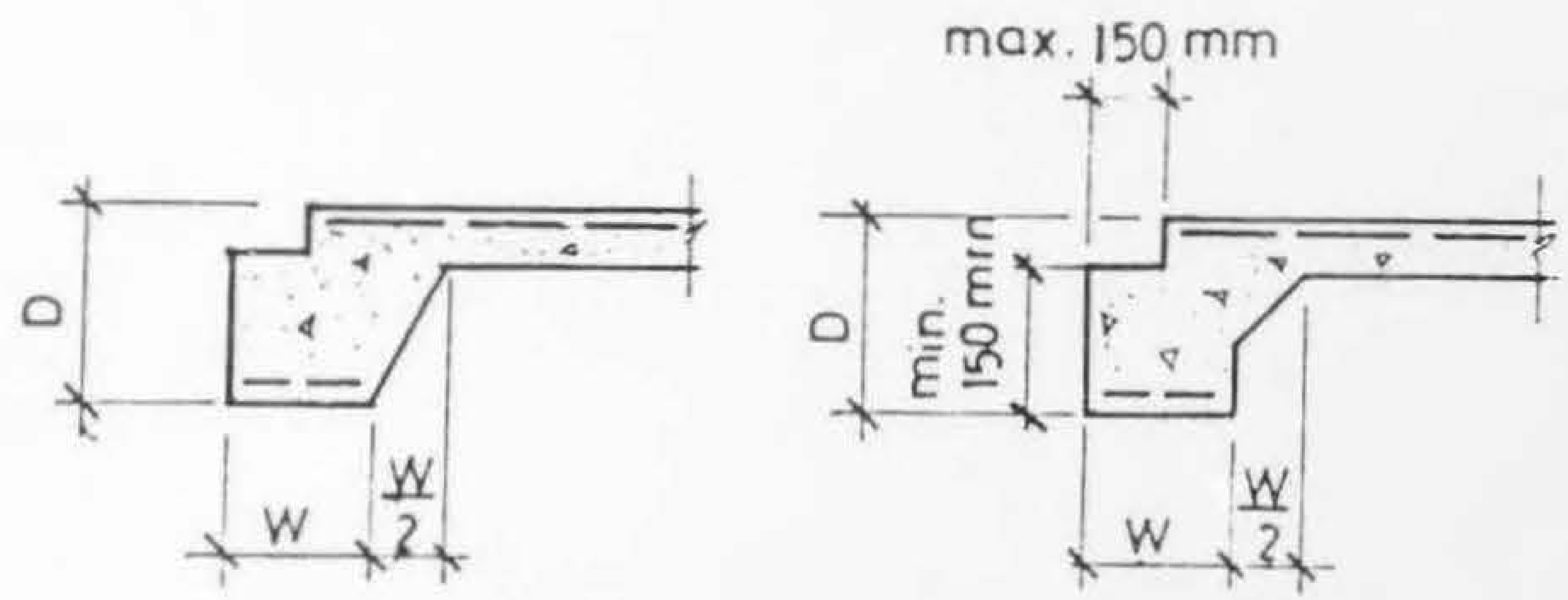


FIGURE 33.7A

TABLE 33.7A
MINIMUM DIMENSIONS AND REINFORCEMENT OF EDGE BEAMS

Foundation Classification and Building Height	*Size (W × D) in mm*	*Alternative Bottom Reinforcement*	
		Number of C12, S16 or Y12 Bars or Main Wires of F11TM Fabric	*Number of Main Wires of F8TM Fabric*
(1)	(2)	(3)	(4)
Stable			
—one storey	300 × 300	2	3
—two storeys	400 × 400	3	4
Intermediate			
—one storey	300 × 400	3	6*
—two storeys	400 × 400	4	8*

TABLE 33.7A—*continued*
MINIMUM DIMENSIONS AND REINFORCEMENT OF EDGE BEAMS

Foundation Classification and Building Height	*Size (W × D) in mm*	*Alternative Bottom Reinforcement*	
		Number of C12, S16 or Y12 Bars or Main Wires of F11TM Fabric	*Number of Main Wires of F8TM Fabric*
(1)	(2)	(3)	(4)
Unstable			
—one storey	300 × 600	3	6*
—two storeys	400 × 600	4	8*

* Reinforcement is to be provided in two equal layers.

(*g*) On completion of the building the top surface of the slab shall be at a height above the adjoining ground level of—

(i) 75 mm, in the case of a slab located adjacent to a drained and paved area;

(ii) 100 mm, in the case of a slab located on a sandy, well drained *site*; or

(iii) 150 mm, in any other case.

(*h*) Stiffening beams shall—

(i) be *constructed* in accordance with the dimensions prescribed in Table 33.7B and in the manner illustrated in Figure 33.7B;

(ii) be reinforced in accordance with the provisions of Table 33.7B; and

(iii) be founded on soil or rock having an allowable bearing pressure of not less than 30 kPa;

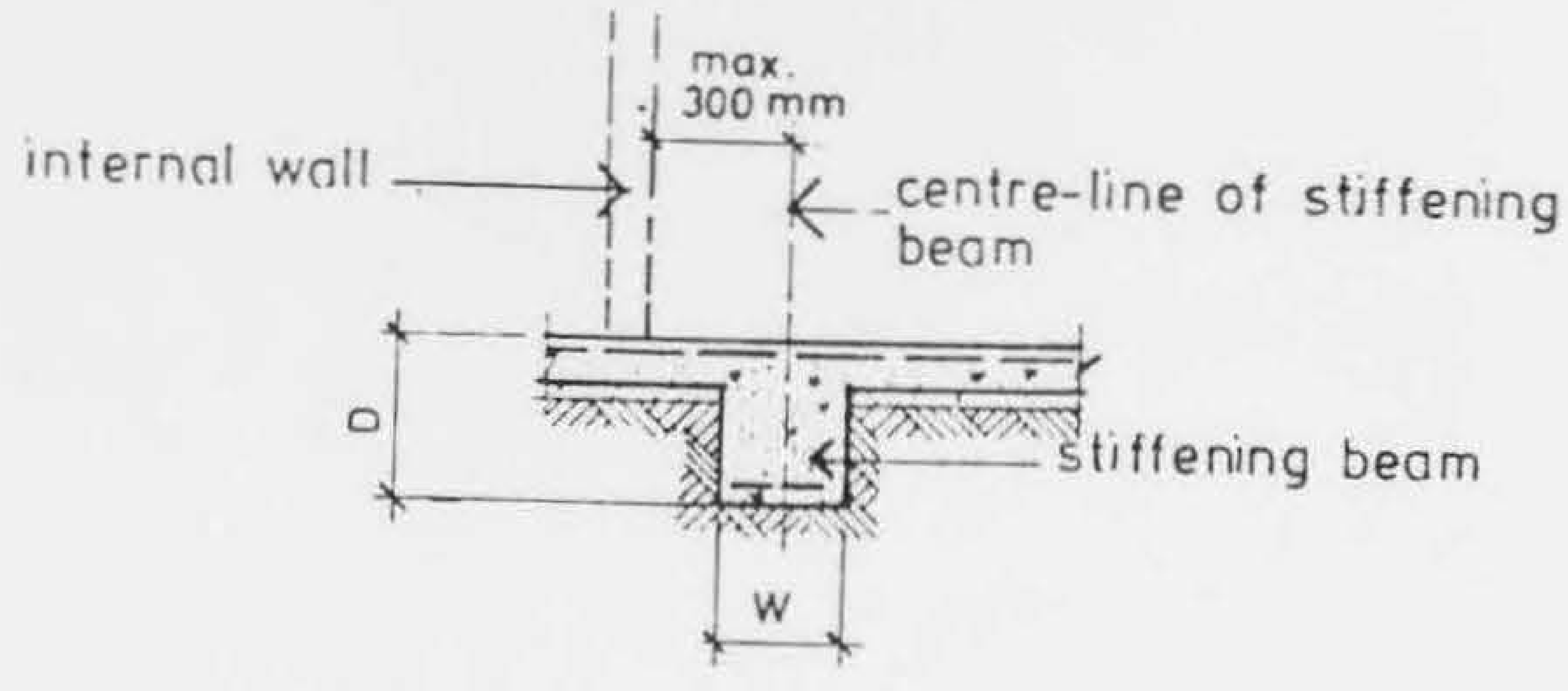


FIGURE 33.7B

TABLE 33.7B

MINIMUM SIZE, SPACING AND REINFORCEMENT OF STIFFENING BEAMS

Foundation Classification	*Beam Size (W × D)*	*Alternative Bottom Reinforcement*		*Maximum Spacing of Stiffening Beams from Edge Beams or other Stiffening Beams in any direction in metres*	
		Number of C12, S16 or Y12 Bars or Main Wires of F11TM Fabric	*Number of Main Wires of F8TM Fabric*	*Timber or Metal Framed Internal Walls*	*Masonry Internal Walls*
(1)	(2)	(3)	(4)	(5)	(6)
Intermediate	300 × 400	3	6*	4·5	3·5
Unstable	300 × 600	3	6*	4	3

* Reinforcement is to be provided in two equal layers.

(*i*) A support shall be provided under any *internal wall* in the manner prescribed by Part 1 of Table 33.7C if the wall is not located within 300 mm of the centre-line of a stiffening beam.

(*j*) A beam providing support for an *internal wall* in accordance with Part 1 of Table 33.7C shall be—

(i) *constructed* in accordance with Part 2 of Table 33.7B for stiffening beams and in the manner illustrated in Figure 33.7B;

(ii) if the wall is a *loadbearing* wall, be founded on soil or rock having an allowable bearing pressure of not less than 100 kPa; and

(iii) if the wall is a non-*loadbearing* wall, be founded on soil or rock having an allowable bearing pressure of not less than 30 kPa.

TABLE 33.7C

PART 1—SUPPORTS FOR VARIOUS TYPES OF INTERNAL WALL CONSTRUCTION

Number of Storeys	*Type of Wall Construction*	*Internal Wall*	
		Loadbearing	*Non-loadbearing*
(1)	(2)	(3)	(4)
one	timber or metal	no requirement	no requirement
two	timber or metal	beam required	additional slab reinforcement required
one	masonry	additional slab reinforcement required	no requirement
two	masonry	beam required	beam required

PART 2—MINIMUM SIZE AND REINFORCEMENT OF INTERNAL WALL SUPPORT BEAMS

Foundation Classification	*Beam Size (W × D)*	*Alternative Bottom Reinforcement*	
		Number of C12, S16 or Y12 Bars or Main Wires of F11TM Fabric	*Number of Main Wires of F8TM Fabric*
(1)	(2)	(3)	(4)
Stable	300 × 300	2	3
Intermediate	300 × 400	3	6*
Unstable	300 × 600	3	6*

*Reinforcement is to be provided in two equal layers.

(*k*) additional slab reinforcement providing support for an *internal wall* in accordance with Part 1 of Table 33.7C shall—

(i) be not less than 800 mm wide;

(ii) be positioned centrally under the wall in the lower part of the slab with a concrete cover of not less than 30 mm;

(iii) be placed in the manner illustrated in Figure 33.7C; and

(iv) comply with the provisions of paragraph (*l*);

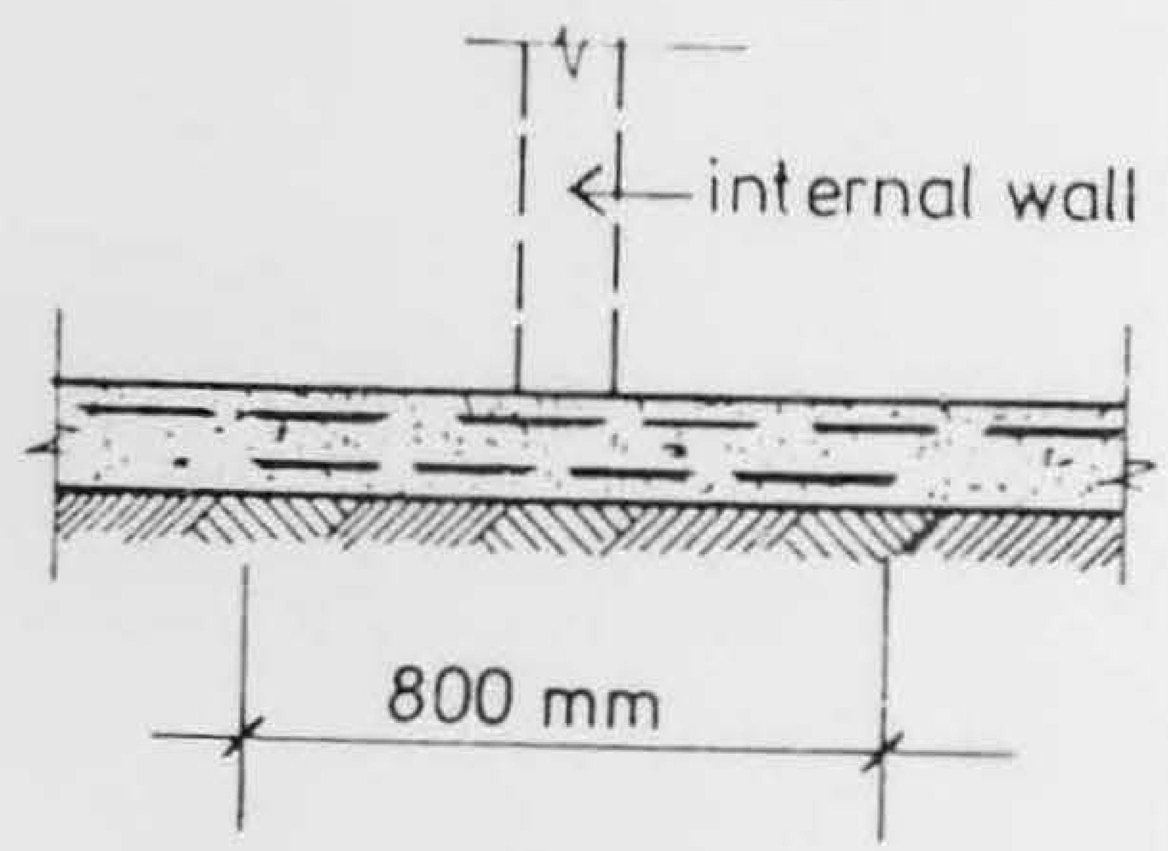


FIGURE 33.7C

(*l*) every slab shall—

(i) in the case of a slab resting on soil classified in accordance with Regulation 33.3 as stable, be reinforced with F72 mesh or bars of equivalent strength; or

(ii) in any other case, be reinforced with F82 mesh or bars of equivalent strength;

(*m*) A slab shall be not less than 100 mm thick.

(*n*) Pipes providing heat to a slab shall not be embedded in a slab less than 125 mm thick.

(*o*) Where pipes are to be embedded or recesses provided in the slab, the slab shall be thickened in an *approved* manner to ensure that there is no loss of strength.

Reduced footing depth permissible

(4) Where a beam of a slab-on-ground, designed and *constructed* pursuant to this Regulation, is to rest wholly or partly on a floater or rock outcrop, the depth of the beam in the vicinity of the floater or rock outcrop may, subject to sub-regulation (5), be reduced to not less than two-thirds of the depth otherwise prescribed by this Regulation.

Reinforcement in reduced footing depth

(5) Where the depth of a beam is reduced pursuant to sub-regulation (4), the reinforcement in the section of the beam of reduced depth—

(*a*) hall be double the amount of that prescribed by this Regulation; and

(*b*) shall extend at least 500 mm beyond the section of strip footing or beam of reduced depth.

FOOTING SLABS

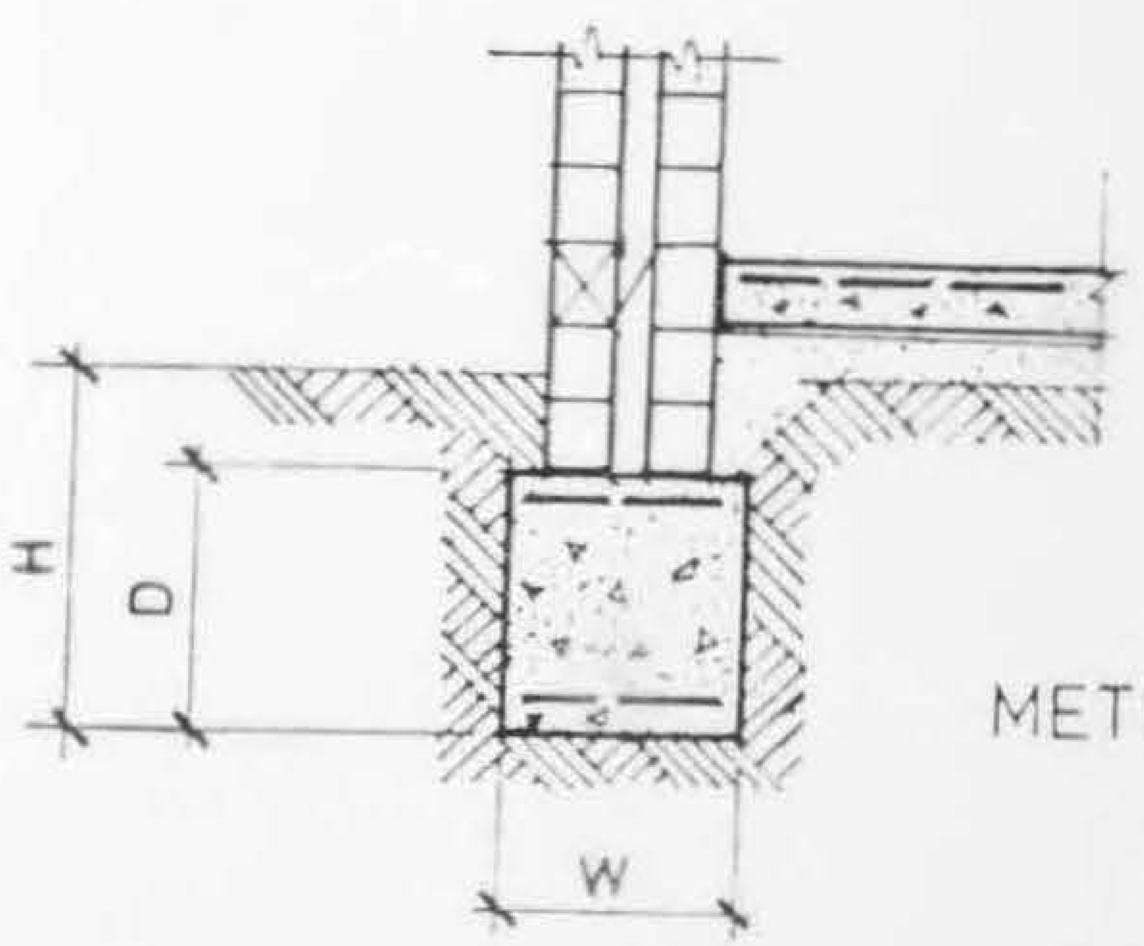
33.8 A footing slab system designed and *constructed* pursuant to this Regulation shall comply with the following:

(*a*) Except where by reason of experience or local knowledge the *building surveyor* permits otherwise, the footing slab shall be founded on a *site* classified in accordance with Regulation 33.3 as stable.

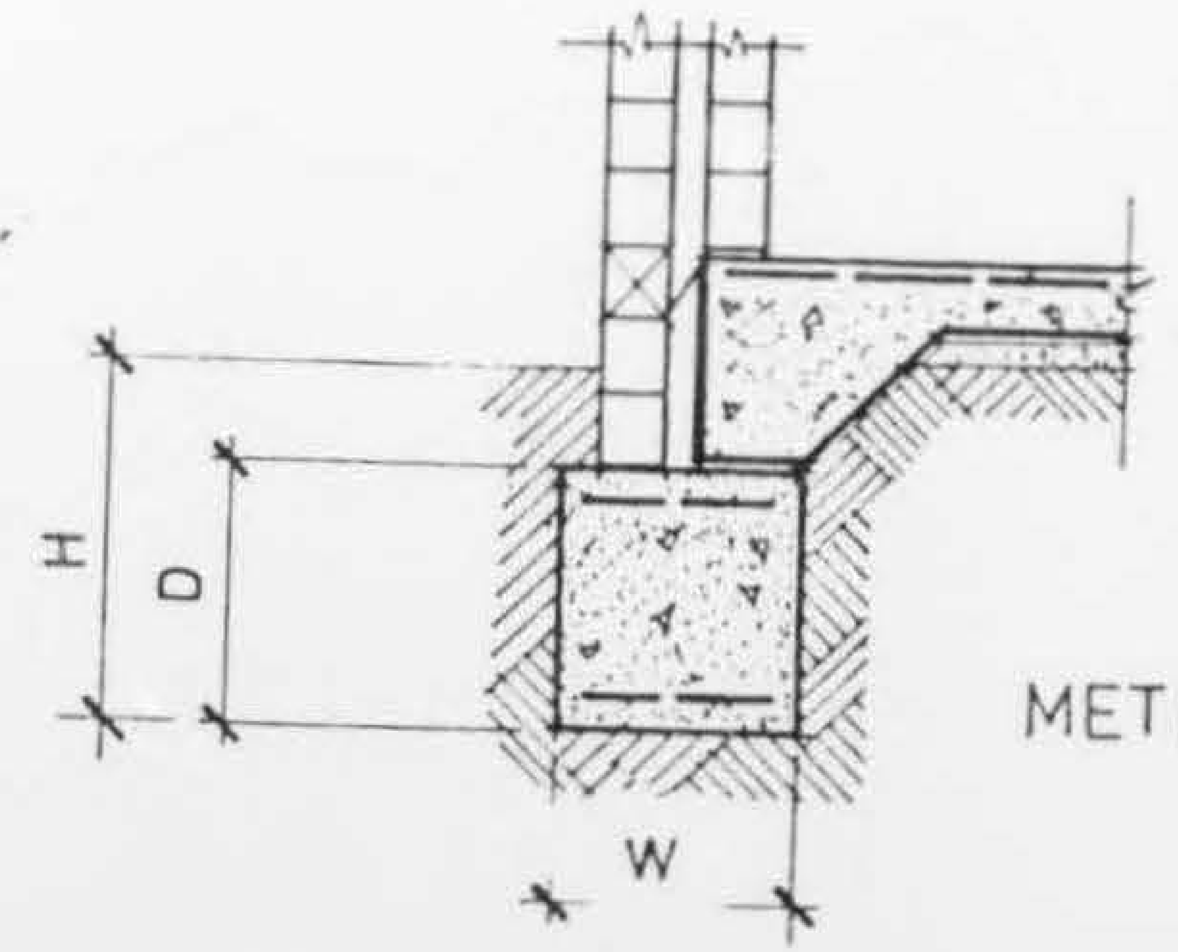
(*b*) The configuration of the system shall conform with—

(i) one of the methods illustrated in Figure 33.8; or

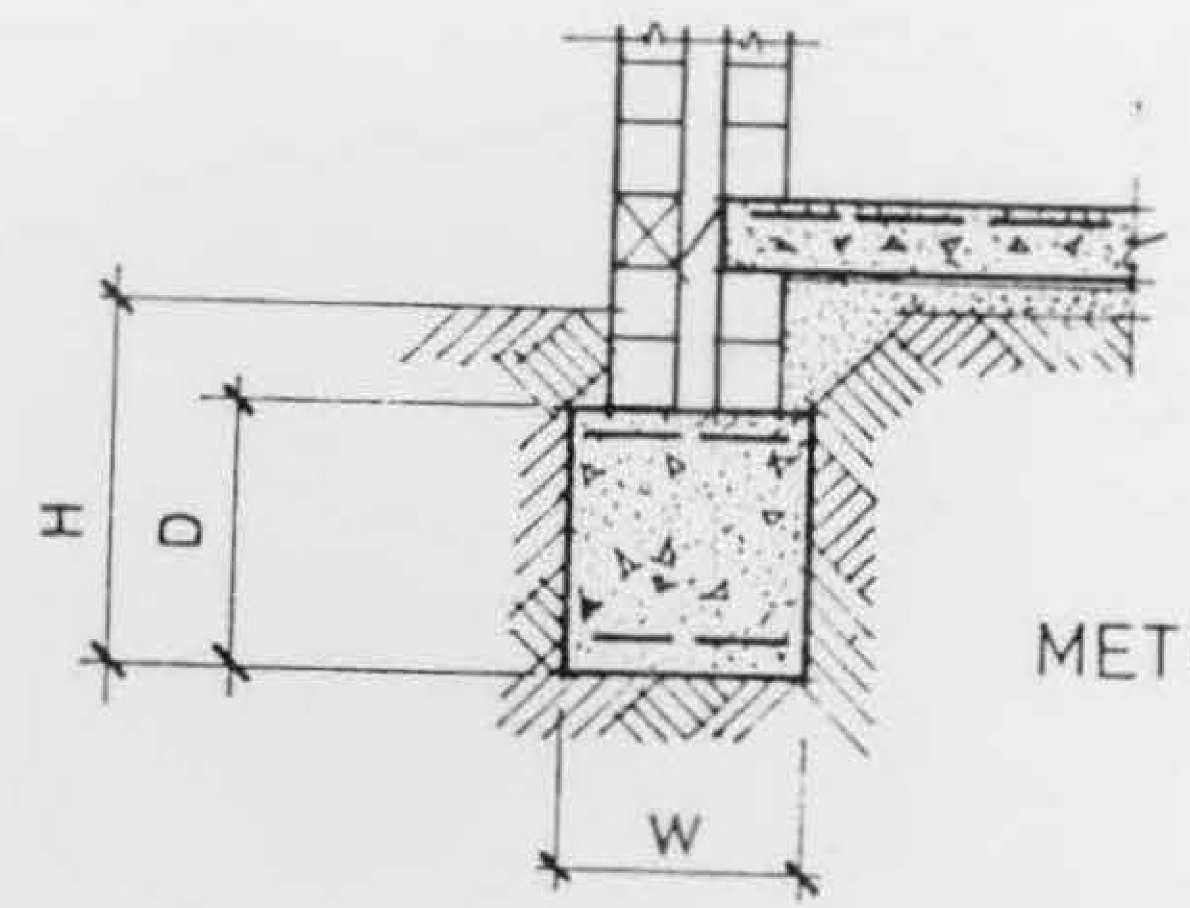
(ii) any other method not less effective than the methods so illustrated;



METHOD A



METHOD B



METHOD C

FIGURE 33.8

(*c*) The footing part of the system shall comply with the provisions of Regulation 33.6 as if it were a strip footing.

(*d*) The slab part of the system shall comply with the provisions of Regulation 33.7 (3) (other than paragraphs (*b*), (*d*) (iii), (*e*) and (*f*)) as if it were a slab-on-ground.

(*e*) Where, in the design and *construction* of a footing slab system, filling is restrained by an *external wall* and the filling is greater than 600 mm in depth, the *external wall* shall be designed by a *qualified engineer* and *constructed* in accordance with that design.

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FOOTINGS ADJOINING BOUNDARIES: PERMISSIBLE PROJECTIONS

33.9 Notwithstanding anything in Part 15, a footing may—

(*a*) support a *party wall*; and

(*b*) extend beyond the boundaries of a *street alignment*—

(i) to a distance of not more than 300 mm where the highest projecting part of the footing is at a depth of not less than 450 mm but is less than 3 m below the ground level; or

(ii) to a distance of not more than 1 m where the highest projecting part of the footing is at a depth of 3 m or more below the ground level.

APPENDIX IV

SITE CLASSIFICATION METHOD
EXTRACT FROM AS2870-1986

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SECTION 2. SITE CLASSIFICATION

2.1 DESIGNATION. Site classes shall be designated as follows:

Foundation	Character	Class
Sand and rock Silt and some clay	Stable	A S
Moderately reactive clay Highly reactive clay Extremely reactive clay	Reactive	M H E
Sand Material other than sand	Controlled fill	A A to P
Mine subsidence Uncontrolled fill Landslip Soft Collapsing soils	Problem	P

2.2 CLASSIFICATION PROCEDURE. All site classifications shall be based on one or more of the following:

(a) Assumption of soil type without any site investigation from a classification map or from well established local knowledge provided that soils are known to be consistent over large areas. The soil type and site conditions shall be checked by a site visit before construction.

(b) Site investigation to identify soil profile using one or more boreholes or test pits in the site or a number distributed over a subdivision.

(c) Site investigation using a penetrometer, for sand sites.

(d) Site investigation including soil sampling and appropriate tests.

(e) Clause 7.2 for South Australia.

Where the Building Authority has designated a presumed site classification or simplified system based on a map of site classifications, this may be used but shall not preclude the adoption of a less severe classification if supported by a site investigation and a classification in accordance with this Section.

2.3 STABLE AND REACTIVE SITES.

2.3.1 Sand or rock site. Sand sites (to depths in excess of the depths of influence as defined in Appendix D or to rock) or rock sites, as defined in the standard, shall be classified as Class A.

2.3.2 Silt sites. Silt sites as defined in the standard or mixtures of sand and silt (to depths of influence as defined in Appendix D or to rock) shall be classified as Class S.

2.3.3 Clay sites. In addition to the general requirements of Clause 2.2, the procedure for the classification of a clay site shall include one or more of the following methods:

(a) Visual assessment of the site and interpretation of knowledge of existing masonry house walls on light strip footings which have existed for not less than 15 years in a similar soil assessed in accordance with Table 2.1.

(b) Identification of the soil profile and a classification in accordance with Appendix C or from established data on the performance of the soil profile.

(c) Computation of the predicted surface movement, y_s, in accordance with Appendix D, with the following limits:

Surface Movement	*Class*
$y_s \leq 20$ mm	S
20 mm $< y_s \leq 40$ mm	M
40 mm $< y_s \leq 70$ mm	H
$y_s > 70$ mm	E

2.3.4 Reduction of reactive site classification. The effect of the treatments below may be taken into account to improve the site classification:

(a) Removal and replacement of reactive clay with a non-reactive material and protection of any remaining reactive clay from moisture changes; or

(b) Covering the site with a layer of compacted stable material preferably well in advance of construction.

2.3.5 Soft foundations. Soft foundations are classified as Class P where the allowable bearing pressure at foundation level is less than the following values as appropriate:

(a) Under strip or pad footings ....100 kPa.

(b) Under beams and slab panels for all slabs, except that 100 kPa is required under the edge footing of footing slabs without ties......... 50 kPa.

Allowable bearing pressures shall be assessed in accordance with Appendix B.

NOTE: Inadequate allowable bearing pressure is not common except for silt sites.

TABLE 2.1

SIMPLE CLASSIFICATION OF CLAY SITES

(Damage categories are given in Appendix A)

Characteristic performance of masonry (veneer or full) house on light strip footings	Classification of site
Rare Category 0 or 1 damage	S
Often Category 1 damage but rarely Category 2 damage. Category 3 damage is very rare even in extreme environmental conditions. (The site may show surface cracking in dry periods.)	M
Often Category 1 or 2 damage with occasional examples of Category 3 damage or more severe. (Ground surface cracking is common in dry periods.)	H
Often Category 3 or more severe and area is usually well known for damage to houses and structures. (Deep ground surface cracking occurs in dry spells.)	E

2.4 CONTROLLED FILL SITES.

2.4.1 Controlled sand fill on sand sites. Controlled sand fill over sand site may be classified as a Class A site.

2.4.2 Shallow controlled fill. The effect of controlled fill up to 800 mm deep for sands and gravels and up to 450 mm deep for clay may be disregarded in the site classification.

2.4.3 Other controlled fill sites. Other controlled fill sites may be classified as Class S sites provided that the settlement and reactivity of both the fill and the underlying natural soil complies with Clause 2.3.3(c).

2.5 PROBLEM SITES. Where the site includes mine subsidence, uncontrolled fill, landslip conditions or soft soil (see Clause 2.3.5), the site shall be classified as a problem site (Class P) and a footing system shall be designed in accordance with Section 5.

APPENDIX C

SITE CLASSIFICATION BY SOIL PROFILE IDENTIFICATION

In some areas, where sufficient data has been established, an assessment of the reactivity of a clay profile may be associated with the descriptions in Tables C1, C2 and C3 provided that the clay on the site is identified as belonging to the profile described in the table. In neighbouring regions, the table may be used if soil types and climates are similar.

Where a range of classifications is given in the tables generally, the typical values should be used. A higher or lower classification may be required if the moisture conditions or soil profile on the site differ markedly from those normally expected.

NOTES:

1. Depth of clay layer refers to the thickness of the clay in the profile. Shallow may be taken to mean less than 0.6 m depth of reactive clay.
2. The tables can only be used in conjunction with a site investigation where variable soil conditions are expected such as Adelaide. Where the soils are consistent, such as Melbourne, geological or pedological maps may be used but the soil type should be checked by a site visit before construction.
3. Only a limited number of profiles are included. If the soil profile is not listed in the table then some alternative classification procedure should be used.
4. Where a range is given the classification may be based on the depth of clay, the depth of the water table, the drainage of the site and a visual assessment of the reactivity of the soil.
5. The soil type notation in Table C2 is taken from Bulletin 46 'Geological Survey of South Australia'. A classification from the table shall not be based solely on the maps given in that report on soil types. The Category E1 has been introduced primarily for those profiles which represent a transition from highly to extremely reactive.

TABLE C1
CLASSIFICATION BASED ON LOCATION AND TYPICAL PROFILE—VICTORIA

Examples	Classification
Melbourne and District	
Basaltic clays—	
≤ 0.6 m depth of clay layer	M
> 0.6 m depth of clay layer	H
Non-basaltic clays—	
(Including silurian and devonian residual clays and quaternary alluvial clays)	
≤ 0.6 m depth of clay layer	S
> 0.6 m depth of clay layer	M
Tertiary sediments	
≥ 1 m sand over clay	A
< 1 m sand over clay, assess on the basis of depth of clay layer—	
≤ 0.6 m	S
> 0.6 m	M
Westernport	
Alluvial clays	S to H
Horsham and district	
Grey brown cracking clays	H to E
Geelong	
Basaltic clays—	
≤ 0.6 m depth	M
> 0.6 m depth	H
Waurn Ponds formation	M to E
Tertiary sediments	S to M
Phillip Island	
Basaltic clays	H
Alluvial clays	M
Shepparton and District	
Quaternary alluvial clay	S to H

NOTE: The reactivity of the tertiary and silurian clays is variable, and some areas of high reactivity have been identified. Whilst the above classifications have generally been shown as satisfactory, particularly when combined with the requirements of Appendix A, if testing is not carried out, local experience should be considered when classifying a site.

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1986/5

Geological Survey of Victoria

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P.G. DAHLHAUS

DEPARTMENT OF INDUSTRY, TECHNOLOGY AND RESOURCES

ENGINEERING GEOLOGY OF MELTON -
ENGINEERING GEOLOGY

P G DAHLHAUS

UNPUBLISHED REPORT 1986/5

ABSTRACT

An engineering geological mapping program has been conducted to provide essential geological information for use by planners and engineers working in the Melton Development Area, Victoria.

A review of past and current examples of thematic mapping for land use purposes was initially conducted. A data base of over 800 sampled locations was collated from previous work, and supplemented by additional drilling and testing in areas where little was known of the geological materials. This information was compiled using available computer facilities and combined with traditional field mapping methods. A map folio presenting individual aspects of the engineering geology was produced.

Large areas of expansive soil have been identified and mapped, and an area affected by soil subsidence was examined in detail. Statistical methods (block kriging) have been used to determine the thickness of soil in the map area. Assesments of the suitability for urban development have been made.

Computer draughting was used to produce the maps, providing the ability for rapid future revision.

This report, which is one of seven unpublished reports on the map area, describes the engineering geology.

KEYWORDS: Engineering Geological Mapping, Medium Scale, Soils, Maps, Engineering Geology, Engineering Soils, Kriging, Urban Planning, Hydrogeological Maps

CONTENTS

TABLES

APPENDICES

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State Electricity Commission (W. Regan)
Dandenong Valley Authority (J. Crouch)

LIST OF ABBREVIATIONS used in reporting the Melton Engineering Geology Mapping Project.

Abbreviation	Definition
AASHO	American Association of State Highway Officials
AEBIRA	Australian Engineering and Building Industry Research Association
AHD	Australian Height Datum
AMG	Australian Metric Grid
AS1276	Australian Standard AS1276 - SAA Site Investigation Code
AS1289	Australian Standard AS1289 - Methods of Testing Soils for Engineering Purposes
AS2870	Australian Standard AS2870 - Residential Slabs and Footings
CAD	Computer Aided Drafting/Design
CBD	Central Business District
CBR	California Bearing Ratio
CRB	Country Roads Board
CSIRO	Commenwealth Scientific and Industrial Research Organisation
DITR	Department of Industry, Technology and Resources
DOE	Department of Environment
DVA	Dandenong Valley Authority
ECS	Engineering Computer Services Pty. Ltd.
EDP	Electronic Data Processing
EPA	Environment Protection Authority
F&L	Farley and Lewers Pty Ltd
FAO	Food and Agriculture Organisation
FS	Free Swell
GEOSIS	Geoscience Spatial Information System
GLQ	Genesis-Lithology-Qualifier
GSV	Geological Survey of Victoria
IAEG	International Association of Engineering Geology
IGS	Institute of Geological Sciences
LL	Liquid Limit
LPS	Land Protection Service
LS	Linear Shrinkage
MMBW	Melbourne Metropolitan Board of Works
MPE	Ministry for Planning and Environment
MSA	Melton Sewage Authority
MSICC	Melton - Sunbury Interim Co-ordinating Committee
MURL	Melbourne Underground Rail Loop
OGS	Ontario Geological Survey
PL	Plastic Limit
PI	Plasticity Index
RCA	Road Construction Authority
SAA	Standards Association of Australia
SCA	Soil Conservation Authority
SCS-USDA	Soil Conservation Service - United States Department of Agriculture
TDS	Total Dissolved Solids
UBR	Uniform Building Regulations
ULA	Urban Land Authority
USGS	United States Geological Survey
VBR	Victorian Building Regulations
WHO	World Health Organisation
XRD	X-Ray Diffraction

INTRODUCTION

The City of Melton is located on the Western Highway 39 km WNW of Melbourne and was chosen by the Victorian Government for satellite township development in 1973.

The Melton Engineering Geological Mapping Project commenced in March 1983, as part of an ongoing mapping scheme conducted by the Geological Survey of Victoria (GSV), now a branch of the Department of Industry, Technology and Resources (DITR). The project aims at the production of a map (or maps) depicting relevant geological features and properties in a useful manner for engineers and planners working in the Melton Development Area.

An engineering geological map is a thematic map which provides a generalized representation of all those components of a geological environment of significance in land-use planning, and in design, construction and maintenance as applied to civil engineering.

A 'state-of-the-art' review of mapping methods for land-use planning was conducted to examine the past and present progress in a broad context. In particular, medium-scale engineering and environmental mapping methods, and their map presentation formats, were examined.

A review of readily accessible data highlighted shortcomings in both the quality and quantity of data outside of the established City of Melton. Consequently, a drilling, sampling and testing program was conducted. Research of previous work and additional geological mapping supplemented the data analysis. The presentation of the study has been largely cartographic, with each component of the geology being a separate theme on a basic map.

Seven reports have been produced in the GSV Unpublished Report series:

Unpublished Report 1986/1
Engineering Geological Mapping - A Review

Unpublished Report 1986/2
Engineering Geology of Melton - The Melton Development Area

Unpublished Report 1986/3
Engineering Geology of Melton - Drilling, testing and mapping program

Unpublished Report 1986/4
Engineering Geology of Melton - Geology and geomorphology

Unpublished Report 1986/5
Engineering Geology of Melton - Engineering geology

Unpublished Report 1986/6
Engineering Geology of Melton - Map presentation of data

Unpublished Report 1986/7
Engineering Geology of Melton - Summary

ENGINEERING GEOLOGY

1 Isopleth Mappin[illegible] f Soil Thickness

Thickness is a soil p[illegible]rty of importance to urban development at Melton. The depth [illegible]oil cover over the basalt has influence on the cost of water [illegible] sewerage supply, the foundation stability of light s[illegible]tures, and road making costs.

Isopleth maps may be used in engineering geological mapping to depict classes of soil, based on similarities of soil properties. It is implied that areas mapped as a particular class will have values of soil properties similar to those recorded for that class, and different from those of at least some of the other classes. Isopleth mapping is often referred to as 'contouring', by analogy with the mapping of topographic height. However, caution is required, as topographic contours are usually drawn to join points of equal measured height, whereas isopleths join points of inferred equal value. In practice topographic contours can be followed continuously, either on the ground or on a pair of aerial photographs, with the result that they can be drawn as accurately as the surveying equipment allows. Soil isopleths on the other hand must be derived from a set of more or less widely spaced points and are therefore subject to sampling variation.

Isopleths maps of soil thickness may be computed from borehole and test-pit observations by utilising computer packages.

1.1 Numeric surfaces

When mapping soil properties, the distribution of a single property may be displayed by assigning to each class on the map the typical value of that property within its class. The value at any one place is not actually recorded - unless there is a sample point (i.e. a borehole or test-pit) - it is predicted. It is realised that the actual value there will differ from the predicted value. The statistical rationale can be expressed as:

$$z_{ij} = \mu + \alpha_j + \epsilon_{ij} \qquad (1)$$

where z_{ij} the value of a property at any place i in class j is the sum of three terms:

μ the general mean of the property for the whole area;

α_j is the difference between the general mean and the mean of class j ; and

ϵ_{ij} is a random component distributed normally with zero mean and variance σ_w^2 .

The parameters μ ,α_j ,σ_w^2 and can all be estimated from data as say $\bar{z}$, a_j and s_w^2 respectively by the least squares analysis and analysis of variance. The predicted value for an unrecorded point in class j is $\bar{z} + a_j$, and confidence limits are determined from s_w^2 , the sample within-class variance. The smaller is σ_w^2 more precise will any prediction be, and the more valuable the map.

Where measured data are sparse, as they often are, this approach to prediction and mapping is the only feasible one. It obviously depends on there being an association between the property of interest and the classification, even though the classes are recognized independently in terms of the model, $|\alpha_j|$ must on average be substantially greater than zero, otherwise the classification does not help to predict the property. However the procedure takes no account of the spatial arrangement of the data points and their relations to predicted points, nor of any gradation of values across boundaries. These can only be of consequence when data are dense, specifically when they are spatially dependant, and in that event a means of prediction and mapping that uses the spatial information is obviously to be preferred.

In such circumstances interpolation provides an alternative to classification for predicting values of a property at unvisited points. Mapping can be achieved by envisaging such values as forming a continued statistical surface over the map plane, which can be represented by isopleths.

.1.2 Interpolation techniques

Several interpolation techniques are available, especially in computer programs for automatic contouring. Webster (1977) has breifly reviewed some of these viz: linear interpolation across triangulation, inverse square distance weighted averaging, least square polynomials, Theisson polygons, and kriging.

Apart from linear interpolation across triangles, most of these techniques consist of placing a fine, rectangular (or square) mesh over the entire area and computing a new value of the property at each mesh point. This process is suited to various forms of processing such as smoothing, filtering and fourier transformations. The size of the mesh interval will govern the frequency content of the resultant model and thus the isopleth map detail. Both the type of data and the desired type of map influence the choice of mesh interval and the choice of interpolation technique.

In using weighted interpolation techniques (such as kriging), three steps are involved.

1 Choosing the number of data points in the vicinity of the mesh point being considered. The selection algorithm may locate the closest 'n' points in 'x' number of sectors surrounding the mesh point normally within a certain distance.

2 Determine whether the chosen points are adequate in both number and in distribution.

3 Interpolate the mesh value by taking a weighted average of the data values. The weight is a function of the assigned value, the distance between the data values and an optional smoothing operator distance.

Grids generated by computer packages can be manipulated to:
- limit the grid to a desired polygon,
- expand to fill a given area,
- mask out (or in) any areas,
- take into account faults, discontinuities, and trends, fill
- be scaled, added, subtracted, multiplied, divided, or restricted to positive values by logarithmic gridding.

The algorithm which chooses data points is important in that the number of points for interpolation plays a significant role. Where the data is suited, kriging has an advantage over other methods which often represent compromises between the mathematically desirable and the computationally feasible. Though these other methods are reasonable for many applications they may give biased interpolation, whilst they provide no estimate of the error of interpolation, nor do they attempt to minimise that error.

1.3 Kriging

Kriging is a form of weighted local averaging that is an optimal means of spatial prediction in the sense that it provides estimates of values at unrecorded places without bias and with minimum and known variance. It is based on the theory of regionalised variables developed by Matheron (1963) and Krige (1966) for the estimation of ore reserves in mining.

1.3.1 Variograms

Kriging depends on first computing an accurate semi-variogram, which measures the nature of spatial dependance for the property. Estimates of semi-variance are then used to determine the weights applied to the data when computing the averages, and are presented in the kriging equations.

The semi-variance is expressed as:

$$\gamma(h) = \tfrac{1}{2} VAR[z(i) - z(i+h)] = \tfrac{1}{2}\sum [z(i) - z(i+h)]^2/n \qquad (2)$$

and is a measure of the similarily, on average, of an observation z at point i and another point at a given distance h away. In other words, the semi-variance is the average half-squared difference between all pairs of points separated by the same distance, h. The quantity $\gamma(h)$ can be estimated for integer values of h from the data and the graph of $\gamma(h)$ versus h is the semi-variogram.

The semi-variogram has certain important characteristics which (a) reveal the nature of the geographic variation in the property of interest, and (b) are needed to provide kriged estimates at previously unrecorded points. These are described in reference to Figure 1.

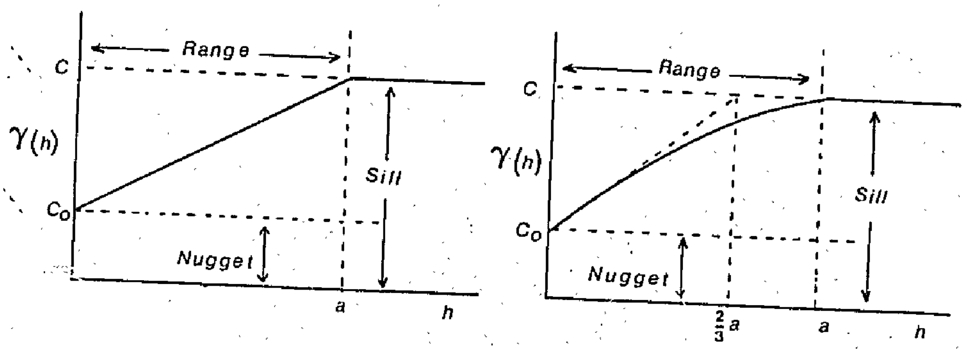


Figure 1. Theoretical Variograms.

In most cases it is found that $\gamma(h)$ increases with increasing h to a maximum, approximately the variance of the data, at a moderate value of h, say a. The distance is known as the range. Points closer than the range are spatially dependant; points further apart bear no relation to one another, unless there is a periodic variation in the soil. When interpolating, the aim is to use only those points closer than the range to the predicted point.

By definition $\gamma(h) = 0$ when $h = 0$. However, in practice, any smooth curve that approximates the values of the semi-variance is unlikely to pass through the origin. Instead there appears to be a positive finite value to which $\gamma(h)$ approaches as h approaches 0. This intercept is known as the nugget variance, and in general is known as the nugget effect. The terms derive from sampling practice in gold mining where the inclusion of a gold nugget in narrow core is a somewhat chance event. The nugget effect accounts for different results in sampling the same site twice. This may occur for reasons such as poor analytical precision, poor sampling practice, or actual erratic values at low scale. Most semi-variograms of soil properties show nugget effects (Burgess and Webster, 1980a). The nugget variance embraces fluctuation in the soil that occurs over distances much shorter than the sampling interval, and limits the precision of interpolation.

The value at which $\gamma(h)$ levels out is known as the sill. It represents the range of variance due to spatial dependence in the data.

There is no general mathematical formula to describe the shape of soil semi-variograms. A linear model, $\gamma(h) = C_0 + mh$, is simplest, and will often describe $\gamma(h)$ well within range. A spherical model, given by

$$\begin{cases} \gamma(h) = C_0 + C\left(\frac{3}{2}\frac{h}{a} - \frac{1}{2}\left(\frac{h}{a}\right)^3\right) & \text{for } 0 < h \leq a \\ \gamma(h) = C_0 + C & \text{for } h > a \end{cases} \qquad (3)$$

may also be used. Other models (De Wysian, exponential, and hole effect) are described by David (1977).

1.3.2 Simple kriging

When estimating a value $z(X_0)$ of a property z at point(X_0) where X is the vector notation (i.e. $X = [x, y]$), the linear sum, or weighted average, of the observed value is expressed as:

$$z_0 = \lambda_1 z(X_1) + \lambda_2 z(X_2) + \ldots\ldots\ldots\ldots \lambda_n z(X_n). \qquad (4)$$

where the λ are coefficients or weights associated with the data points, as discussed in section 1.2.

In kriging, the weights are so chosen that the error associated with the estimate is less than for any other linear sum. The weights take account of the known spatial dependences expressed in the semi-variogram and the geometric relationships among the observed points. In general, near points carry more weight than distant points, points that occur in clusters carry less weight than lone points, and points lying between the point to be interpolated and more distant points screen the distant points in that the latter have less weight than they would otherwise.

The model for simple kriging, analogous to equation (1), section 1.1, for usual survey practice, is:

$$z(X) = \mu_v + \epsilon(X) \qquad (5)$$

where $z(X)$ is the value of the property at X within a neighbourhood v, μ_v is the mean value in that neighbourhood and $\epsilon(X)$ is a spatially dependent random component with zero mean and variation defined by:

$$VAR[\epsilon(X) + \epsilon(X + H)] = E[\{\epsilon(X) + \epsilon(X + H)\}^2] = 2\gamma(H) \qquad (6)$$

and equals $2\gamma(h)$ if variation is isotropic. It is assumed that μ_v is constant for the neighbourhood, though different neighbourhoods may have different means, and that the semi-variogram is the same over the whole area. The last assumption implies that there are no sharp boundaries (faults, cliffs). If such boundaries are known to exist then interpolation is carried out separetely on either side.

The coefficients (or weights) are calculated using the equations:-

$$\sum_{j=1}^{n} \lambda_j \gamma(X_i, X_j) + \mu = \gamma(X_i, X_0) \quad \text{for } i = 1,2,3,\ldots..n \qquad (7)$$

where μ is a Lagrange multiplier

The matrix notation is given by : $\begin{bmatrix}\lambda\\ \mu\end{bmatrix} = A^{-1}B$

$$A = \begin{bmatrix} \gamma(x_1,x_1) & \gamma(x_2,x_1) & \gamma(x_3,x_1) & 1 \\ \gamma(x_1,x_2) & \gamma(x_2,x_2) & \gamma(x_3,x_2) & 1 \\ \vdots & \vdots & \vdots & \\ \gamma(x_1,x_n) & \gamma(x_2,x_n) & \gamma(x_n,x_n) & 1 \\ 1 & 1 & 1 & 0 \end{bmatrix} ; \quad B = \begin{bmatrix} \gamma(x_1,x_0) \\ \gamma(x_1,x_0) \\ \vdots \\ \gamma(x_n,x_0) \\ 1 \end{bmatrix} \tag{8}$$

$$\begin{bmatrix}\lambda\\ \mu\end{bmatrix} = \begin{bmatrix}\lambda_1\\ \lambda_2\\ \vdots\\ \lambda_n\\ \mu\end{bmatrix} \tag{9}$$

The minimum estimation variance is σ_E^2 given by:

$$\sigma_E^2 = B^T\begin{bmatrix}\lambda\\ \mu\end{bmatrix} \tag{10}$$

The accuracy of kriged estimates depends on the goodness of the computed semi-variogram and two precautions are taken to ensure that the values of $\gamma(h)$ used in the kriging equations are satisfactory. First, the spatial analysis should be performed on long runs of data (or a number of short runs), so that the semi-variances at short lags can be computed from many pairs of comparisons. Second, a sensible model must be chosen to describe the results, and individual estimates of $\gamma(h)$ can be weighted according to the number of comparisons on which the they are based when fitting the model.

An example of simple kriging (Farrelly, 1985) is shown in Appendix I

1.3.3 Block kriging

In simple kriging, the grid points at which we make estimates represent volumes with the same size and shape as the volumes of soil from which the original property was measured. For example, if observations are derived from 10cm diameter cores, then the estimated grid points are strictly cylinders 10cm diameter. If the observations were test pits, then the computed grid points would also represent test pits. Although sampling is carried out in this fashion for convenience, economics and time, the observation at a single sample point is usually taken by the observer to represent the surrounding area, or at least the area nearer to it than any other sample point. When interpolating the geologist may wish to interpolate an average value for an area or block many times larger than the actual sampled volume.

Kriging can be carried out over areas, in a procedure known as block kriging. In block kriging, instead of considering a point X, we consider a region ϑ with an area H with its centre at X.

The semi-variances between data points and the interpolated point are replaced by the average semi-variances between the data points and all the points in the region. Thus each $\gamma(X_i, X_0)$ of equation 9 is replaced by the integral $\int \gamma(X_i\ X)\,\rho(x)\,d(x)$ where $\rho(x)$ is given as follows:

$$\rho(x) = \frac{1}{H_{\vartheta}} \quad \text{if } X \text{ belongs to } \vartheta \tag{11}$$

$$\rho(x) = 0 \quad \text{otherwise,}$$

and (12) $\int \rho(x)\,d(x) = 1$

The weights for block kriging are therefore given by

$$\begin{bmatrix} \lambda \\ \mu \end{bmatrix} = A^{-1} s \tag{13}$$

where

$$s = \begin{bmatrix} \int \gamma(X_1, X)\ \rho(x)\,d(x) \\ \int \gamma(X_2, X)\ \rho(x)\,d(x) \\ \vdots \quad \vdots \quad \vdots \\ \int \gamma(X_n, X)\ \rho(x)\,d(x) \\ 1 \end{bmatrix} \tag{14}$$

The estimated variance for the area H is

$$\sigma_H^2 = s^T \begin{bmatrix} \lambda \\ \mu \end{bmatrix} - \iint \gamma(X, Y)\,\rho(x)\ \rho(y)\ d(x)\ d(y) \tag{15}$$

Although a map drawn from point estimates is the more accurate isopleth map, local minor variation can obscure regional trends. Block kriging results in a smoother map showing average values calculated over a number of broader areas.

1.3.4 Universal kriging

A third means of kriging, universal kriging, takes into account local trends in data when minimising the error associated with estimation. The presence of such trends or drifts is identified qualitatively, and their form found quantitatively by one of two methods. Either (1) a structural analysis may be carried out, which simultaneously estimates semi-variances of the differences between the drift and the actual data. The resulting semi-variograms are then used for the interpolation. Or (2) prior generation of a regional surface and semi-variograms are calculated for the residuals. Simple kriging is then used to produce the numeric surface.

Universal kriging is not comprehensively applicable to soil survey (Webster and Burgess, 1980), mainly because of the large nugget variances usually encountered, which arise in part because measurements are made on small widely separated volumes of soil. These effectively prevent any distinction between constant and changing drift.

Universal kriging would not be applicable to the data obtained at Melton for these reasons.

1.4 Soil thickness mapping - Definitions and parameters

Soil thickness, like many geological parameters, has a certain amount of subjective judgement incorporated into a definition. Past work in the area (MacIsaac and Key, for the Melton Sewage Authority, 1972) has set a definition of soil thickness as being the depth to powered-auger refusal. A map depicting "rock contours" was produced and used as a guide in the drafting of tenders for the installation of sewerage and storm-water pipelines. Since cost of excavation increases rapidly when basalt is encountered, such guides are valuable.

In the engineering geological sense, soil is defined by "all unconsolidated materials above bedrock" (Bates and Jackson, 1980). This definition includes cobbles and boulders ('floaters') surrounded by soil which are common in residual basaltic soils and such floaters are sufficiently large and unweathered to cause powered-auger penetration refusal when encountered. Thus, power-auger refusal may be a misleading definition for soil depth, although the only feasible one. This in turn, results in a variation soil depth over short distances (which accounts for the large nugget effects in the semi-variograms).

Where a sample point represents an excavated test-pit, the soil depth is taken as depth to excavator refusal. If the sample point represents a borehole drilled by a percussion or diamond drill, then the soil depth is judged from the borelogs as being the top of the first encountered rock which would refuse penetration by a powered-auger.

Sample points where rock was not encountered and points where penetration refusal occured on alluvial gravels or calcareous nodules were not included in the computations. However, they were used in checking the accuracy of the isopleth map on completion.

Approximately 1100 sample points (boreholes and test-pits) were available for scrutiny within the map area. Of these, 648 were selected into the data base which formed the basis for the soil depth computations. The selected points represented those for which a positive soil depth value could be given.

Since the data were collected from several sources, the quality varied according to the origin. For some of the data the locations were approximate, as the coordinates were scaled from locality diagrams included in reports. For other data the sample locations were surveyed.

The soil depth values were generally precise. The only exception was where data from percussion drilling was included - the depths tended to correspond to the length of a drill-rod (i.e. 1.50m, 3.00m, etc.) in most of the locations. However, for one metre isopleths, this data remains valid.

The quality of the data is tabulated below.

Source	Number of data points	Location technique	Estimated error (metres)	Drilling method	Estimated soil depth error (metres)
GSV	184	Surveyed	Nil	Auger + pit	Nil
	24	Scaled	5	Unknown	1
MSA	207	Scaled	5	Auger + diamond	Nil
RCA	105	Scaled	2	Auguer + pit + diamond	Nil
F&L	105	Scaled	2	Percussion	0.5
				Diamond	Nil
Other	23	Scaled	20	Various	0.5

Table 1. Soil Depth Data Quality.

The distribution of data (Fig. 2) presents a challenging difficulty in numeric surface calculations. The 'clumping' of the sample points makes gridding difficult, because the uneven distribution makes the selection of mesh size a problem.

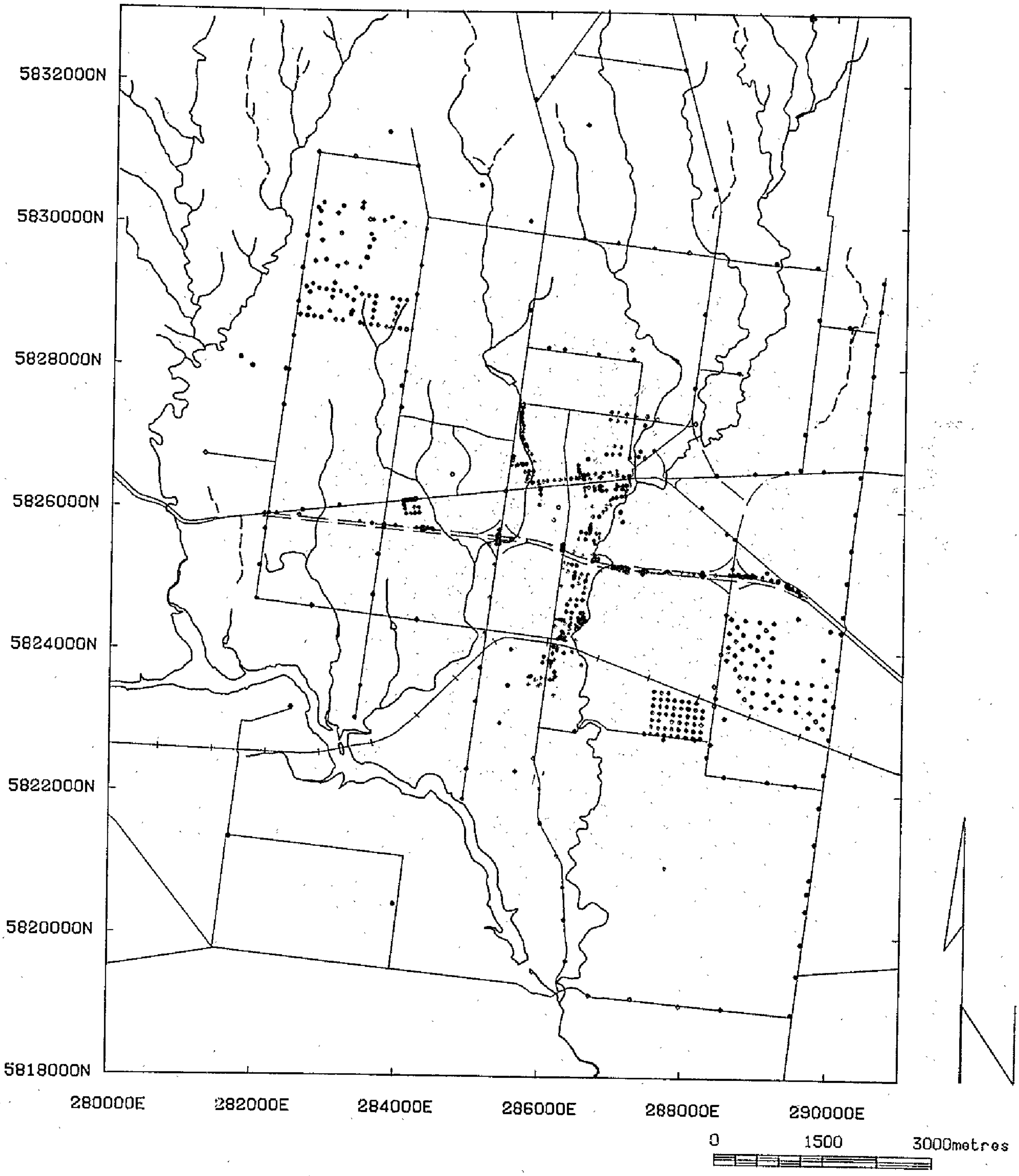
1.5 Numeric surface computation

The numeric surface representing soil depth was computed using a package supplied by Engineering Computer Services Pty Ltd (ECS). The program - GPCKRG - is part of an interactive general purpose gridding and contouring package known as GPC/GPCINT.

GPCKRG allows the computation of the semi-variograms, the fitting of either a simple linear or spherical model, and kriging using simple, block or universal kriging methods. During the computation of the grid manipulations may be made such as

* including trends or faults,
* expansion of the grid beyond the data points,
* applying smoothing operators to the grid,
* restricting the gridding to a defined polygon,
* limiting the grid to positive values only,
* the use of a sample location tolerance to simulate sampling error,
* limiting the interpolation to a given range, and
* including data from outside the grid area.

10



LEGEND

o - DATA POINT

ENGINEERING GEOLOGICAL MAPPING
OF MELTON, VICTORIA.

Figure 2.
DISTRIBUTION OF SOIL DEPTH DATA

PROJ NO. GE 14	DATE: 16-NOV-87

The computed grid may then be masked to include or exclude given areas.

1.5.1 Variogram computation

For the data set available, semi-variograms were computed to establish the suitability of the data to kriging. The shape of the variogram computed (Fig. 3) showed a distinct nugget effect, range and sill value. This indicated that the data set is suited to treatment by kriging.

Semi-variograms were computed in the four cardinal directions to check for any possible anisotropies (Fig. 4). The resultant variograms showed no substantial differences in their shapes, which indicated a lack of anisotropy in the data set. The similarities of shape in the directional variograms also indicate the absence of strong regional trends (or drift), which alleviates the need for universal kriging.

From the total variogram, the modelling parameters were chosen. A spherical model was judged to best fit the data, with a nugget of 0.5 metres, a sill of 3.8 metres, and a range of 400 metres. The model is plotted on the variogram in Figure 5.

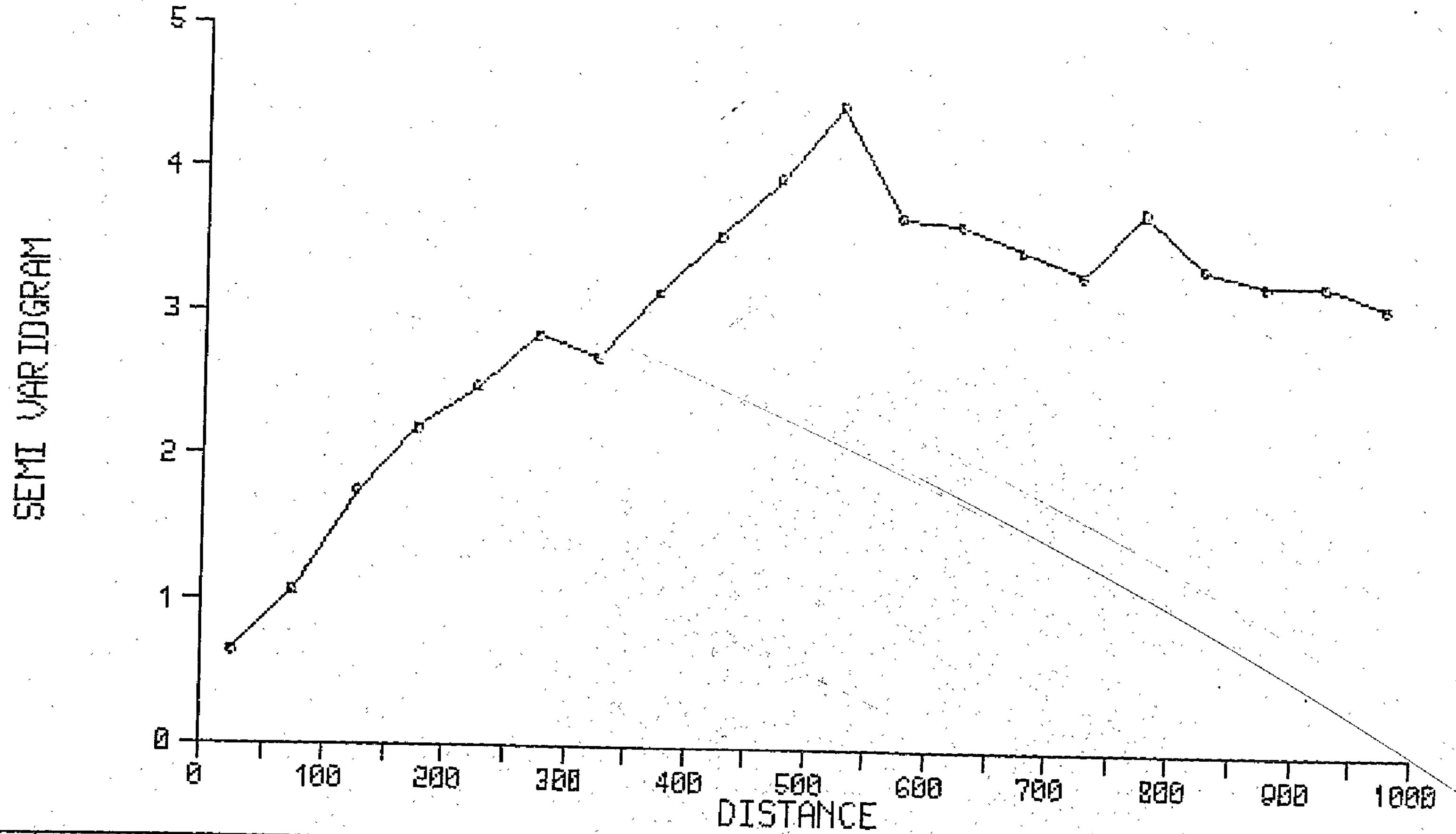
Several interesting observations can be made from the variograms. The average nugget effect indicates that the uncertainty in sampling the same location twice is 0.5 metres. More simply, this means that the soil depth can only ever be predicted to the nearest half-metre, even in the most frequently sampled locations. The average range of 400 metres indicates that the soil depth can be predicted (with a calculated confidence) from an observation in one place to another place up to 400 metres distant, after which there is no relationship. The sill of 3.8 metres represents the average difference in observations greater than 400 metres apart, or more simply, the maximum error in prediction.

1.5.2 Kriging

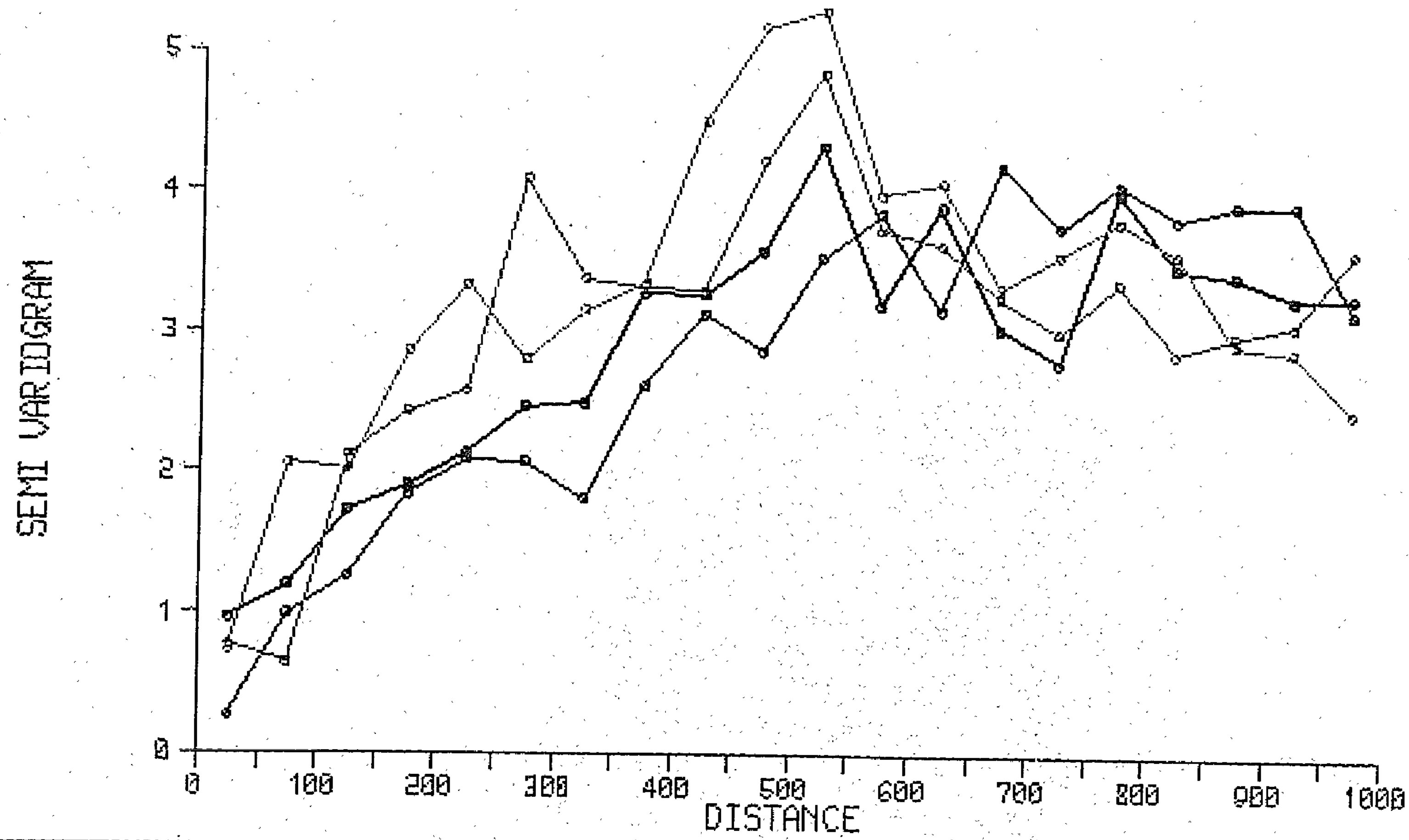
The variograms illustrate that the data set is suited to either block or simple kriging techniques. Kriging the data in blocks of 100 X 100 metres (same as the grid mesh size) was chosen as the most applicable method. This choice is based on an examination of the end requirement, ie. the production of a soil depth map which indicates the average thickness in an area, without being site-specific. By using block kriging the map indicates the average thickness that would be encountered over 100 X 100 metre cells (ie. 10,000m^2) which provides a suitable basis for making decisions pertaining to an area, rather than a specific site.

The gridding parameters were as follows:

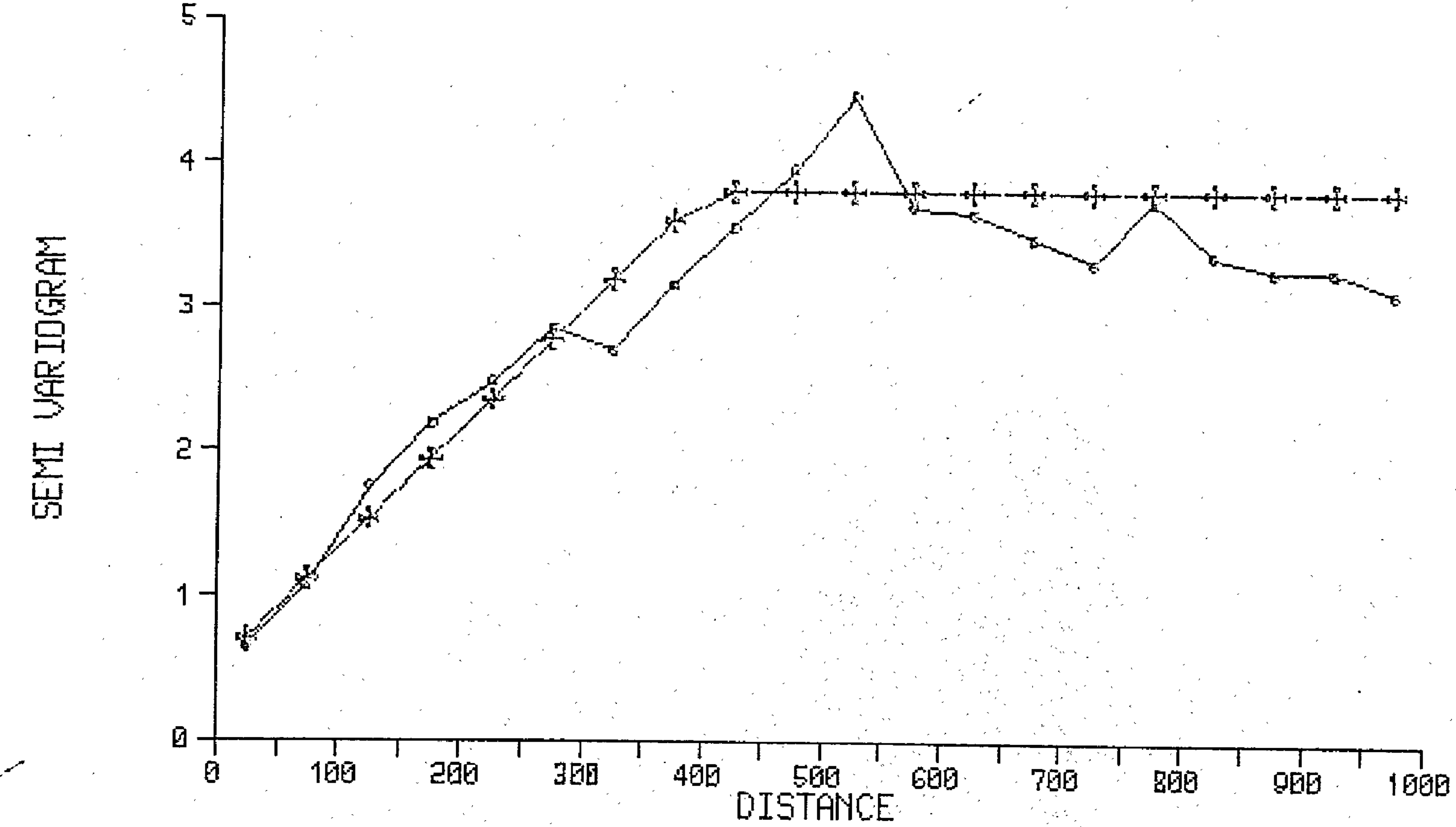
mesh size	100 X 100 metres
scan distance	2800 metres
data distance tolerance	5 metres
points searched per octant	2



No.	Covariance	Population																			
0	3.54	233	508	767	736	997	950	071	037	266	106	161	146	267	118	229	229	301	382	505	520



No.	Covariance	Population																			
1	3.48	70	191	219	231	291	229	305	251	348	278	349	264	360	291	359	288	302	322	392	324
2	3.19	37	88	184	180	237	252	227	217	232	232	199	258	222	184	220	209	210	222	204	248
3	3.68	85	155	187	195	271	230	297	291	362	354	345	344	375	336	358	412	455	454	478	502
4	3.67	41	74	177	130	198	239	242	278	324	242	268	280	310	307	292	320	334	384	431	446



No.	Covariance	Population																			
0	3.54	233	508	767	736	997	950	071	1037	266	106	161	146	267	118	229	229	301	382	505	520

There was no:

- grid rotation,
- extrapolation beyond the mesh points,
- logarithmic gridding,
- smoothing,
- limit to interpolation, and
- restriction to a boundary polygon,

The grid was later masked to exclude the gridded area outside of the Melton development area boundary. This was carried out by utilising another GPC program (GRDMSK).

The gridding parameters were chosen to best fit the density and distribution of the data. The scan radius naturally presents a problem, since it can be seen from the variograms that the radius of search should be 400 metres. Using sample points 2800 metres away to predict mesh point values is not reliable (even though their weighting would be very small), but unfortunately necessary to prevent gaps in the grid. The only alternative would be to acquire more data.

By using a data distance tolerance two problems are simultaneously solved. The first is that kriging is a process which honours the data (subject to the mesh size selected). This assumes that the sampling is repeatable, even when a nugget effect is present, which is not always valid. The use of a distance tolerance simulates possible error in sampling and results in non covariance between samples, which relaxes the criterion to honour the data. The second problem is that much of the data was scaled from plans and diagrams to provide AMG co-ordinates. This introduces a real error (assumed to be in the order of five metres), which can be accounted for by the data distance tolerance.

1.6 Validity checking

The resultant soil depth plot required a validity check in order to assess the end result in terms of the real data. For this purpose an isopleth plot was compared with plots of the distribution of sample points in each range (Figs 1 to 10, Appendix II). The comparison was good, as would be expected from the gridding method chosen.

A rigorous test was performed by comparing the resultant isopleth map with plots of sample locations which were not included in the data set, but for which minimum a depth of soil is known (Figs 11 to 17, Appendix II). These sampled locations are those where the borehole or test pit did not encounter rock, and so strictly were not originally included. The comparison was generally very good, although in one small area known soil depths were consistently deeper than that predicted by the numeric surface. In this area an adjustment was made by including bores with the known soil depth.

The resultant variogram was little changed (the sill was adjusted to 3.6 metres), and so the overall adjustment to the grid was minimal.

1.7 Numeric surface representation

Two numeric surfaces are produced by the kriging techniques described above.

1 Soil depth
The resultant grid of soil depth values is illustrated as a shade-colour plot in Figure 6. By using shade colour, the minor variations are not so obviously displayed as on the corresponding isopleth plot (Fig. 7). The overall impression of gradation is given, rather than the concept of distinct boundaries.

2 Confidence values
The second surface computed by the program represents the confidence placed on the soil depth values. This surface is illustrated by a shade colour plot in Figure 8. The confidence values are expressed in metres and depict the 'plus-or-minus' values that can be placed on the predicted soil depth values at any given place. As expected, the confidence is high in areas where the data is dense, and poor in areas where data is sparse (as modelled by the variograms).

The confidence values highlight the lack of data in some areas of the map, such as the southeast corner, where the soil depth can be read as one metre plus-or-minus three and a half metres. However, the philosophy behind the attempt to illustrate soil depth is that any information is better than none at all, and the confidence grid can be included as a reliability diagram.

1.8 Geological interpretation

The resultant isopleth map of soil depth illustrates some interesting trends which have geological implications.

The variation in depth of soil may be due to two main causes. Firstly, differential weathering, or weathering that occurs at different rates as a result of variations in composition and resistance of a rock, or differences in intensity of weathering due to topographic or climatic conditions. Secondly, differences in geologic age of various surfaces, resulting in the development of younger and older weathering profiles.

In the Melton development area, the variations in soil depth can be interpreted as being due to a combination of geologic age and differential weathering. For example, the area of Melton closest to Mt Cottrell (i.e. the south-eastern portion of the map) shows uniformly shallow soil. This implies that some lava flows from the past volcanic eruptions are represented here by their shallow weathering profiles. The edge of the flow is bounded by Toolern Creek, the other side of which the soils thicken. These thicker soils would be formed on older lava flows and so have had time to weather deeper at an accelerated rate due to the moisture provided by Toolern Creek.

Similarly, the shallow soils in the north-west area would represent the most recent lava flow from the eruption point at 'Melton Park'. The thicker soils south and east of this flow would represent the soils in the older flows.

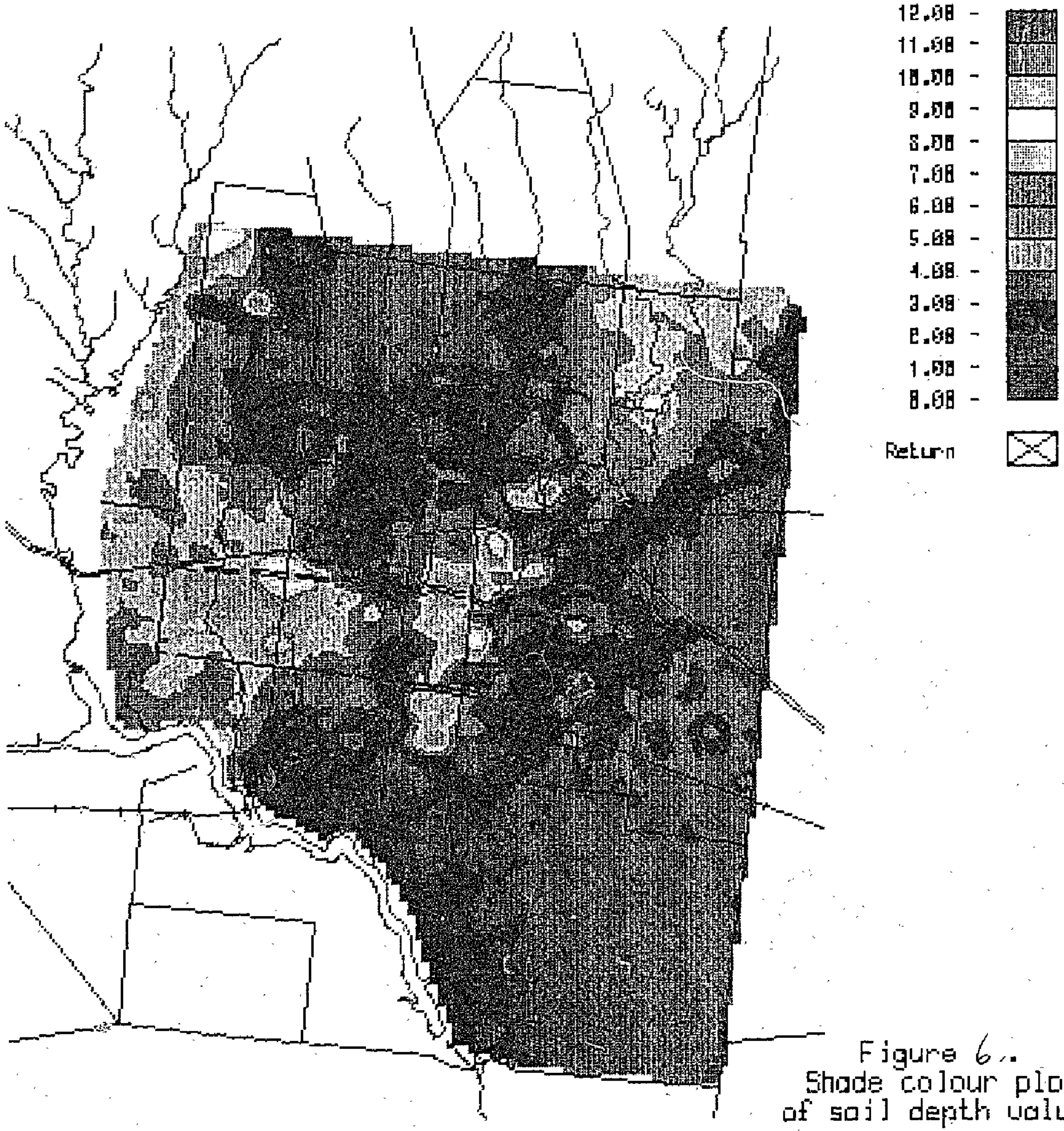
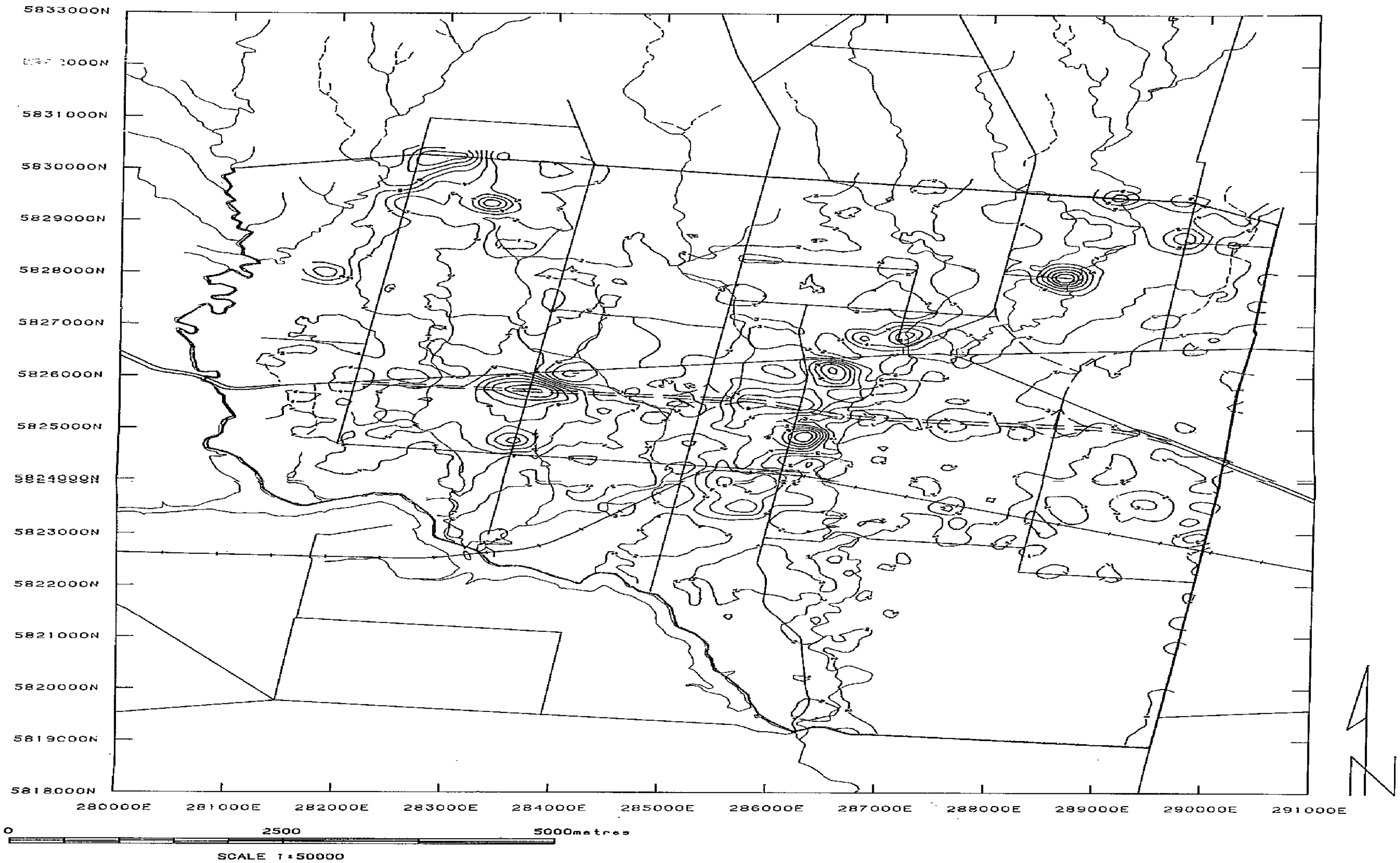


Figure 6.
Shade colour plot
of soil depth values



Numeric surface generated
using block kriging -
100 X 100 m. blocks.

Isopleth interval
- one metre
< 3 m. - red
3 - 6 m. - magenta
6 - 9 m. - purple
> 9 m. - blue

ENGINEERING GEOLOGICAL MAPPING
OF MELTON. VICT.

Figure 7.

SOIL DEPTH ISOPLETHS

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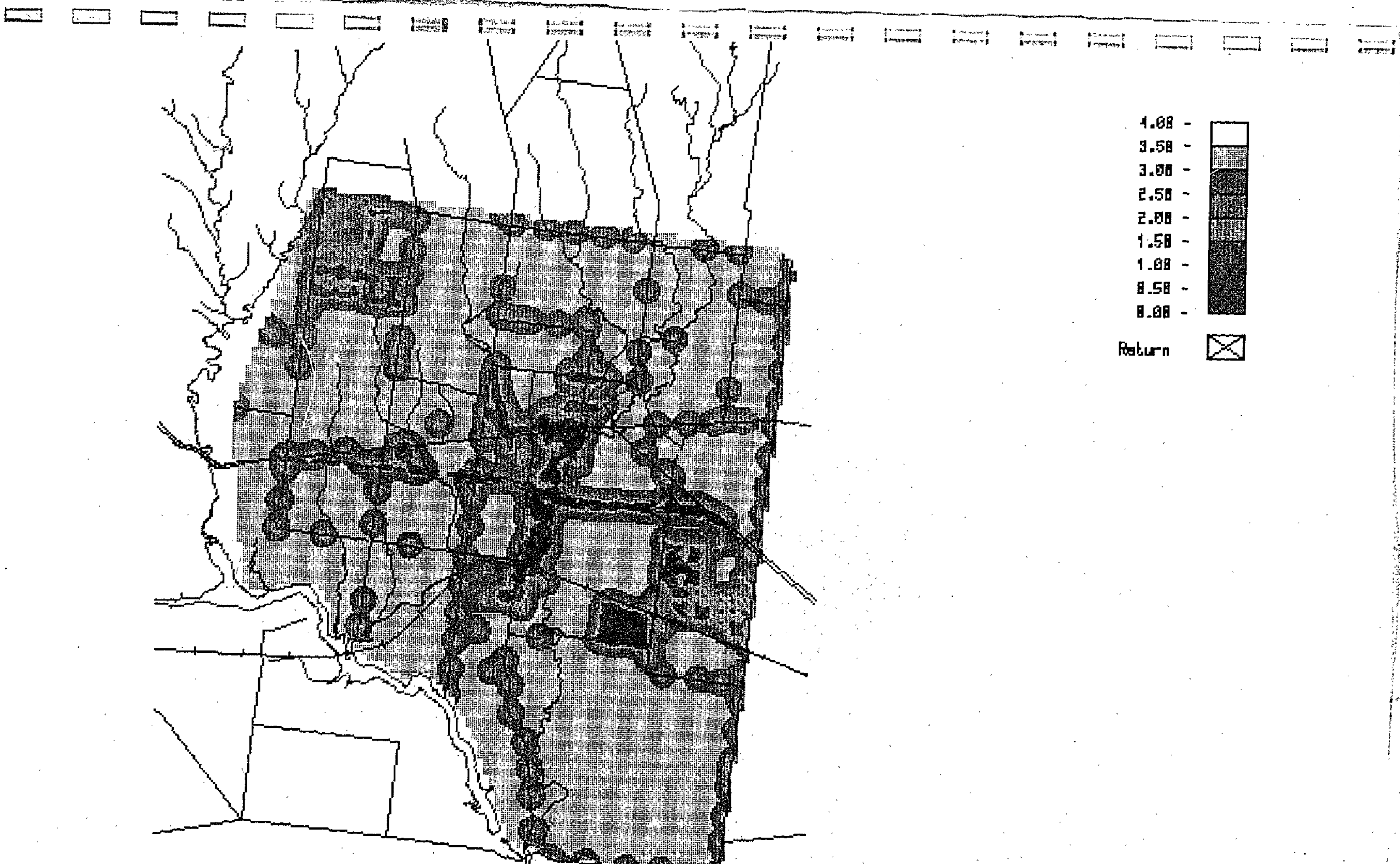


FIGURE 8. SOIL DEPTH MAP
CONFIDENCE VALUES

1.9 Geotechnical interpretation

The soil isopleths are a very useful guide to geotechnical parameters for decision making in development planning. The areas where soils are thin present problems for the servicing of developments with sewerage and storm-water drainage. In these areas excavation would often be carried out by expensively and laborously drilling and blasting.

Areas where soils are thick represent the problem areas for housing development, as the basalt soils have high shrink-swell potential, and maximum heave would occur on the thickest soils (section 4.2). Similar consequences relate to road making.

1.10 Further work

The upgrading of the soil isopleth map can be very easily carried out by adding new information to the data set and computing new numeric surfaces to plot. This would refine both the accuracy and confidence of new plots.

One spin-off from using kriging as a gridding technique is that sampling programs can be easily planned to achieve a required confidence value. For example, if soil depths to the nearest metre were required to be known in an certain area, then by perusing the variograms an optimal grid spacing could be chosen to achieve the desired result. This could be useful for the development of large subdivisions, where it is advantageous to know where shallow rock occurs.

2 Groundwater

The nature of the groundwater regime can be an important consideration in the engineering development of urban environments. Shallow groundwater tables can cause settlement of building foundations, as can over-extraction of a groundwater resource (Leggett, 1973). Large expanses of pavement increase runoff from storms and decreases the total amount of water recharging underground supplies. Similarly small residential lot size renders large areas impervious (Leopold, 1968).

Groundwater was not encountered at any time during the drilling program. Several open standpipes were installed in the deeper soil areas, and with one exception, groundwater was not encountered during one year of monitoring. The one exception (Djerriwarrh 102) was adjacent to the Toolern Creek, where water was encountered in gravel beds.

Table 2 documents the monitoring program.

Aspects of the occurrence and distribution of groundwater in the region have been documented by Kenley (1960 & 1977), Thompson (1972), Rhia (1975 & 1976), Plier-Malone (1977), and Williams (1983 & 1986). Essentially, the groundwater in the region occupies the two major sub-horizontal geological formations, the Newer Volcanics and the Werribee Formation (U.R. 1986/4).

Bore	Installation	17.11.'83	8.12.'83	29.3.'84	18.7.'84	16.8.'84	19.9.'84	17.10.'84	15.11.'84
		Date & Readings							
Djerriwarrh 100 Harkness Rd	10.11.'83 7.5 m Dry	7.5 m Dry	7.5 m Dry	7.5 m Dry	7.5 m Dry	7.54 m Dry	7.59 m Dry	7.57 m Dry	7.56 m Dry
Djerriwarrh 101 Hardy's Rd	11.11.'83 2.75 m Dry	2.7 m Dry	2.7 m Dry	2.7 m Dry	-	2.47 m Wet	Area flooded	1.98 m Wet	2.0 m Wet
Djerriwarrh 102 Shire Pound	14.11.'83 8.75 m Dry	7.21 m Wet	7.0 m Wet	7.15 m Wet	7.59 m Wet	7.42 m Wet	7.53 m Wet	7.17 m Wet	Destroyed
Djerriwarrh 103 Buckle Crt	15.11.'83 12.5 m Dry	12.4 m Dry	12.4 m Dry	12.4 m Dry	12.4 m Dry	11.94 m Dry	11.94 m Dry	11.96 m Dry	11.96 m Dry
Djerriwarrh 104 The Bullock Trk	16.11.'83 5.2 m Dry	5.1 m Dry	5.1 m Dry	5.1 m Wet	5.0 m Wet	4.83 m Dry	4.88 m Dry	4.89 m Dry	4.89 m Dry
Djerriwarrh 105 Cnr Harkness Rd & Porteous Rd	18.11.'83 12.5 m Dry	-	12.0 m Dry	12.0 m Dry	12.0 m Dry	11.94 m Dry	11.36 m Wet	11.38 m Dry	11.36 m Dry
Djerriwarrh 106 Bulmans Rd	21.11.'83 7.4 m Dry	-	7.0 m Dry	6.9 m Dry	6.9 m Dry	6.74 m Dry	6.79 m Wet	6.79 m Dry	6.79 m Dry
Djerriwarrh 107 Bulmans Rd	23.11.'83 8.0 m Dry	-	7.5 m Dry	7.5 m Dry	7.5 m Dry	7.29 m Dry	7.31 m Dry	7.32 m Dry	7.31 m Dry
Djerriwarrh 108 Bulmans Rd	23.11.'83 3.4 m Dry	-	3.3 m Dry	Destroyed					
Kororoit 67 Cnr Ryans Rd & Finchs Rd	25.11.'83 8.1 m Dry	-	8.0 m Dry	8.0 m Dry	8.0 m Dry	7.83 m Dry	7.86 m Dry	7.86 m Dry	7.86 m Dry

Table 2. Open standpipe readings.

Note. From 16.8.'84 the accuracy of recording was improved.
Groundwater encountered in bore Djerriwarrh 102.
Bore Djerriwarrh 101 was situated in low-lying area.

The Newer Volcanics consist of layered sequences of basalt flows and interbedded soil layers of low permeability. The basalts are differentially cracked by large numbers of thermal contraction joints which represent about 0.5% by volume of the rock and serve both as the main space for water storage and the channelways for water movement. They generally contain at least two separate aquifers - upper and lower - which in places are vertically interconnected and operate as a two aquifer system (Kenley, 1977).

The upper basalt aquifer is an unconfined water table aquifer which is recharged by direct slow infiltration of rain or stream water. The lower basalt aquifer is a confined low pressure aquifer. Water enters this aquifer in areas where it locally outcrops and partly also by vertical leakage from the upper aquifer in places where the low permeability interbeds are lacking.

The Werribee Formation contains a number of porous sand and gravel layers each of which behaves as a confined aquifer under considerable hydrostatic head. The groundwater in this formation occupies the pore spaces between the sand grains which may represent up to 25% by volume of the sand. These sands do not outcrop in the map area, and recharge may be from the north and west where the Formation outcrops, or from downward leakage from the basalt aquifers.

Information on the groundwater in the Melton Development Area is generally lacking. Only 36 of the researched bores had standing water level measurements, while 45 had groundwater quality information. Two numeric surfaces were computed from the data.

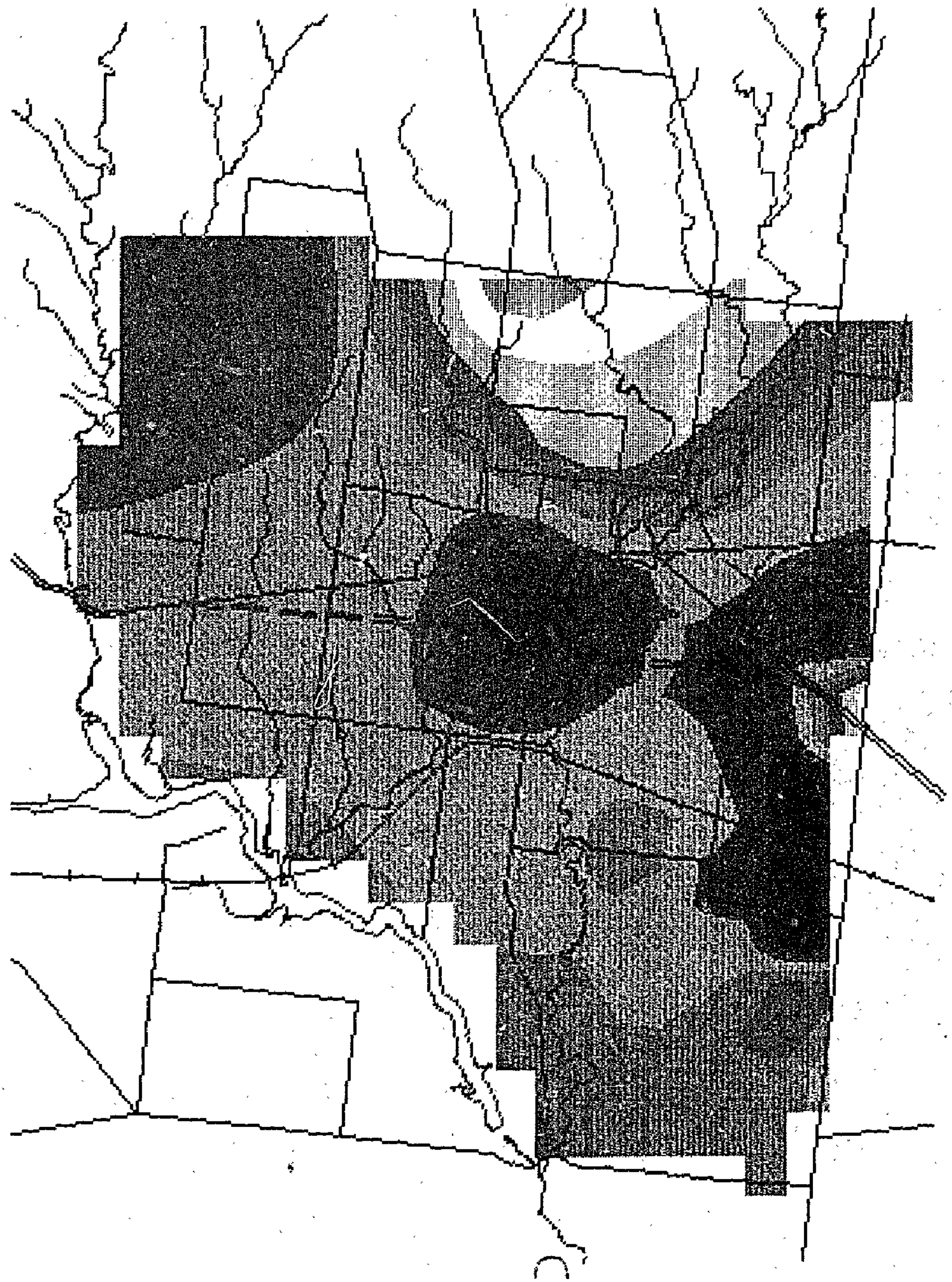
The first - a grid of the standing water levels - was computed using a general purpose gridding program (GPCGRD; ECS, 1986). The second - a grid of the height above the AHD of the potentiometric surface - was computed by subtracting the standing water level grid from a grid of the topographic surface. These surfaces are illustrated in Figures 9 & 10.

2.1 Water Quality

Table 3 details the quality of the groundwater within the map area.

In general terms, the groundwater in the Melton map area is of the sodium-chloride type with some magnesium sulphate. There are, however, some differences both in concentration and percentage.

A major study of the groundwater conditions of a proposed quarry site (bounded by Ferris Rd, Mt Cottrell Rd and the North-Western Railway) found that two water masses, one in the upper and one in the lower aquifer, can be identified from water quality (Williams, 1986). Figure 11 illustrates this difference where the Stiff diagrams for the upper and lower aquifers from test pump well samples and two farm bores have been compared. The difference is due to a greater percentage of magnesium chloride in the lower aquifer and sodium bicarbonate in



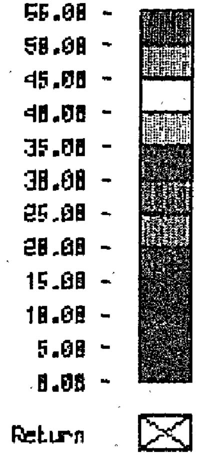


FIGURE 9. STANDING WATER LEVELS

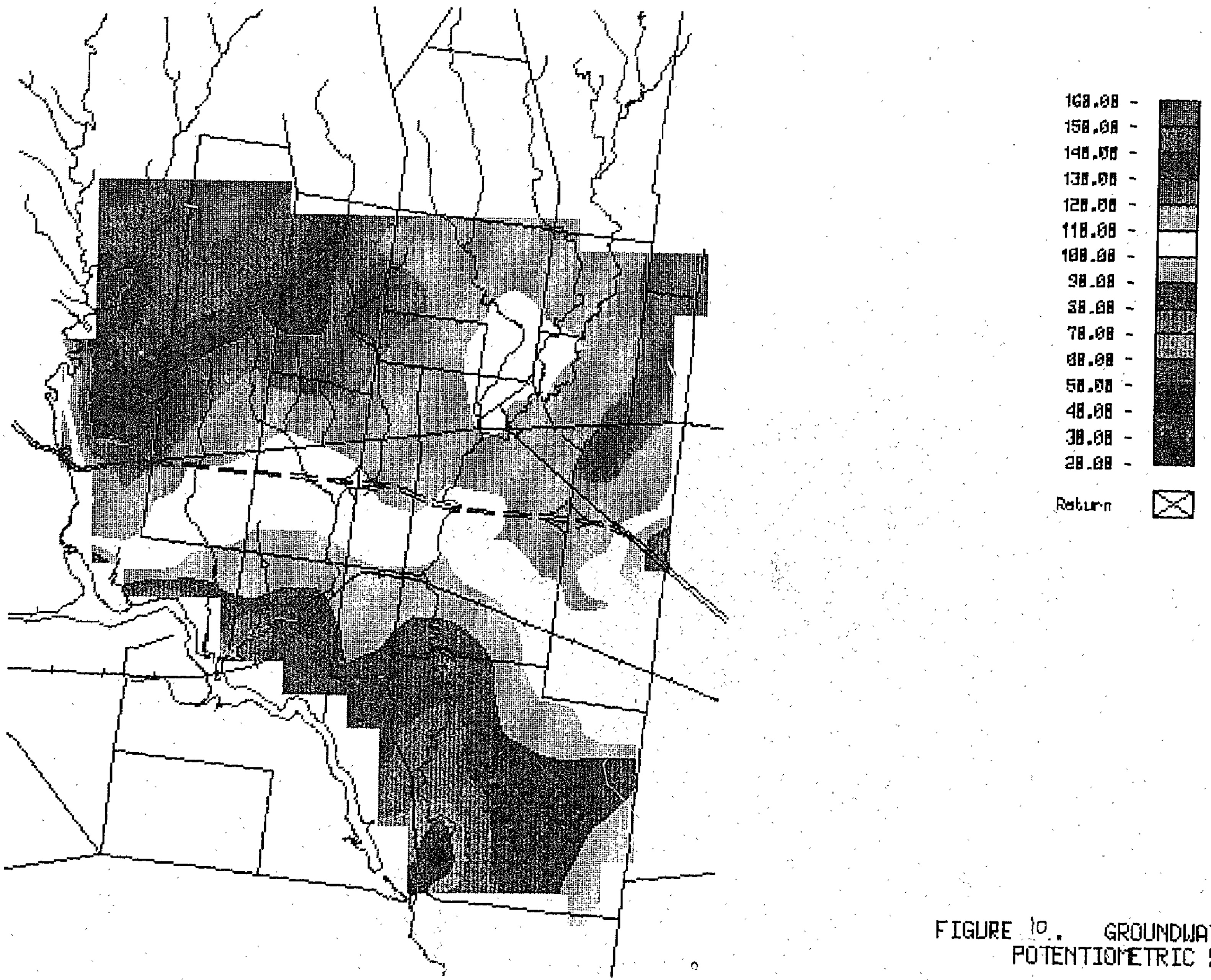


FIGURE 10. GROUNDWATER POTENTIOMETRIC SURFACE

BORE	pH	TDS mg/l	$CaCO_3$	Fe soluble	Fe total	ANALYSIS Na	K	Ca	Mg	SiO_2	Cl	SO_4	CO_3	HCO_3	Cond. omhs
D8001	7.5	5110									2250			774	
D8003	7.6	613	171	.01	.36			21	29	11	169	65		187	
D8003	7.6	584									194			208	
D8004	7.8	5930									2956		6	462	
D8005		7260									3320			1244	
D8006		5040									2400		33	790	
D8007		10080									5070			447	
D8008	7.8	4080									2300				
D8009	8.1	3400									1508		30	408	
D8010	7.8	4880									2446			530	
D8011	8.1	1826	625	.04	.13			67	111	30	832	77		311	
D8012	8.2	3270									1615			303	
D8013	8.0	3240									1645			335	
D8014	7.8	10380									5330			1007	
D10001	8.1	2061	348		.20	539	13	32	65	72	742	105	0	493	3200
K16	8.4	3016	840	.00	.20	850	15	42	83	76	1116	158	30	632	4500
K8001	8.0	6600	1940	.03	.07	1340	29	104	410	23	2570	640	0	290	
K8002	8.2	3580	1020	.02	.09	880	12	56	210	39	1500	270	0	680	
K8006	7.6	2420									1130			391	
K8007	8.0	1740									807			362	
K8008	8.0	3900	960	.03	.46	940	20	68	190	65	1700	240	0	460	
K8035	7.9	2396	656	.00	.40	573	20	25	144	72	1078	154	0	316	3840
K8036	7.8	3500	880	.06	.18	870	16	68	170	67	1520	250	0	500	
K8053	8.2	5746	1544	.00	.02	1530	38	58	340	59	2960	418	30	309	9300
K8070	7.9	5090	990	.03	.09	1420	39	76	190	43	2310	440	0	390	
K8071	7.9	2450	380	.02	.35	740	24	48	63	40	1010	210	0	440	
K8072	8.0	3300	990	.08	.09	790	32	96	180	56	1300	430	0	410	
K8086	7.8	2060	190	.02	.49	690	14	22	33	50	830	170	0	460	
K8087	7.8	2800	590	.02	.49	730	20	66	103	21	1230	270	0	290	
K10009	8.2	2884	846		1.50	677	15	70	163	55	1280	192	12	419	4710
K10009	8.1	2480	250	.02	.02	850	15	32	41	54	1050	240	0	460	
K10011	7.9	2061	156	.01	4.80	621	9	18	27	64	639	142	0	507	2995
K10011	7.8	2020	180	.02	.02	690	11	24	29	67	790	150	0	500	
K10012	8.1	3803	612	.01	2.20	1100	22	59	113	59	1550	318	0	553	5950
K10015	7.9	5752	895	.01	.00	1750	32	108	152	44	2730	608	0	317	9000
K10019	8.5	2300	550		.09	560	17	64	97	51	890	170	0	488	
P8003	8.4	1817									647		60	406	
P8004	8.5	1807	200	.02	.84			13	41	19	635	72	65	409	
P10001	7.8	8347	2680					207	526		3980			100	12884
P10010	8.2	1972	306	.01	.70	542	11	17	64	31	739	85	0	481	3175
Y8001	8.2	7260									3830		6	251	
Y8002	7.8	3240									1603			243	
Y8003	8.0	2340									1080		6	240	
Y8004	8.3	3500									1584			308	
Y8006	7.8	3030									1331		18	511	
Y8007	8.1	3460									1513		24	631	
Y8009	7.9	2540									1240		12	396	
Y10004	7.8	7510									3730			407	12000

Key to bores: D - Djerriwarrh; K - Kororoit; P - Pywheitjorrk; Y - Yangardook.
(u) - upper aquifer (l) - lower aquifer

Table 3. Water quality.

the upper aquifer. These differences are probably caused by the residence time being less in the upper aquifer. The analyses also show a clear cut salinity gradient with depth. In the upper aquifer total dissolved solids (TDS) values of 2020 - 2480 mg/l are recorded while in the lower aquifer the values ranged from 2800 - 6600 mg/l.

With the exception of one bore (Djerriwarrh 8003), all the groundwater recorded has levels of total dissolved solids higher than generally recommended for human consumption. The World Health Organisation (WHO) sets a desirable limit of 1500 mg/l with an objective limit of 500 mg/l. The sodium content is well in excess of the taste threshold recommended by the WHO of 150 mg/l. Generally, the iron content is such that it would cause staining of bathroom fixtures and could stain clothes washed with the water.

The groundwater would be suitable for some stock watering, the limits for TDS and magnesium in drinking water of livestock are given in Table 4.

The agressiveness of water is a more complicated quality to determine, involving many variables related to both the water chemistry and flow rate and the nature of the material under attack. The State Chemistry Laboratory consider that, in general, water with a TDS of 3000 mg/l or greater is probably aggressive toward metal.

	Total Soluble Salts mg/l	Magnesium mg/l
Poultry	3500	-
Pigs	4500	-
Horses	6000	250
Cows in milk	6000	250
Ewes with lambs	6000	250
Beef cattle	10000	400
Adult sheep on dry feed	14000	500

Table 4. Limits for total soluble salts and magnesium in drinking water of livestock.

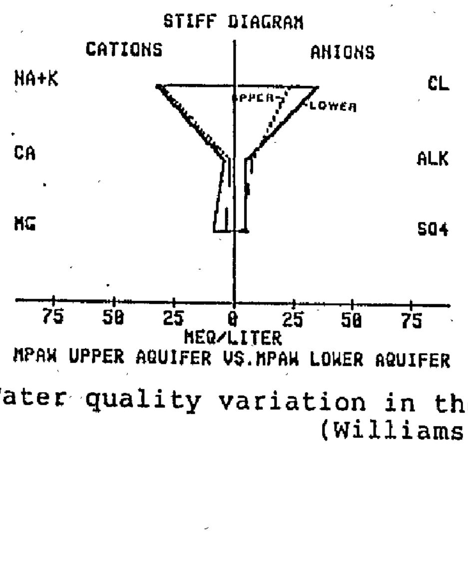


Figure 11. Water quality variation in the aquifers. (Williams,1986)

3 Engineering properties of the soil

One problem with the quantity of test data is that there are insufficient to form accurate numeric surfaces. Gridding the data and producing isopleth maps is not feasable because of the extreme variability in the data over short distances (in three dimensions), which makes prediction speculative.

The distribution of soil samples tested to determine the various engineering properties is illustrated in Figure 12. The data set is essentially a scatter of points (in three dimensional space) which require graphical representation to study their relationship in any configuration. These graphical pictures are useful to confirm or contradict previous concepts, and may reveal new ideas in a dramatic way.

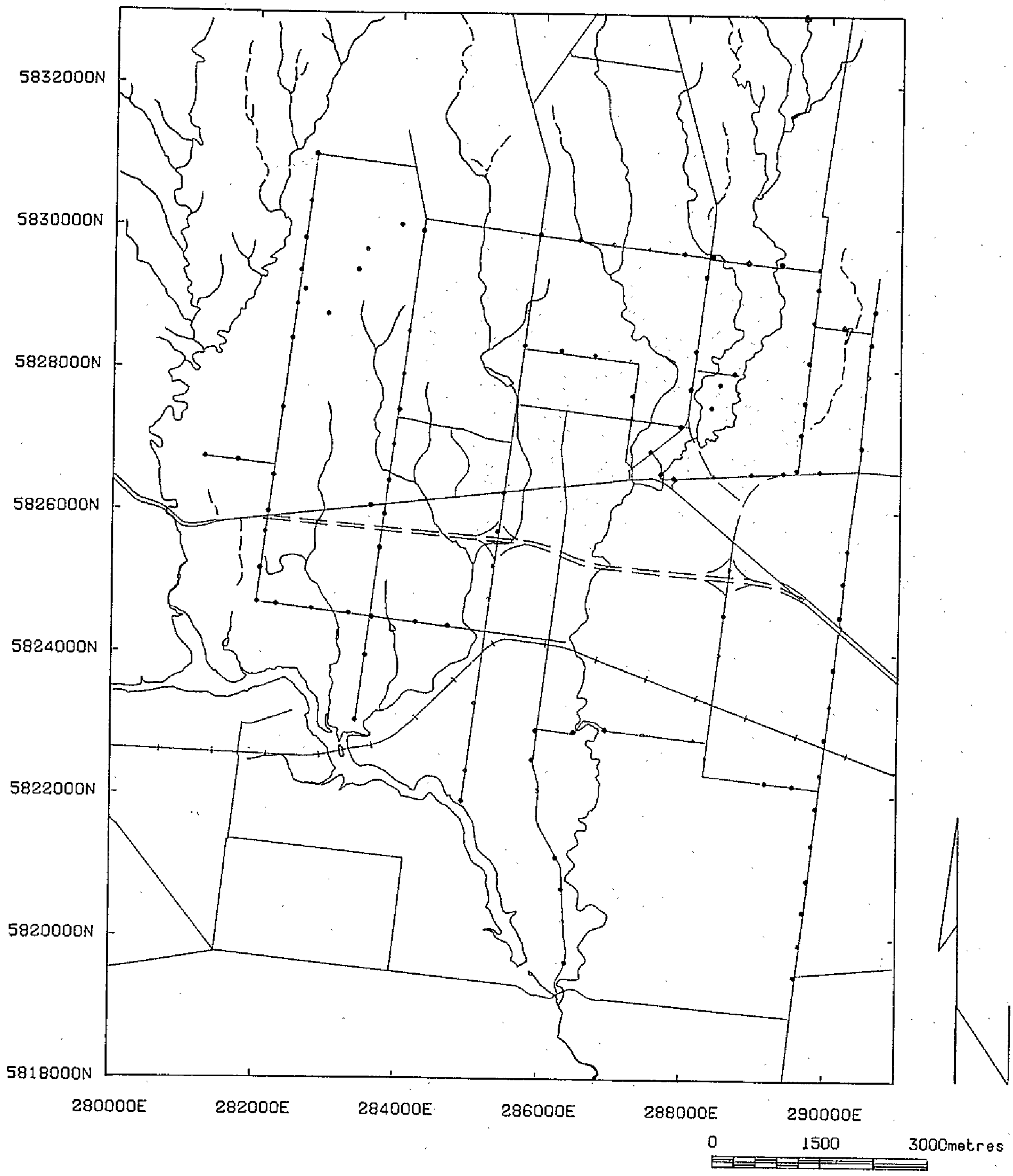
There are many of these graphical techniques available. The simplest code a single numerical value into a simple character (Fig. 13). Others code single values of two or more variables into one compound character (Fig. 14).

When designing or choosing compound character scales, consideration must be given to whether the scales are separable (i.e. whether one can easily shift attention from one coded aspect to another), and whether the coded aspects are individually value-mergeable into impressions of regional trends.

Of the compound character scales shown in Figure 14, the most unusual and versatile is the Chernoff face (Chernoff, 1973). A revised version of the face by Davis (Bruckner, 1978) allows the coding of up to 20 variables (Fig. 15 & Table 5). Much has been written on the merits and demerits of the use of Chernoff's faces (Wang, 1978).

The major difficulty in trying to represent the Melton soil test data is that the data has variation in all three dimensions (i.e. there are multiple 'z' values at any x,y location). Representation of this foliation or layering at quite different levels of 'z' has not been solved. One suggestion is the procedure of locating the 'most imposing gap' in the collection of 'z' values for nearby (x,y) points followed by smoothing (Tukey & Tukey, 1980). Then to study the foliated structure several kinds of plots can be generated, such as smoothed gap location values, display the original points coded in some way to indicate which layer they are in, or make separate displays for the points in each layer. All these methods, however, are inappropriate, since the variables still require 'layering' into intervals.

In representing geochemical data, the use of 'flag maps' can overcome the difficulty of three dimensional representation (Farrelly, pers. com., 1984). Figure 16 shows the liquid limit values illustrated as a flag map. From this it can be seen that the geographic variation and the variation with depth is not clearly represented. In general, the area to the north and west have higher liquid limit values than the areas to the south and east.



LEGEND

o - DATA POINT

ENGINEERING GEOLOGICAL MAPPING OF MELTON, VICTORIA.
Figure 12. DISTRIBUTION OF SOIL TEST DATA
PROJ NO. GE 14 DATE: 16-NOV-87

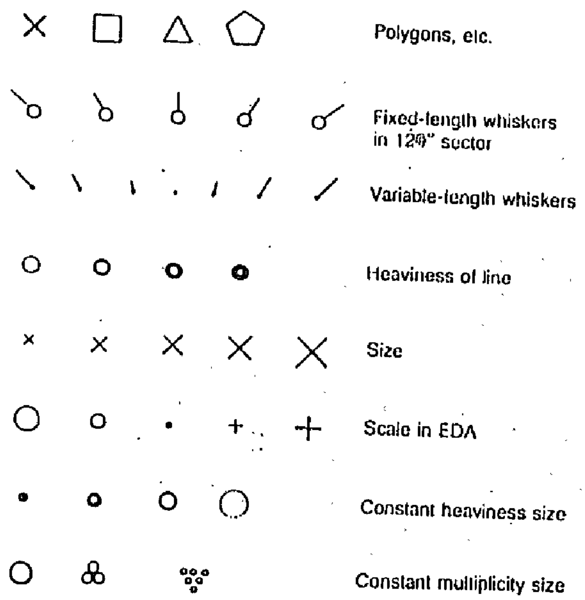


Figure 13. Several possible individual-value simple character scales. (Tukey & Tukey, 1980)

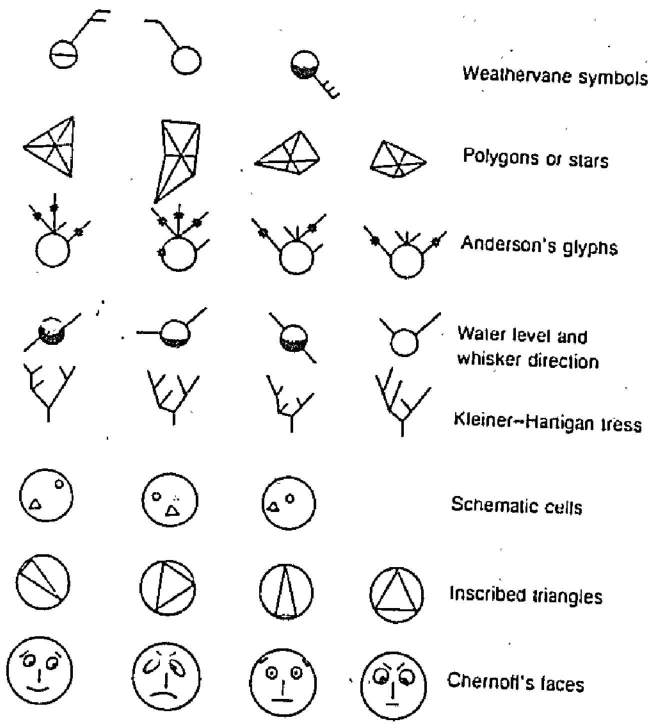


Figure 14. Several possible individual-value compound character scales. (Tukey & Tukey, 1980).

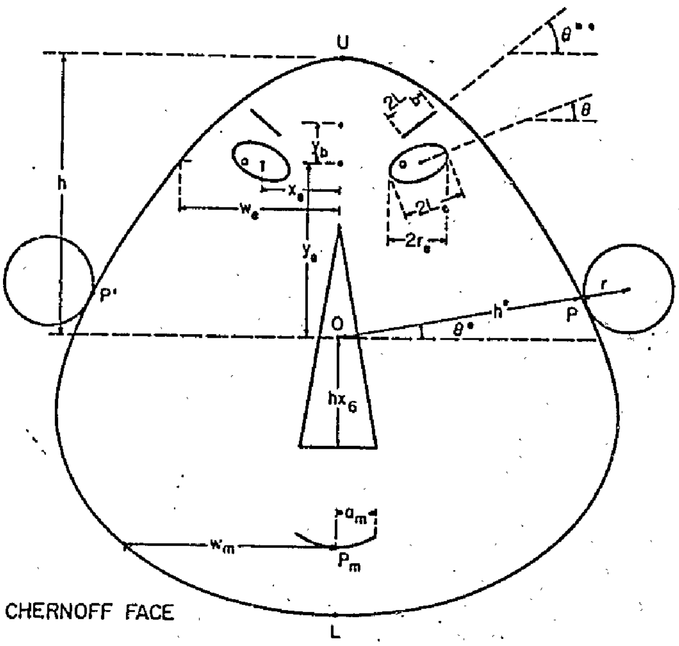


Figure 16. Chernoff Face. (Bruckner, 1978)

Variable		Facial Feature	Default Value	Range	
x_1	controls h^*	face width	.60	.20	.70
x_2	controls θ^*	ear level	.50	.35	.65
x_3	controls h	half-face height	.50	.50	1.00
x_4	is	eccentricity of upper ellipse of face	.50	.50	1.00
x_5	is	eccentricity of lower ellipse of face	1.00	.50	1.00
x_6	controls	length of nose	.25	.15	.40
x_7	controls P_m	position of center of mouth	.50	.20	.40
x_8	controls	curvature of mouth	0.00	4.00	4.00
x_9	controls	length of mouth	.50	.30	1.00
x_{10}	controls y_e	height of center of eyes	.10	0.00	.30
x_{11}	controls x_e	separation of of eyes	.70	.30	.80
x_{12}	controls θ	slant of eyes	.50	.20	.60
x_{13}	is	eccentricity of eyes	.60	.40	.80
x_{14}	controls L_e	half-length of eye	.50	.20	1.00
x_{15}	controls	position of pupils	.50	.20	.80
x_{16}	controls y_b	height of eyebrow	.80	.60	1.00
x_{17}	controls $\theta^{**} - \theta$	angle of brow	.50	.00	1.00
x_{18}	controls	length of brow	.50	.30	1.00
x_{19}	controls r	radius of ear	.50	.10	1.00
x_{20}	controls	nose width	.10	.10	.20

Table 5. Description of facial features and ranges. (Bruckner, 1978)

The correlation of high Atterberg limit values, LS values and FS values with areas of deeper soil is quite good (Fig. 17) These areas with high swell potential and deep soils constitute the worst possible conditions for development. In these areas the soil will exhibit large shrink-swell values and have the maximum depth (therefore the maximum volume) for heave (section 4.2)

4 Suitability for Development

4.1 Past assessments

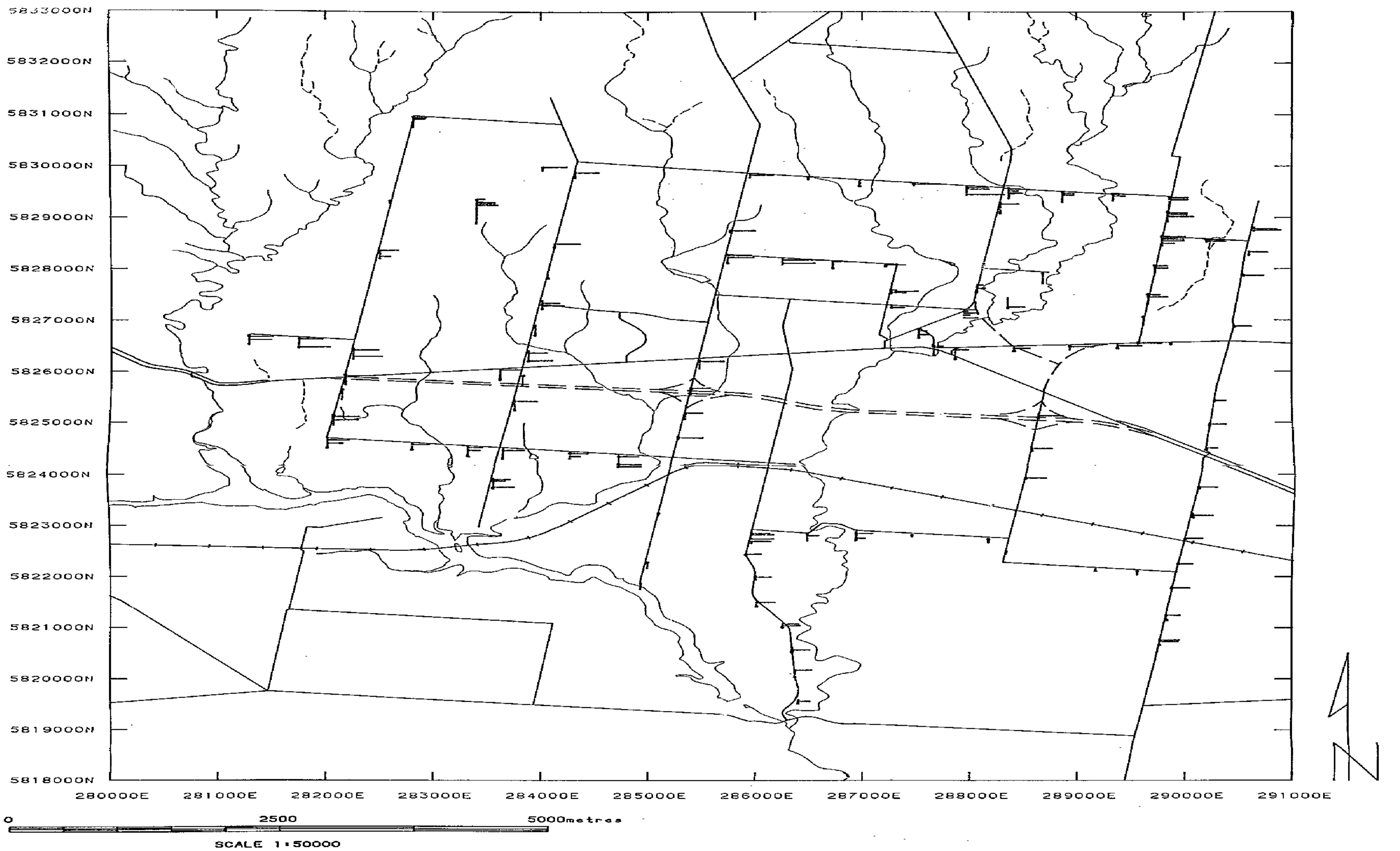
In their assessment of the existing environment at Melton, Clarke Gazzard Planners Pty. Ltd. found:

> "Foundation conditions experienced in Melton are similar to those in Melbourne's Western Suburbs.
>
> The material underlying most of the study area is of basaltic clay type. It is moderately expansive but does not present serious problems nor impose significant cost penalties in normal types of single and double storey domestic and commercial construction.
>
> Design of services and foundations needs to compensate for some seasonal fluctuations.
>
> Minor areas such as that in the south east near Greigs Road where Rockbank Association soil occurs could present problems in road construction. Special measures may need to be taken to combat erosion in the zones of alluvium associated with Toolern Creek and most extensive in the north east and near the reservoir in the south west of the Designated Area." (Clarke Gazzard, 1976)

This assessment was based on a CSIRO terrain classification map (Grant, 1972) produced at a scale of 1:250000, and enlarged to approximately 1:55555 for inclusion in their report. The Terrain Patterns of this map compare remarkably closely to the 1:250000 Melbourne geology map (1972). Both these maps are erroneous.

On the matter of soil classification (presumably in the engineering sense), Clarke Gazzard Planners Pty Ltd concluded:

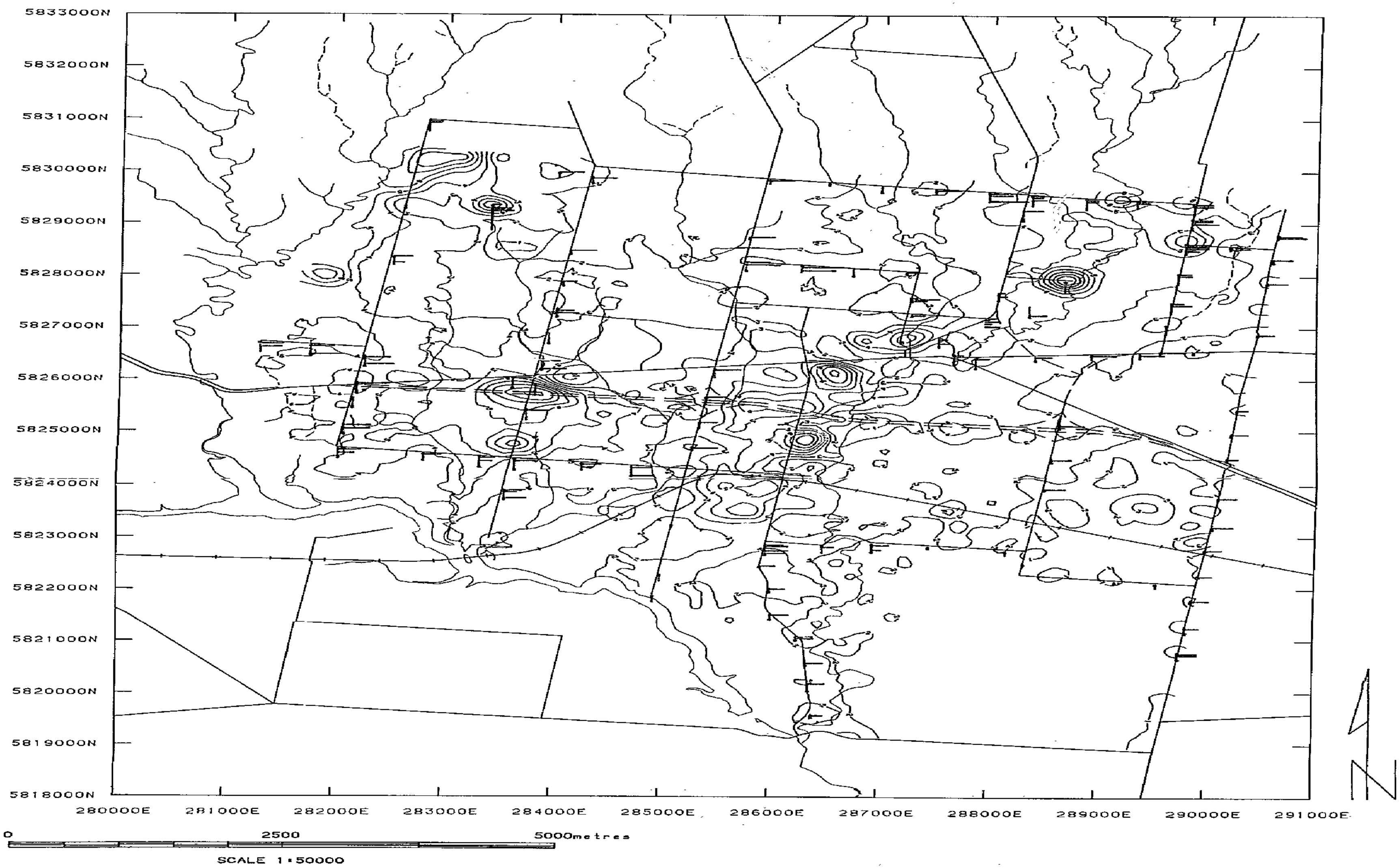
> "The major proportion of the study area is underlain by a basaltic clay type material. This material is similar to that which occurs in the western suburbs of Melbourne, in summer being very hard with considerable surface cracking whilst in winter it is moist and puggy. It does not present any serious problems in regard to conventional domestic or commercial types of building up to two storeys in height when properly designed to account for the seasonal movements and the expansive properties of the soil. Excavations for footing or services is somewhat more difficult because of the presence of rock floaters which vary considerably in size and may require the use of explosives to achieve the desired shape. Such difficulties in excavation are reflected in higher earthwork costs than would apply in other areas.
>
> Based on the preliminary information available soil conditions in the study area do not represent a planning constraint." (Clarke Gazzard, 1976)



Bore depth shown
in brown
Scale 1:1000

Liquid limit value
shown in green
Scale 1:20

ENGINEERING GEOLOGICAL MAPPING OF MELTON. VICT.			
Figure 16. LIQUID LIMIT VALUES			
PROJ NO.	GE 14	DATE:	16-NOV-87



Numeric surface generated using block kriging – 100 X 100 m. blocks.

Isopleth interval – one metre
< 3 m. – red
3 – 6 m. – magenta
6 – 9 m. – purple
> 9 m. – blue

Bore depth shown in brown
Scale 1:1000

Liquid limit value shown in green
Scale 1:20

ENGINEERING GEOLOGICAL MAPPING OF MELTON. VICT.

Figure 17.
SOIL DEPTH ISOPLETHS AND LIQUID LIMIT VALUES

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33

Following recommendations made in the abovementioned report, the DVA was commissioned to examine the requirements for drainage and flood mitigation at Melton. As a part of this study the DVA requested the SCA to report on the land capability of the area, part of which examined the suitability of particular areas for subdivision and septic effluent disposal.

The investigation identified 12 separate land units, based on topography, drainage line entrenchment and soil types. (Fig. 18) The capabilities for the various units for urban subdivision are presented in Table 6 (SCA, 1978).

This study highlights the benefits of an initial rapid assessment, for planning purposes. The important aspect is that certain land was recognised as being poorly suited to subdivision.

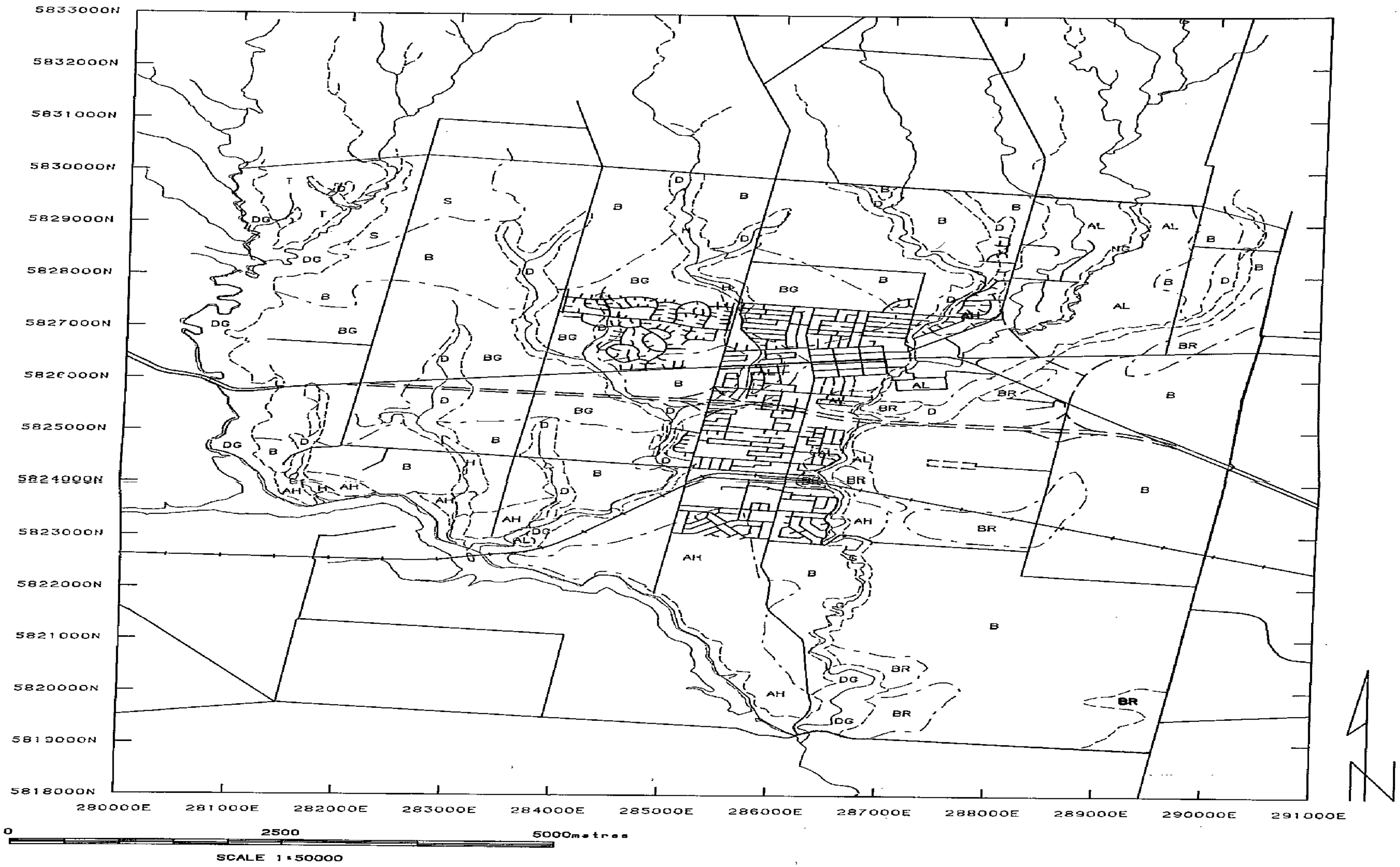
A further study by White and Kelyneck (1985) delineated 32 map units describing a specific topographic element and associated soil type. The capability of the land was then assessed for various land utilisations (viz. secondary roads, septic tank absorption fields, building foundations, farm dams, shallow excavations, rural subdivisions, and urban developments).

This study emphasizes the pitfalls in rapidly producing maps which imply that a detailed study of the area has been made. Examples of poor assessments are easily found - the area affected by gilgai and subsidence 'sinkholes' (U.R. 1986/4) is rated as "good" for building foundations while the area between Toolern Creek and Gisborne-Melton Road (which is here considered good) is rated by the assessment as being "very poor".

4.2 Building Foundations

The geological conditions beneath urban areas provide the ultimate support for all structures in that city. The relevance of the engineering properties of the geological foundation materials has therefore been studied for centuries. Since the evolution of cities in society, the construction of buildings has been subject to regulatory control, often including rules regarding foundation conditions. The Code of Hammurabi (2067-2025 BC) is thought to be the first set of building regulations ever recorded (Leggett, 1973).

In Victoria, the building regulations first incorporated strict control of the foundations for building construction in 1980, when an amendment to the Uniform Building Regulations (UBR) was introduced in response to pressure from the building and insurance industries to decrease the incidence of distress in domestic housing. There were several reasons for this. Firstly, the trend in housing from the 1950's had been toward single-leaf masonry veneer construction ('brick veneer'). This form of construction is less tolerant to movement (i.e. less flexible) than the 'weatherboard', or 'fibro' houses of the pre-1950's. In addition, the growing desire for quality and the increased



NON CREEK ASSOCIATED UNITS

B - Basalt Plain
BG - Gilgaid Basalt Plain
BR - Stony Rises on Basalt
AH - Higher Alluvial Terrace
S - Sinkhole Plain
T - Tertiary Plateau

CREEK ASSOCIATED UNITS

D - Dished Drainage Lines
H - Hilly Sided Drainage Lines
G - Stony Gorge
DG - Deep Gorge
NG - Northern Toolern Creek Gorge
AL - Low Level Alluvium

ENGINEERING GEOLOGICAL MAPPING OF MELTON, VICT.

Figure 18.

SCA LAND UNITS

(SCA, 1978)

PROJ NO.

DATE: 16-NOV-87

Land Unit and Map Symbol		
Non creek associated units	Capability	Factors causing limitations
Basalt Plain (B)	Fair	Shrink-swell, drainage, rock, erosion during development
Gligaied Basalt Plain (BG)	Fair to Poor	Shrink-swell, drainage, rock, erosion during development
Stony Rises on Basalt (BR)	Fair to Poor	Depth to rock
Higher Alluvial Terrace (AH)	Fair	Shrink-swell, drainage, erosion during development
Sink-hole Plain (S)	Poor	Subsidences
Tertiary Plateau (T)	Fair	Erodibility
Creek Associated Units		
Dished Drainage Lines (D)	Fair	Lower areas flood, shrink-swell, erosion during development
Hilly-sided Drainage Lines (H)	Poor	Slope, depth to rock, flooding on low level terraces
Stony Gorge (G)	Very Poor	Slope, rock outcrop
Deep Gorge (DG)	Very Poor	Steep, unstable banks
Northern Toolern Creek Gorge (NG)	Very Poor	Unstable, steep banks

Table 6. SCA Land Units - Development limitations.

awareness of consumer rights meant that house owners were less forgiving toward signs of distress. Less geologically suitable land was being subdivided as Melbourne's western and south-western suburbs grew. A trend toward tree planting in home gardens, particularly native gardens, also increased the incidence of distress, since most of the problems were with seasonal movement of expansive soils. This same ammendment to the regulations also required that a builder guarantee his/her work for six years.

In 1983 the Victoria Building Regulations (VBR) were introduced to bring Victoria into general compliance with regulations in other states. The relevant section of these regulations covering footings and foundations is included as Appendix III. Essentially these regulations state that foundations are to meet three requirements:

i) Assessment of adequacy - (regulation 32.2)

The adequacy of foundations shall be based on approved

a) well established and relevant local knowledge and experience of foundation conditions in the vicinity of the proposed building; or

b) tests on the foundation materials.

ii) Allowable bearing pressures - (regulation 32.4)

the bearing pressure on the foundation of a building shall not exceed the values given in the regulations unless-

a) an investigation of the foundations has been conducted and the building surveyor is satisfied in the light of the report on that investigation, that higher bearing pressures are justified; or

b) an investigation of the site has been conducted under AS1726 and the bearing pressures are based on the information obtained from that investigation.

iii) Foundation classification (sub-regulations 3 & 4, regulation 33.4)

The classification of the foundation of any site on which footings are to be constructed shall be in accordance with the "Classification of Expansive Behaviour of Melbourne Soils for Domestic Construction" published by CSIRO and AEBIRA (Australian Engineering and Building Industry Research Association). (sub-regulation 3)

In areas outside the Metropolitan area (those shown in figure 3 of the publication referred to in sub-regulation (3)) the site on which the footings are proposed to be constructed shall be similarly classified by adopting, where practicable, the principles stated in that publication and taking into account experience or knowledge of local or traditional building construction practices. (sub-regulation 4)

The last of these three requirements has special relevance at Melton. The classification system relates the expected expansive behaviour of the foundation to the performance of the minimum standard footing design recommendations. Three categories of movement are used, viz. stable, intermediate and unstable.

The soils of Melbourne are subdivided primarily in terms of their geological origin. A simplified map of the major soil types referred to in the CSIRO and AEBIRA publication is reproduced as Figure 19. Soils of the one geological origin are then further subdivided on the basis of their typical soil profile. Table 7 reproduces the classification.

The requirements for the classification of sites and the design and construction of residential slabs and footings are now covered by Australian Standard AS2870-1986 (Residential Slabs and Footings). The standard was prepared in response to an Australia-wide need for guidance on the design of slabs and footings for houses, and although a wide range of conditions is covered, the standard places particular emphasis on the design for reactive clay sites susceptible to significant ground movement due to moisture changes. The standard may be used to satisfy the requirement that the structural design of footings and floor slabs shall take account of the following:

a) Swelling and shrinkage movements of reactive clay soils due to moisture changes.

b) Settlement of compressible soils or fills

c) Distribution to the subgrade of the applied loads

d) Tolerance of the superstructure to movement

The standard sets out the requirements for:

a) the classification of a site; and

b) the design and construction of a footing system, including slab supported on the ground, strip and pad footings or a piled or piered system, which supports a masonry or framed one or two storey house, extension or outbuilding.

The sections of the standard relevant to site classification are included in Appendix IV.

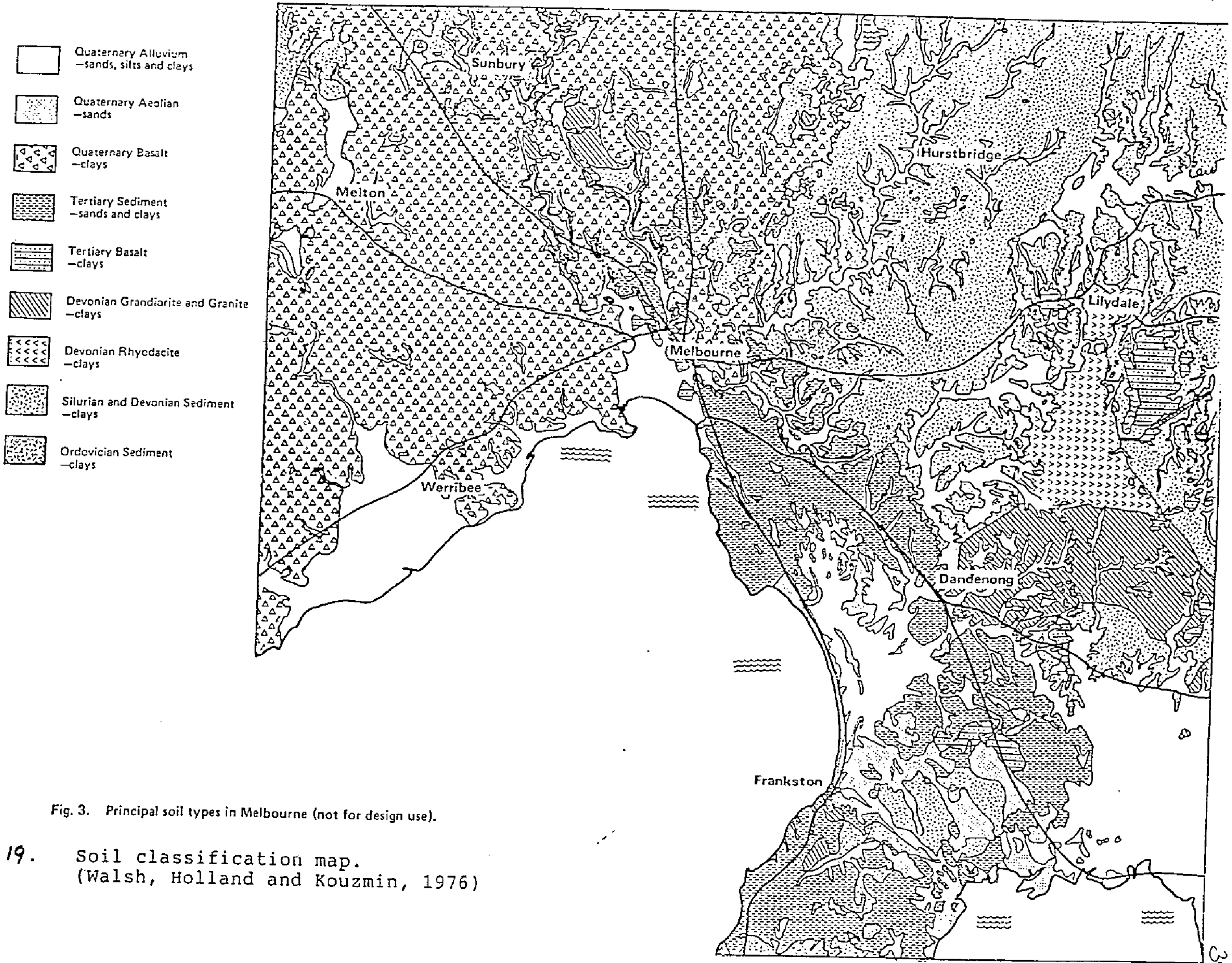


Fig. 3. Principal soil types in Melbourne (not for design use).

Figure 19. Soil classification map.
(Walsh, Holland and Kouzmin, 1976)

Geological description of soil	Typical soil profile: Approximate depth (mm)	Typical soil profile: Description	Classification: Strip & Stump Footings	Classification: Slabs or Footing Slabs
Quaternary Alluvium – sand, silts and clays		Werribee Delta		
		Geol. Map 7622, Ref. Opw Grant [7] Ref. 52010–01/2		
	0–100	Red brown clayey silt	Stable	Stable
	100+	Red brown silty clay to clay		
		Carrum Swamp		
		Geol. Map 849 7922, Ref. Qvm, 859 Ref Q5. Grant Ref. 52010–00/3		
	0–300	Dark grey sandy topsoil	Stable	Stable
	300–2000+	Grey and brown sandy clay with occasional gravel layers		
	0–600	Black clay topsoil	Stable	Stable
	600–2000+	Grey and brown sand to clayey sand with very occasional sandy clay layers		
	0–300	Black silty sand or clay topsoil	Stable	Stable
	300–2000+	Brown grey yellow sand, silty sand or clayey sand with occasional sandy clay layers.		
		The following profile is uncommon but may occur near creeks and rivers.		
	0–200	Black clay topsoil	Intermediate	Intermediate
	200–2000+	Grey and brown clay		
		Port Melbourne – South Melbourne Area		
		Geol. Map Qrs		
	0–600	Black stratified silty clay	Stable	Stable
	600+	Sand silt or silty clay		
		Geol. Map Qrp		
	0–400	Dark grey sandy topsoil	Stable	Stable
	400–2000	Grey sand		
Quaternary Aeolian – sands	0–150	Dark grey brown silty sand topsoil	Stable	Stable
	150–300+	Light yellow grey sand		
	300+	Various clays		
	0–2000+	Uniform grey to dark grey sand	Stable	Stable
Quaternary Basalts – clays	0–100	Brown to black clay topsoil		
	100–rock	Brown to black highly plastic clay may contain floaters		
		(a) Deeper clay soils (>1 m) or soils for which local knowledge indicates past problems.	Unstable	Intermediate
		(b) Shallower clay soils or soils for which local knowledge indicates satisfactory past performance.	Intermediate	Intermediate
		(c) Very shallow soils (<200 mm clay) where edge beams of slabs or footings may be founded on rock.	Stable	Stable
	0–150	Brown to light brown silty clay topsoil	Intermediate	Intermediate
	150–rock	Red to red brown clay		
Tertiary Sediments – sands and clays	0–1000+	Deep uniform grey sand	Stable	Stable
	0–200	Grey silty topsoil	Stable	Stable
	200–500	Grey to yellow sand or clayey sand		
	500–rock	Light grey to yellow sandy or silty clay		
	0–300	Dark brown or grey sandy topsoil	Stable	Stable
	300–800	Loose brown or grey sand		
	800+	Brown clay to sandy clay generally becoming more sandy with depth		
	0–300	Brown sandy topsoil	Stable	Stable
	300–1000	Loose brown sand		
	1000+	Medium dense red brown, or brown sand with occasional gravel layers		
	0–600	Black sandy topsoil	Stable	Stable
	600+	Yellow brown and grey sandy clay, becoming more sandy with depth.		
Tertiary Basalts – clays	0–400	Dark grey to reddish clayey topsoil	Intermediate	Intermediate
	400–rock	Dark grey or reddish brown to brown highly plastic clay		
Devonian Granodiorite and Granite – clays	0–200	Grey or brown sandy to silty topsoil	Stable	Stable
	200–600	Grey and brown silty sand to clayey silt		
	600–rock	Mottled red, brown and grey clay		
Devonian Rhyodacite – clays	0–200	Grey to brown silty topsoil	Stable	Stable
	200–700	Grey brown, yellow brown or red brown silty or sandy clay, mottled		
	700–rock	Mottled red brown and grey yellow brown or orange clay generally stiff to very stiff, may contain silt, sand or gravel		
Ordovician, Silurian and Devonian Sediments – clays	0–100	Grey or grey brown silty topsoil	Stable	Stable
	100–400	Grey, grey brown or yellowish silt to silty clay. Soft when wet, hard if dry.		
	400–rock	Mottled yellow grey or reddish brown clay		

* This table may be applied to solid masonry construction if the internal walls are articulated (for example, by the use of full height door openings) otherwise the standard designs may be inadequate.

SOILS CLASSIFICATION FROM GEOLOGICAL ORIGIN AND TYPICAL SOIL PROFILE FOR MASONRY VENEER OR TIMBER CONSTRUCTION

Table 7. Soil classification table.
(Walsh, Holland and Kouzmin, 1976)

Essentially, the site classes are designated as follows:

Foundation	Character	Class
Sand and rock	Stable	A
Silt and some clay		S
Moderately reactive clay	Reactive	M
Highly reactive clay		H
Extremely reactive clay		E
Sand	Controlled fill	A
Material other than sand		A to P
Mine subsidence	Problem	P
Uncontrolled fill		
Landslip		
Soft		
Collapsing soil		

Table 8. Site Classes (AS2870-1986).

All site classifications for Victoria are based on one or more of the following:

a) Assumption of a soil type without any site investigation from a classification map or from well established local knowledge provided that soils are known to be consistent over large areas. The soil type and site conditions shall be checked by a site visit before construction.

b) Site investigation to identify soil profile using one or more boreholes or test pits in the site or a number distributed over a subdivision.

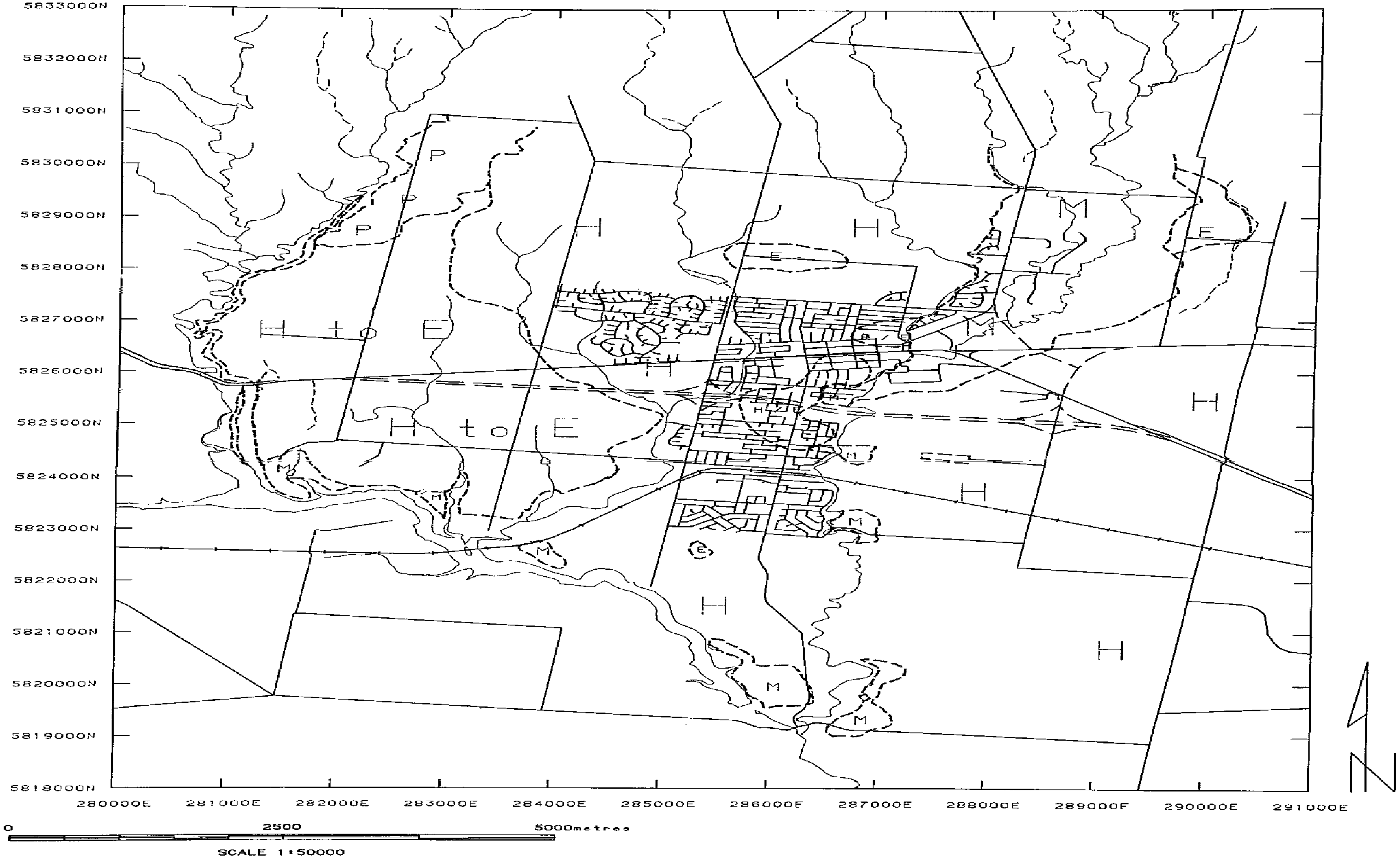
c) Site investigation using a penetrometer, for sand sites.

d) Site investigation including soil sampling and appropriate tests.

Where a Building Authority has designated a presumed site classification or simplified system based on a map of site classifications, this may be used but shall not preclude the adoption of a less severe classification if supported by a site investigation and a classification in accordance with the standard.

This standard is not referred to in the VBR at present, however it is intended to be incorporated if possible (E. Carroll, pers. comm., 1986).

An attempt to map the site classifications referred to in AS2870 is illustrated in Figure 20. This map is derived from a combination of soil depth, swell potential and soil genesis. The site classification method, then, is by soil profile identification (Appendix C, AS2870) rather than by surface movement calculation.



SITE CLASSIFICATION –
AUSTRALIAN STANDARD AS2870

Foundation	Character	Class
Moderately reactive clay	Reactive	M
Highly reactive clay		H
Extremely reactive clay		E
Subsidence area or uncontrolled fill	Problem	P

ENGINEERING GEOLOGICAL MAPPING
OF MELTON, VICT.

Figure 20.

SITE CLASSIFICATION

PROJ NO.	GE 14	DATE:	5-JAN-88

4.2.1 Class P (problem) sites

The area described in Unpub. Rept. 1986/4 - the 'sinkhole plain' - is the largest of the problem sites identified within the Melton development area. The most obvious problem for development of this site is the active subsidence. Roads, housing, and sewerage systems would be seriously damaged by sinkhole development. The unpredictability of the sinkholes makes the siting of any structures hazardous. It is likely however, that areas of the site could be found where the potential for subsidence would be lower.

The soil subsidence occurring in this area would classify the site as a class P (problem) site according to AS2870. The presence of deep basaltic clay classifies the site as "unstable" for strip and stump footings and "intermediate" for slabs or footing slabs according to the Victoria Building Regulations (VBR). The regulations set out the minimum dimensions for either type of footing, which are not condidered adequate for this site. The depth of highly expansive clay and the potential for sinkhole development create unusual foundation conditions which require special engineering design for footings.

Buildings founded on expansive soil need careful attention paid to building design and maintenance, in order to mitigate or control structural damage. Properly engineered foundations, segmented interior design, flexible connections to utility lines, and carefully designed lot drainage and landscaping are required for satisfactory building performance.

Selection of building sites in areas where the soil is thinnest and removal of all trees surrounding buildings would lessen the risk of subsidence occurring, although not entirely rule it out. Placement of the footings on the rock (by designing pier and beam footings) would ensure that the building would not subside, even though the soil may. Chen (1975) warns that pier and beam design does not always work in expansive soils, since the swelling and shrinking can produce considerable lateral and frictional forces on the piers.

An alternative solution would be to replace the foundation soils with non-swelling granular soils. Chen (1975) suggests at least 1.5m under the footings and 3m beyond the building line Soil replacement will lessen the chances of building distress considerably since it would overcome the effects of the expansive soils and cushion the effect of any subsidence. The possibility of subsidence occuring still remains, although the effects would be less dramatic on the surface due to the compensatory movement of the granular soil.

Other problem sites identified are very small areas where farm dams seen on aerial photographs taken in 1943 and 1967 have been infilled.

4.3 Sewerage

An investigation of the usefulness and limitations of various methods of treating or disposing of domestic waste waters was carried out by the Environment Protection Authority (EPA) and reported on in 1975. The report found that reticulated sewerage is the only really satisfactory method of dealing with domestic waste water discharges on long-term basis. However, in non-sewered areas several alternatives were available for waste disposal (Table 9). For environmental reasons only the "all-waste" treatment systems are considered suitable, as the previous practice of sullage disposal by direct discharge to creeks or drains is no longer acceptable.

Of the all-waste disposal units the septic tank with soil absorption has some specific geological requirement. Although in principle, surface irrigation with effluent is possible, normal domestic waste disposal do not use this method for both aesthetic and health reasons. Nearly all septic tanks using soil absorption of effluent use absorption trenches. In the ground absorption process, the soil factors which determine the rate of absorption of water are:

- infiltrative capacity of the liquid soil interface
- percolative capacity of the soil itself
- effective soil particle size
- trench loading

The first two factors determine the rate at which liquid enters the soil and can percolate away, and therefore play the major part in absorption process. Treated effluent however is considerably different in composition to pure water, and for the case where a septic tank effluent is being applied the infiltration rate is always less than the percolation rate due to clogging of the interface with suspended matter and biological growths, as well as a swelling of hydrated soil particles and deflocculation by added sodium or potassium ions. (EPA, 1975a)

Measurement of infiltrative capacity is difficult and it is usual to measure percolation rates or soil permeability (hydraulic conductivity) instead. Both the Victorian Health Commission and the EPA have used percolation tests to assess infiltrative capacity. The EPA has developed a standard test to measure the percolation rate of soils in relation to septic effluent absorption (EPA, 1975b). This procedure is based on that of the U.S. Public Health Service, and simply entails excavating a standard hole in the ground, soaking the soil in the hole for a minimum time, and then measuring the percolation rate of the soil as a drop in water level in the hole over a standard time.

Research of three methods of measuring soil permeability (Winneberger, 1974) shows poor reproducibility when using the percolation test method. Past experience in performing many of these tests has led to a disregard for the usefulness of the test. The results vary markedly according to the soil fabric, season and site specific location.

WC only

* Septic tank plus: soil absorption
  sand filter
  transpiration bed
  chlorinated discharge

* Incenerator systems

* Humus toilets

* Storage in holding tank - periodic removal

* Chemical stabilisation - storage and removal

* Cesspits

* Pan service

All Waste

* Septic tank plus: soil absorption
  sand filter
  transpiration bed
  lagooning
  chlorinated discharge

* Small treatment plant plus: chlorinated discharge
  lagooning
  transpiration bed
  soil absorption
  sand filter

Sullage Only

* Septic tank plus: soil absorption

* Soakage pit

Table 9. Current domestic waste water disposal methods.

The soil requirements for absorption of septic tank effluent are as follows:

- moderate to high permeability (above 10-5 cm/sec)
- percolation rate greater than 2.5 cm/hr.
- low clay content
- low shrink/swell potential
- not subject to flooding
- no shallow impermeable horizons
- ground water-table at least 1m below the trench bottom (EPA ,1975c)

For the development area, it is intended that most of the dwellings will be conected to a reticulated sewerage system. However, in the low density areas, some septic tank systems may be installed. For these reasons no regional assessment has been made of suitability of effluent absorption. In general, clay soils are not very good because of their expansive nature and very low permeability.

The area in the north west corner (the 'sinkhole plain') presents a difficulty for sewage disposal. Because the site is isolated from the Melton City by a proposed regional cemetery, is was not intended by the ULA to service the site with reticulated sewerage.

Alternative sewage disposal would be limited to above ground methods (e.g. composting, chemical or incinerating toilets, "grey water" irrigation, etc.), since the permeability of the clay is too low to provide adequate effluent absorption. Even in areas where septic system absorption lines could be located in the gravel/sand/silt layer, the localised addition of moisture to the underlying expansive clay would cause excessive swell and distress in the sewerage system. Similarly, sand filters or other in-ground disposal would be ultimately unsatisfactory.

4.4 Roadmaking

In Victoria the common practice in roadmaking has been to use flexible pavements for most highway and suburban roads. The RCA have published guidelines for the design of flexible pavements which are used by road engineers in Victoria (CRB, 1980a).

In the process of arriving at a pavement thickness and composition it is necessary to consider many factors. These may be classified into five broad categories:

(i) Function of the road
(ii) Traffic loading
(iii) Subgrade conditions
(iv) Properties of the available pavement materials
(v) Drainage conditions

The conditions of the subgrade is the most relevant to the engineering geological mapping in the Melton development area. In general, the support provided by the subgrade is the most important factor in determining pavement design thickness, composition and performance (CRB, 1980a). The subgrade should be prepared and compacted so that its long term bearing strength is as uniform and as high as possible. In situ strengths during construction may differ greatly from the strengths ultimately developed at the equilibrium moisture content.

The long term strength of the subgrade is governed by:

- the type of material
- the subgrade moisture regime
- the sensitivity of the subgrade to moisture changes

Of these factors, the last is of particular importance at Melton. The extensive expansive clay soils have the potential to appreciably change volume with changes in moisture. This swelling or shrinkage of expansive clays is rarely uniform and the resulting distortion can severely damage an otherwise sound pavement.

Volumetric changes can be minimised by:

1. Minimising changes in the moisture content, eg. compacting the subgrade at a moisture content close to the equilibrium moisture content, and maintaining it at this level until covered by the pavement.

2. Placing sufficient weight of material over the subgrade to counteract the swelling pressure.

3. Modifying the subgrade to reduce its sensitivity to moisture by the addition of a stabilising agent such as lime.

The RCA have extensively tested the Melton soils for the construction of the Western Freeway Melton By-pass and opted for lime stabilization as a suitable soil treatment. The addition of lime (approx. 4%) to the subgrade material greatly improves the roadmaking properties.

The pavement thickness design procedure described by the RCA is based on an empirical relationship between:

- the strength of the subgrade in terms of its CBR (section 3.11.3), and

- the pavement thickness required over the subgrade to carry the predicted traffic loading at the desired level of performance.

APPENDIX I

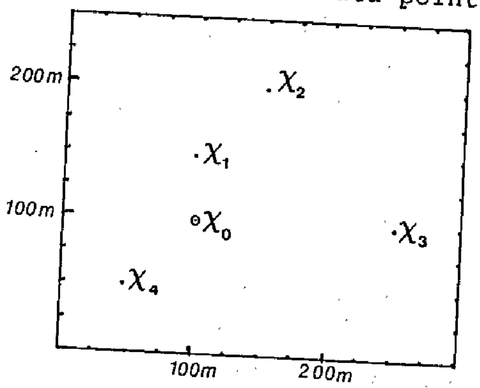
SIMPLE KRIGING EXAMPLE

(Modified from Farrelly, 1985)

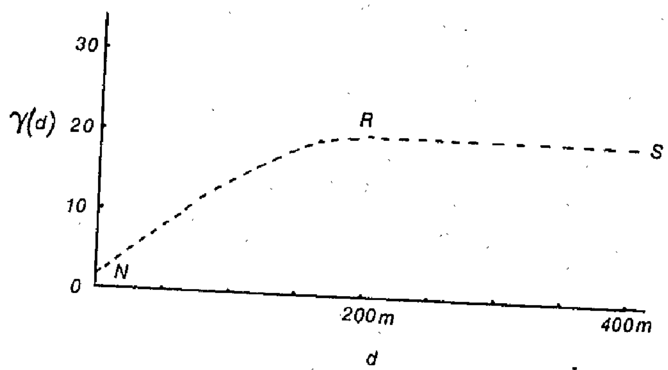
SIMPLE KRIGING EXAMPLE

Assuming isotropic and stationary semi-variances, all that is required to calculate the kriging weights is the semi-variogram model and the arrangement of the data points with respect to the point, area or volume being estimated.

Take the following area, with data points distributed as shown :



Given the semi-variogram for the whole area :



$$\gamma(d) = 18\left[\frac{3}{2}\frac{d}{200} - \frac{1}{2}\frac{d^3}{200^3}\right] + 2 \; ; \qquad \text{for} \quad d \leqslant R$$

$$\gamma(d) = 20 \; ; \quad \text{for} \quad d > R$$

The point X_0 is estimated using the weighted average :

$$\widehat{X}_0 = \lambda_1 X_1 + \lambda_2 X_2 + \lambda_3 X_3 + \lambda_4 X_4$$

where $\lambda_1 + \lambda_2 + \lambda_3 + \lambda_4 = 1$

In matrix form, the solution to the set of kriging equations is written:

$$\begin{bmatrix} \lambda_1 \\ \lambda_1 \\ \lambda_1 \\ \lambda_1 \\ \mu \end{bmatrix} = \begin{bmatrix} \gamma_{1,1} & \gamma_{1,2} & \gamma_{1,3} & \gamma_{1,4} & 1 \\ \gamma_{2,1} & \gamma_{2,2} & \gamma_{2,3} & \gamma_{2,4} & 1 \\ \gamma_{3,1} & \gamma_{3,2} & \gamma_{3,3} & \gamma_{3,4} & 1 \\ \gamma_{4,1} & \gamma_{4,2} & \gamma_{4,3} & \gamma_{4,4} & 1 \\ 1 & 1 & 1 & 1 & 0 \end{bmatrix}^{-1} \times \begin{bmatrix} \gamma_{0,1} \\ \gamma_{0,2} \\ \gamma_{0,3} \\ \gamma_{0,4} \\ 1 \end{bmatrix}$$

where
$$\gamma_{1,1} = \gamma(d_{1,1}) = \gamma(0) = 0$$
$$\gamma_{1,2} = \gamma(d_{1,2}) = \gamma(70.7) = 11.1$$
$$\gamma_{1,3} = \gamma(d_{1,3}) = \gamma(50) = 8.6$$

etc. etc.

The solution to the example is

$$\begin{bmatrix} .51 \\ .03 \\ .09 \\ .37 \\ .87 \end{bmatrix}$$

Thus our estimate is :

$$X_0 = .51X_1 + .03X_2 + .09X_3 + .37X_4 + \epsilon$$

with the variance of ϵ , the kriging variance, being :

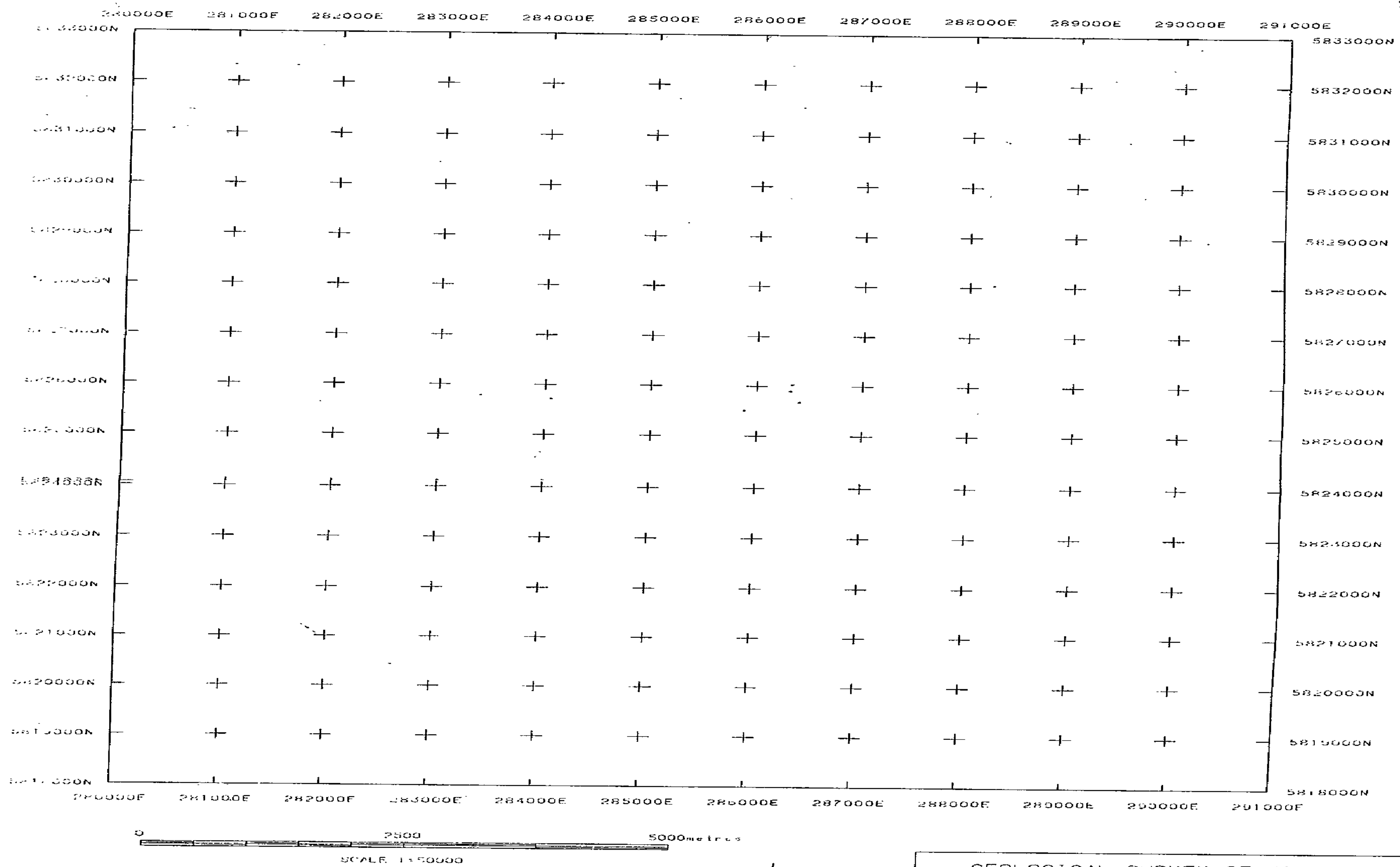
$$\sigma_k^2 = \mu + \sum_{i=1}^{n} \lambda_i \gamma_{i,0} = 11.5$$

This is a measure of estimation error, and the ability to derive such a measure is one of the advantages of kriging. Another advantage is the automatic down-weighting of samples in a direction in which we already have information. This 'screen effect' can be seen in our example where X_2 has a lower weighting than X_3, even though it is closer to X_0.

APPENDIX II

CHECK PLOTS FOR SOIL DEPTH

52



INFORMATION COLLATED FROM:

GEOLOGICAL SURVEY OF VICTORIA
ROAD CONSTRUCTION AUTHORITY •
RAILWAY CONSTRUCTION BOARD •
STATE ELECTRICITY COMMISSION •
PUBLIC WORKS DEPARTMENT •
MELTON SEWERAGE AUTHORITY •
FAWLEY AND LEWERS PTY. LTD. •

• UNPUBLISHED INFORMATION

LEGEND

+ DATA POINT

N

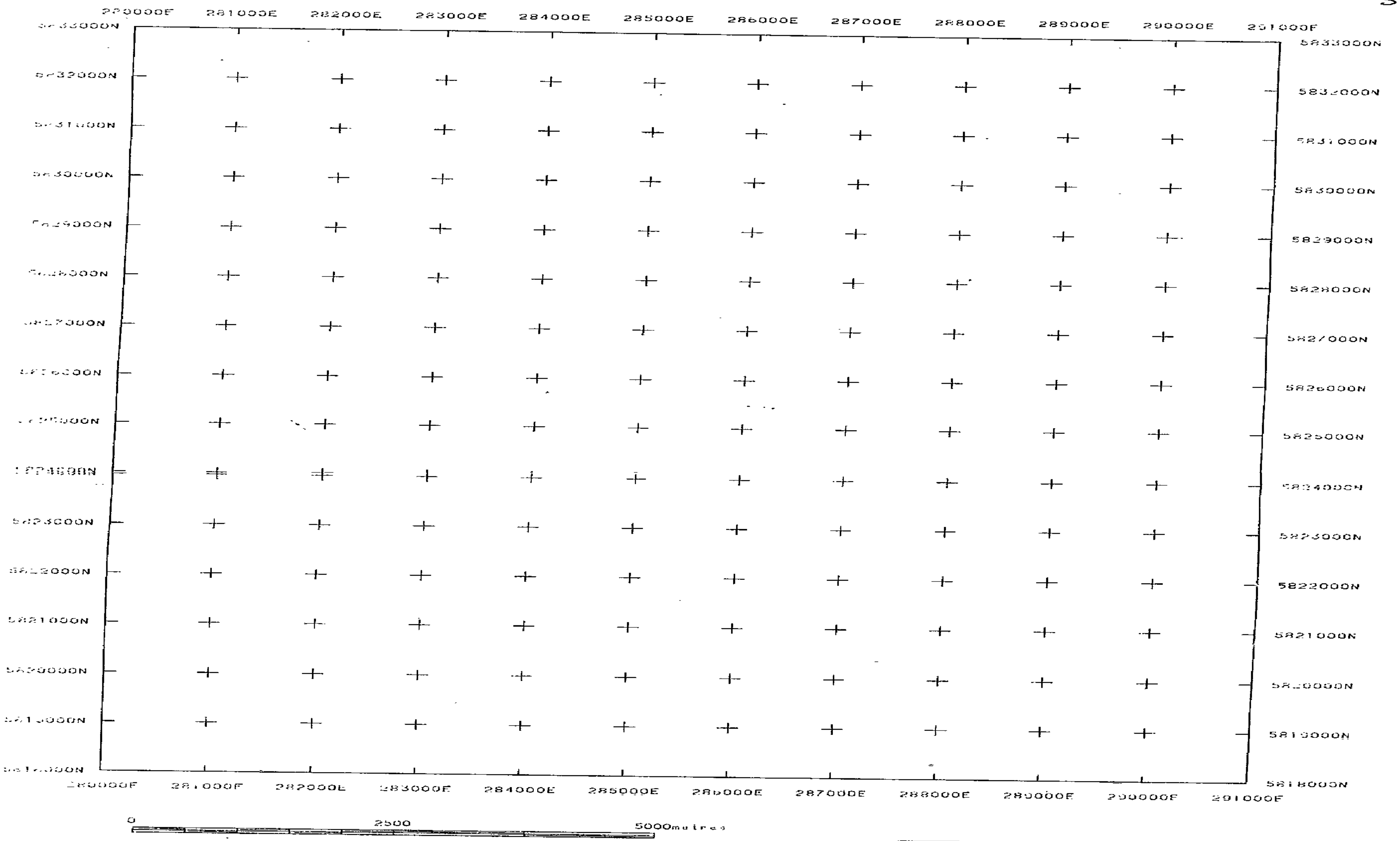
GEOLOGICAL SURVEY OF VICTORIA

ENGINEERING GEOLOGICAL MAPPING OF MELTON, VICTORIA.

ROCK MORE THAN 7m DEEP

PROJ NO.	DATE: 17-SEP-85

S3



SCALE 1:50000

INFORMATION COLLATED FROM:

GEOLOGICAL SURVEY OF VICTORIA
ROAD CONSTRUCTION AUTHORITY *
RAILWAY CONSTRUCTION BOARD *
STATE ELECTRICITY COMMISSION *
PUBLIC WORKS DEPARTMENT *
MELTON SEWERAGE AUTHORITY *
FARLEY AND LEWERS PTY. LTD. *
C.R.A. EXPLORATION LTD. *

* RESTRICTED INFORMATION

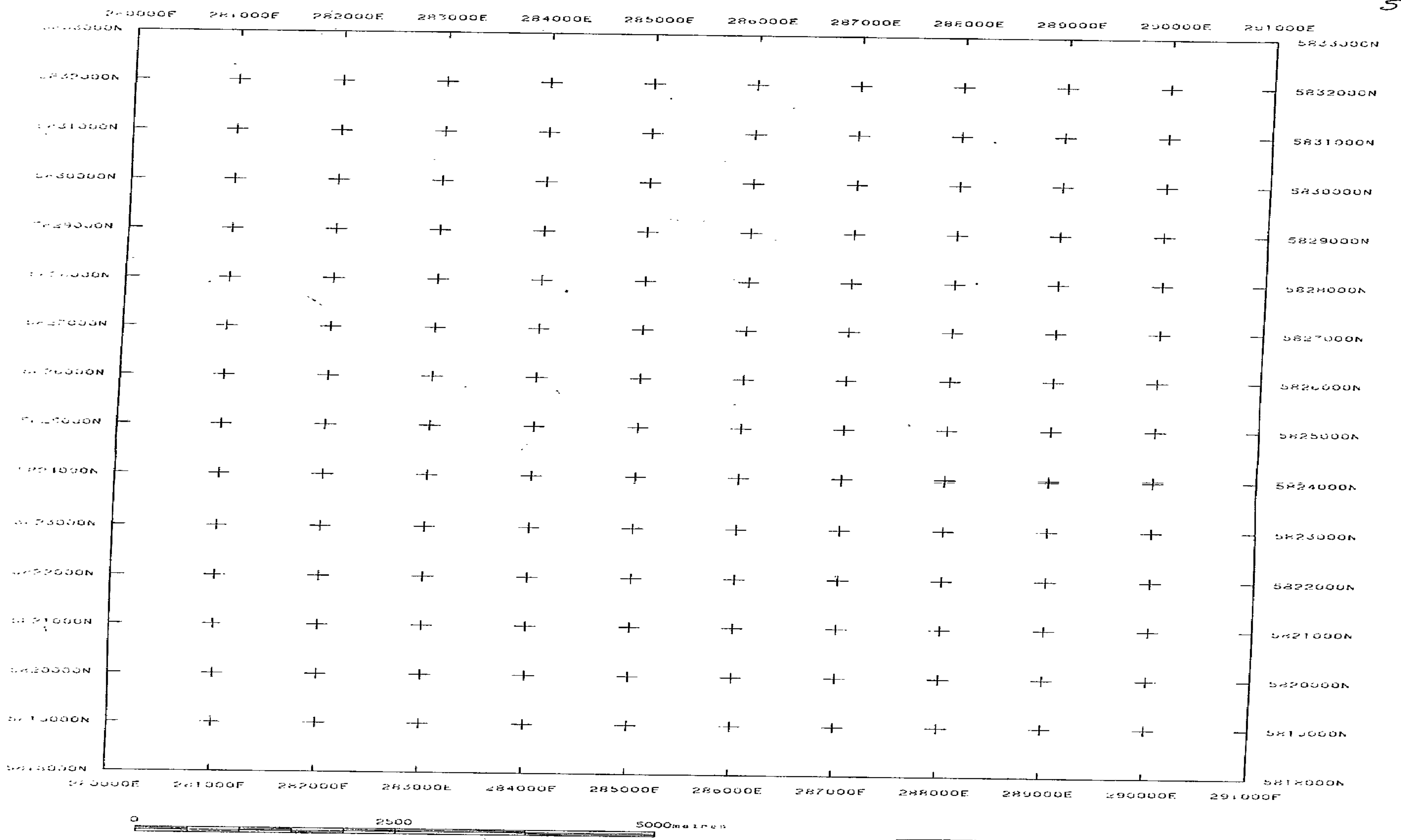
LEGEND

+ DATA POINT

N

GEOLOGICAL SURVEY OF VICTORIA	
ENGINEERING GEOLOGICAL MAPPING OF MELTON, VICTORIA.	
SOIL DEPTH ROCK MORE THAN 6m DEEP	
PROJ NO.	DATE: 17-SEP-85

54



SCALE 1:50000

INFORMATION COLLATED FROM.

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ROAD CONSTRUCTION AUTHORITY •
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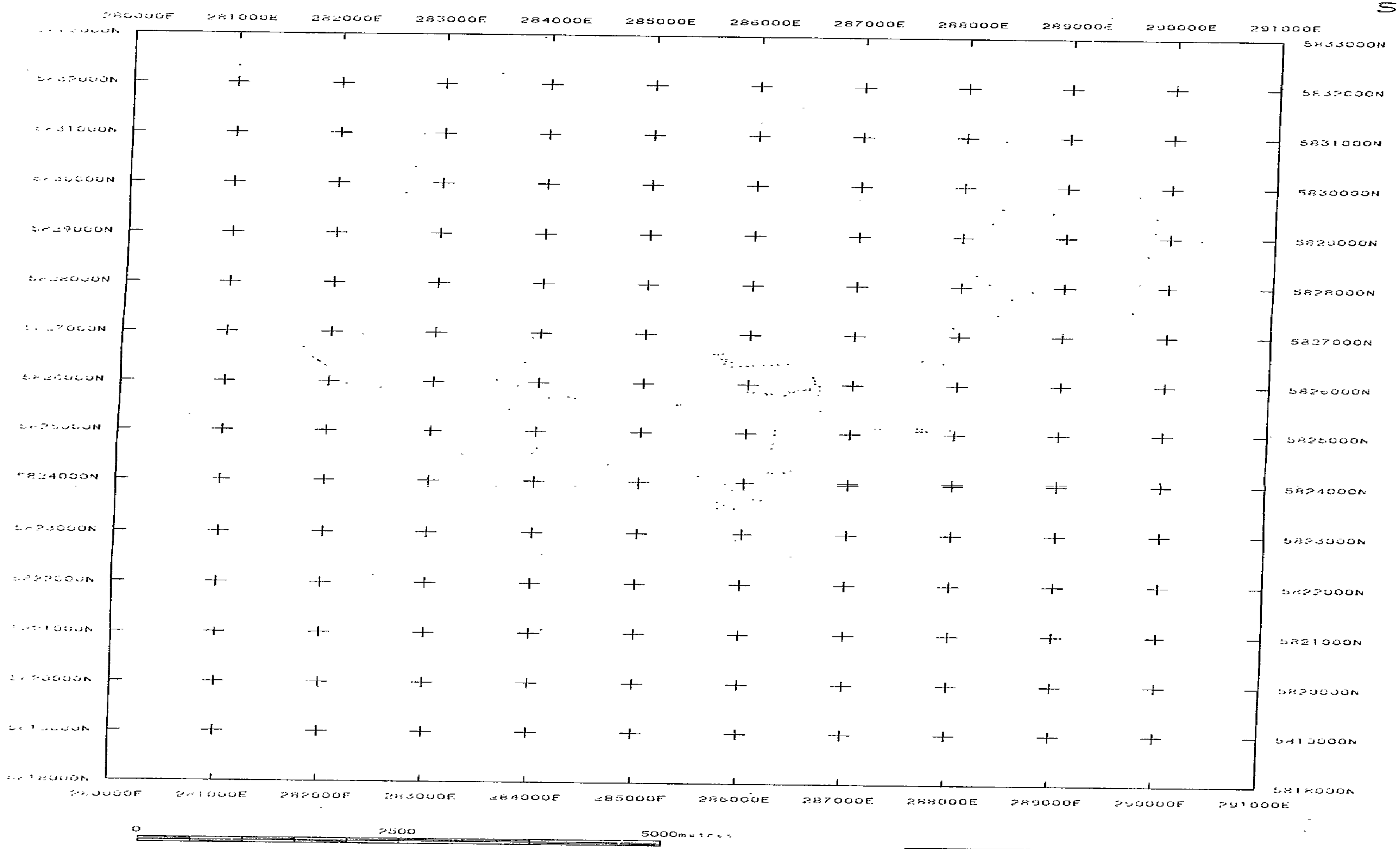
• DATA POINT

GEOLOGICAL SURVEY OF VICTORIA

ENGINEERING GEOLOGICAL MAPPING OF MELTON, VICTORIA.

ROCK MORE THAN 5m DEEP

PROJ NO.	DATE. 17-SEP-85



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ENGINEERINC CEOLOGICAL MAPPING OF MELTON, VICTORIA.

ROCK MORE THAN 4m DEEP

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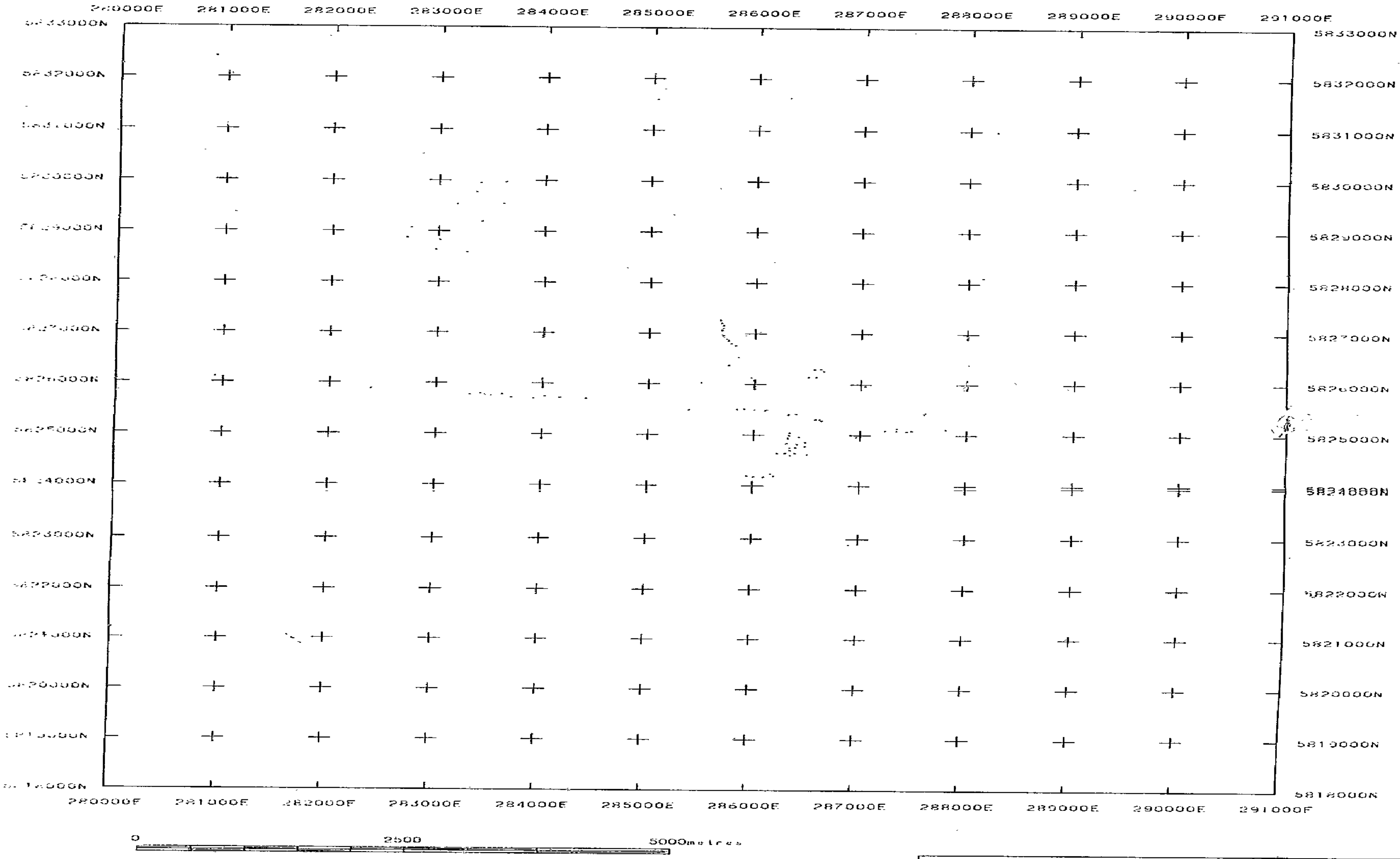

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ROCK MORE THAN 3m DEEP

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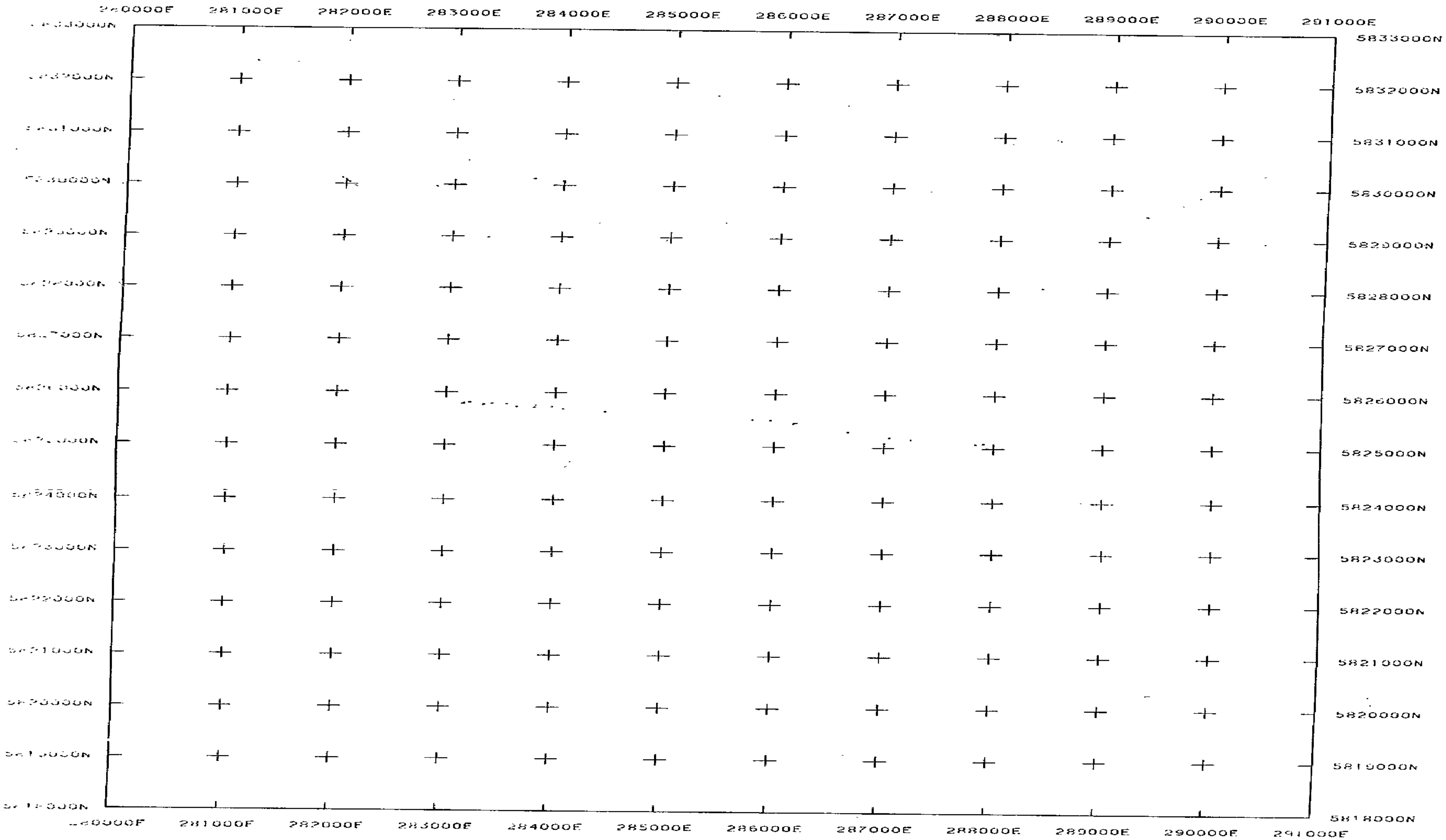

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GEOLOGICAL SURVEY OF VICTORIA

ENGINEERING GEOLOGICAL MAPPING OF MELTON, VICTORIA.

ROCK MORE THAN 2m DEEP

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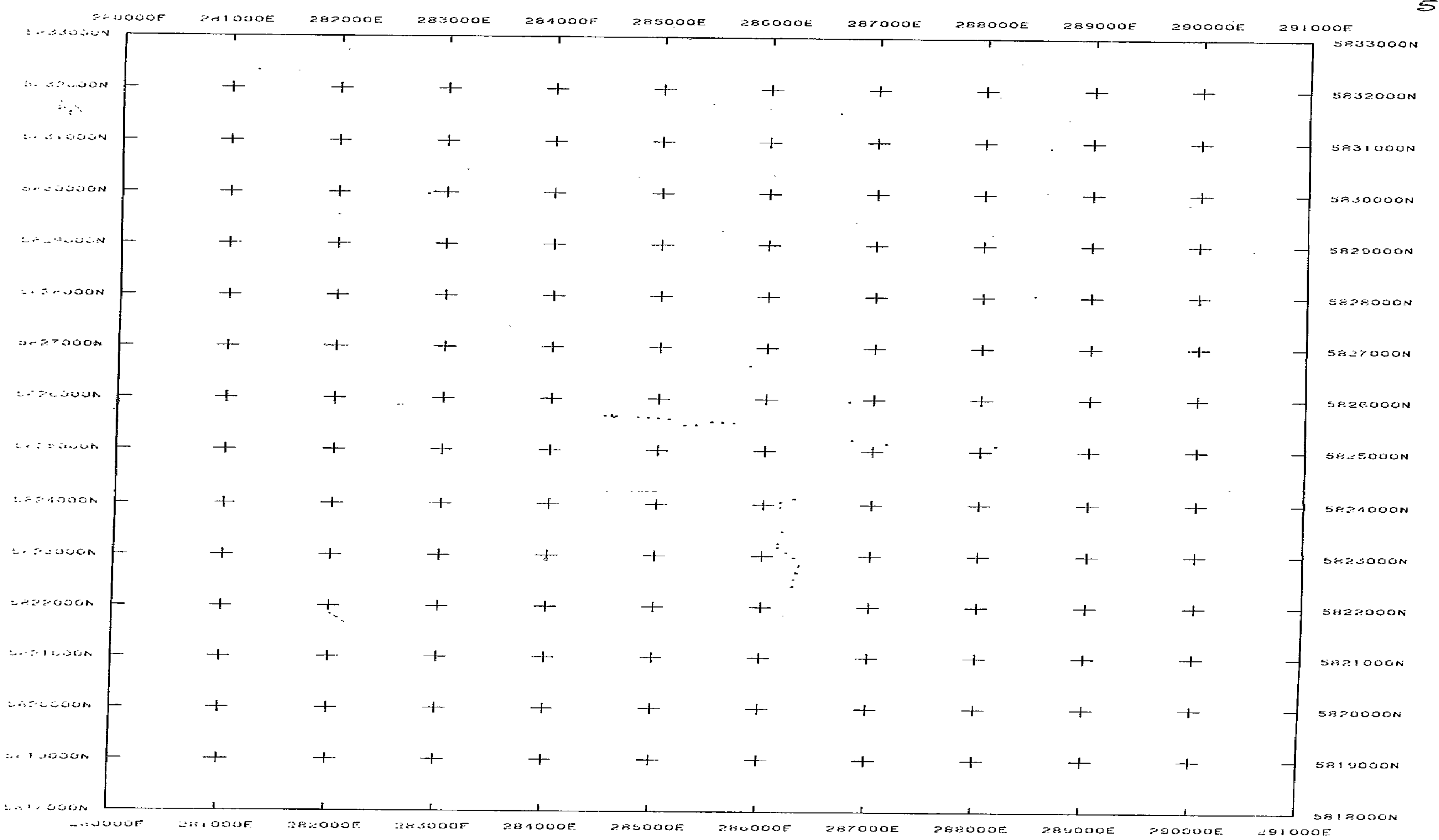

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ENGINEERING GEOLOGICAL MAPPING OF MELTON, VICTORIA.

ROCK MORE THAN 1m DEEP

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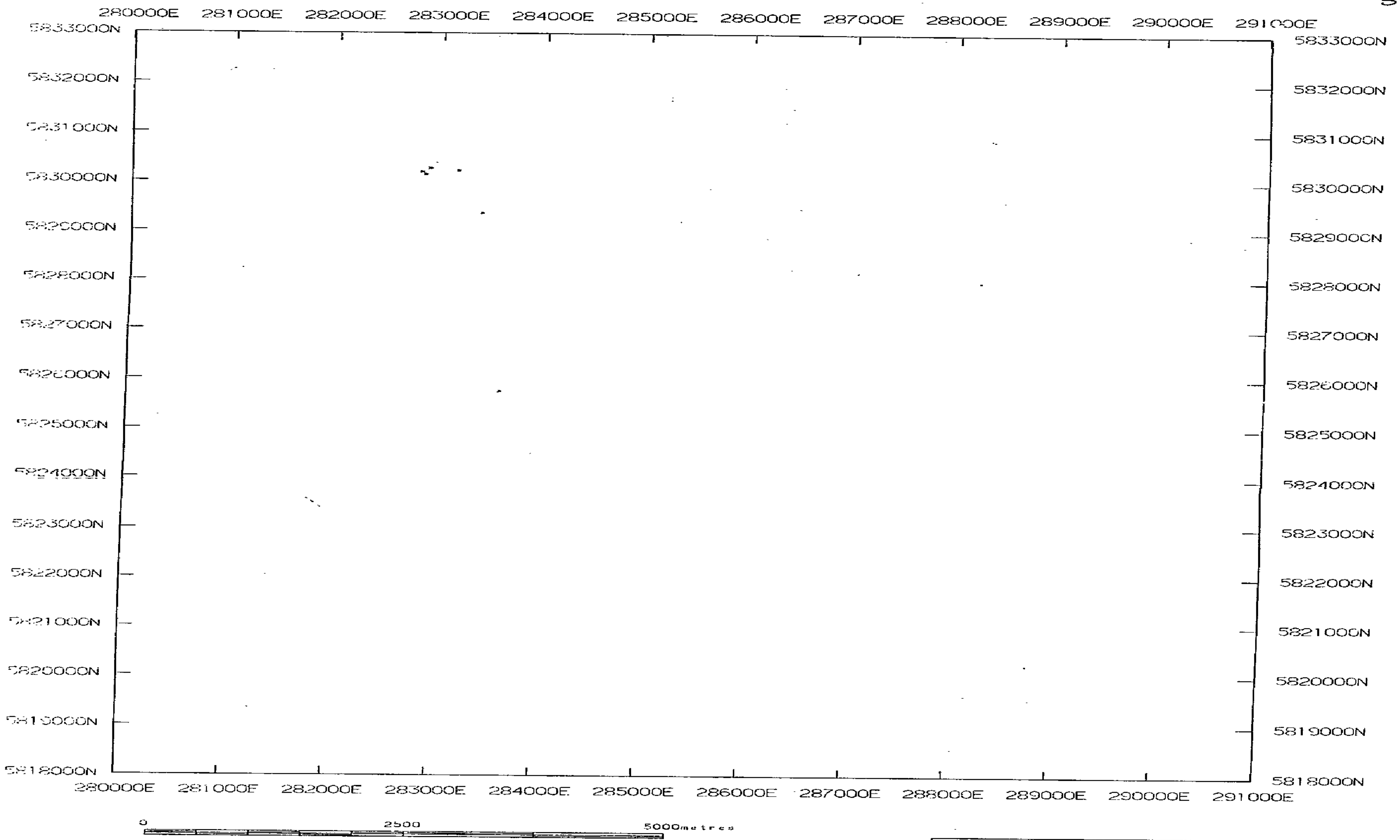

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* SOIL DEPTH DATA POINT WHERE DEPTH IS BETWEEN NINE AND TEN METRES

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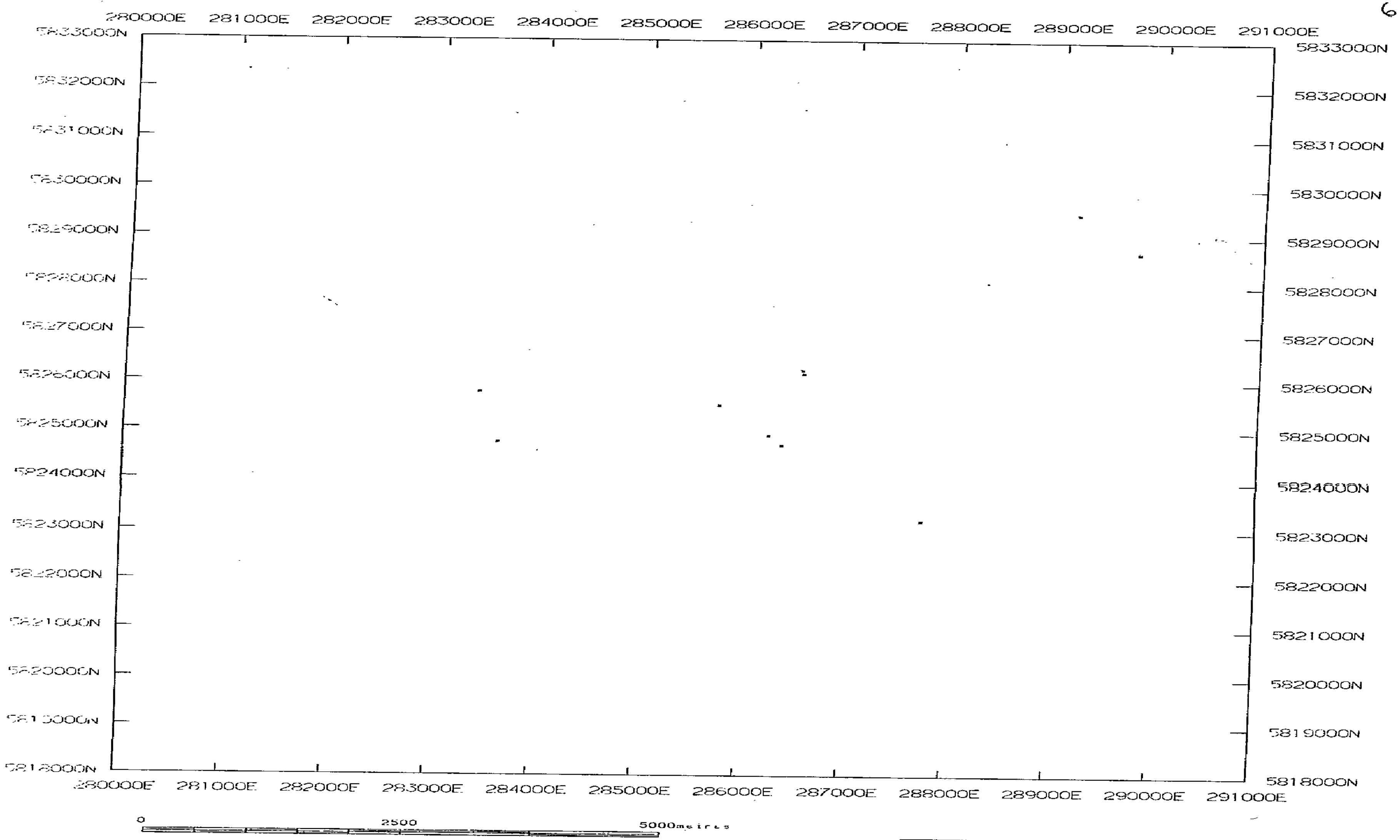
PROJ NO. | DATE: 15-JAN-86



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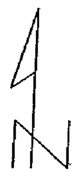
* SOIL DEPTH DATA POINT WHERE DEPTH IS BETWEEN EIGHT AND NINE METRES



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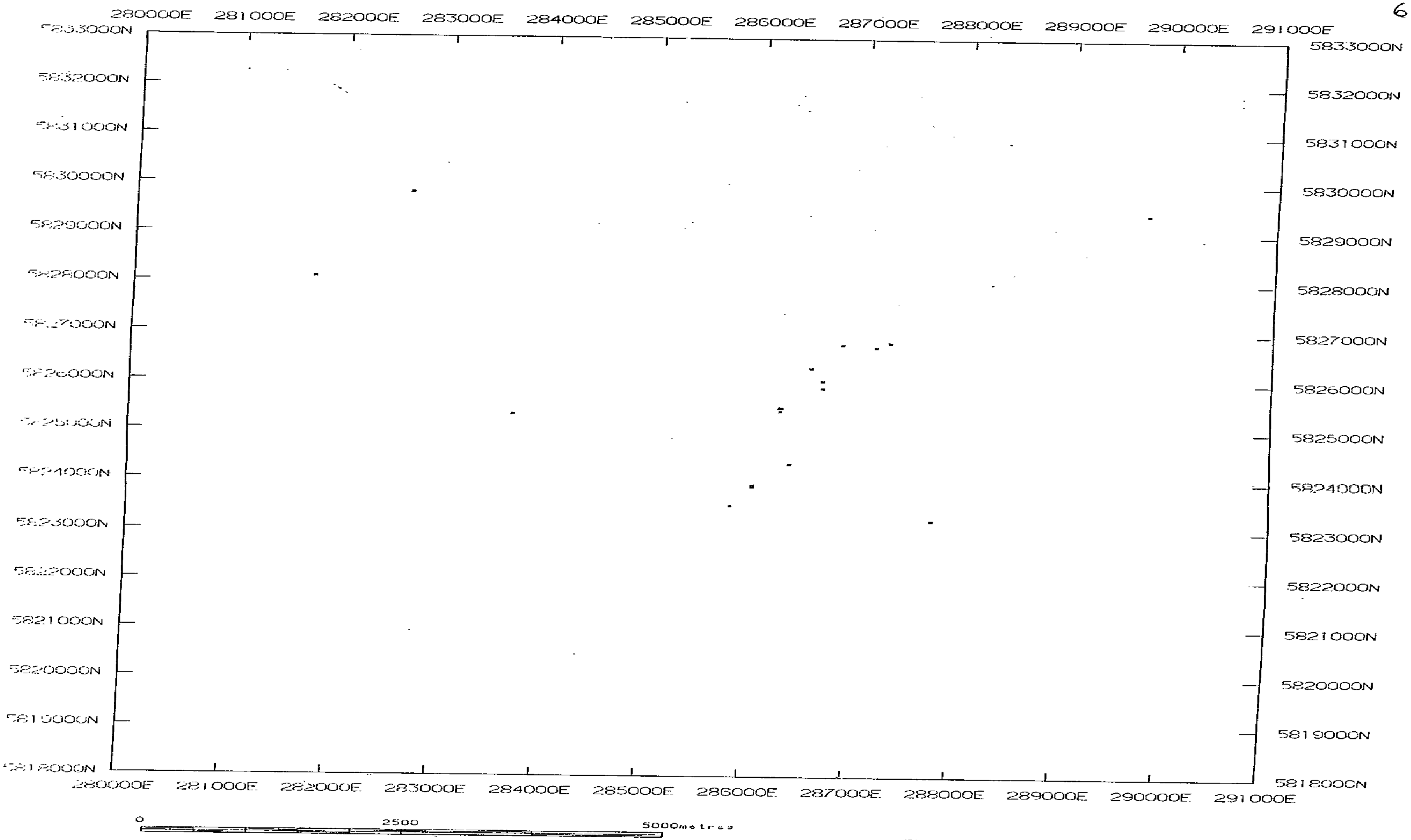

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• SOIL DEPTH DATA POINT WHERE DEPTH IS BETWEEN SEVEN AND EIGHT METRES

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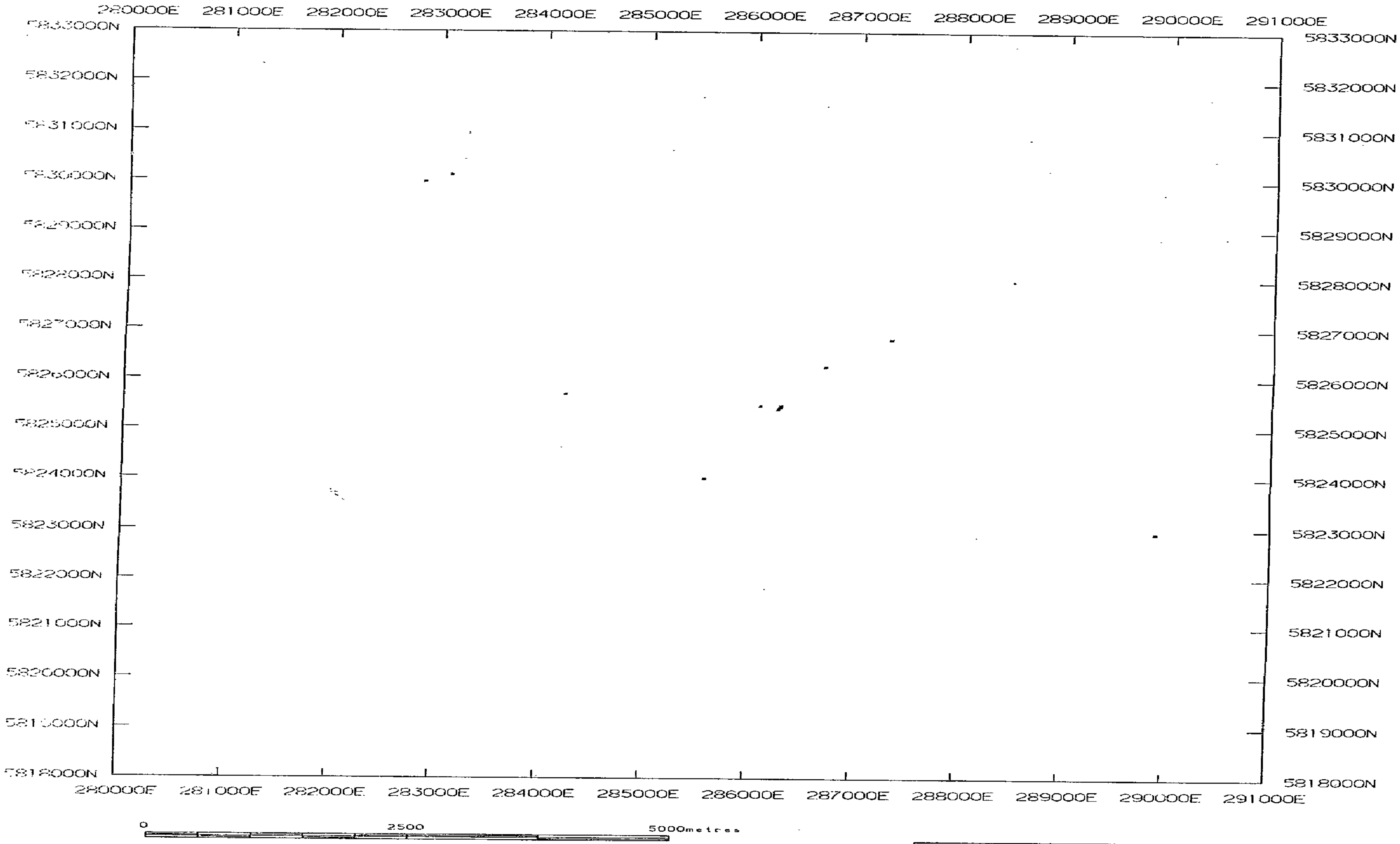
PROJ NO.

DATE: 15-JAN-86



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• SOIL DEPTH DATA POINT WHERE DEPTH IS BETWEEN SIX AND SEVEN METRES

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ENGINEERING GEOLOGICAL MAPPING OF MELTON, VICTORIA.

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DATE: 15-JAN-86

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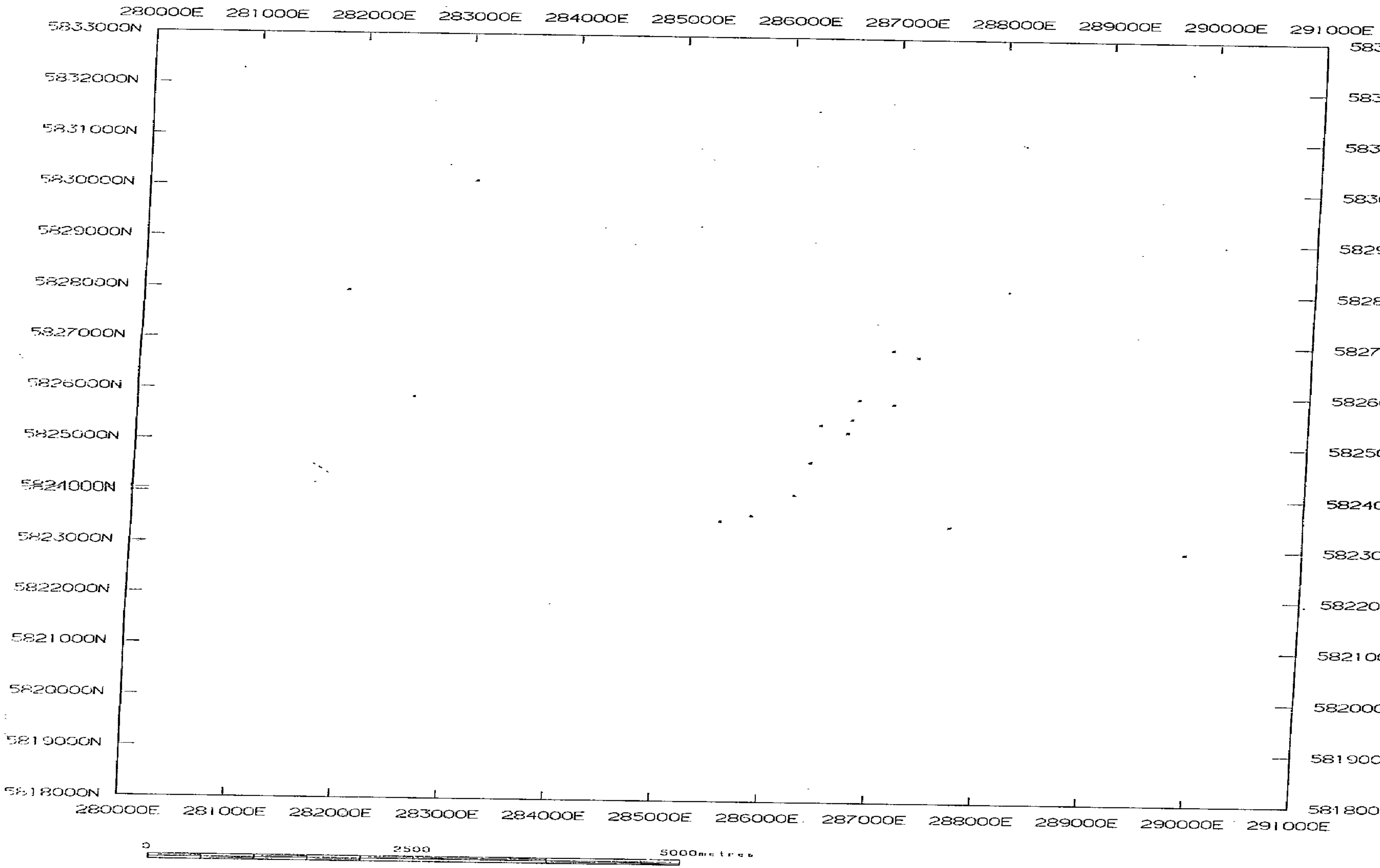

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• SOIL DEPTH DATA POINT WHERE DEPTH IS BETWEEN FIVE AND SIX METRES

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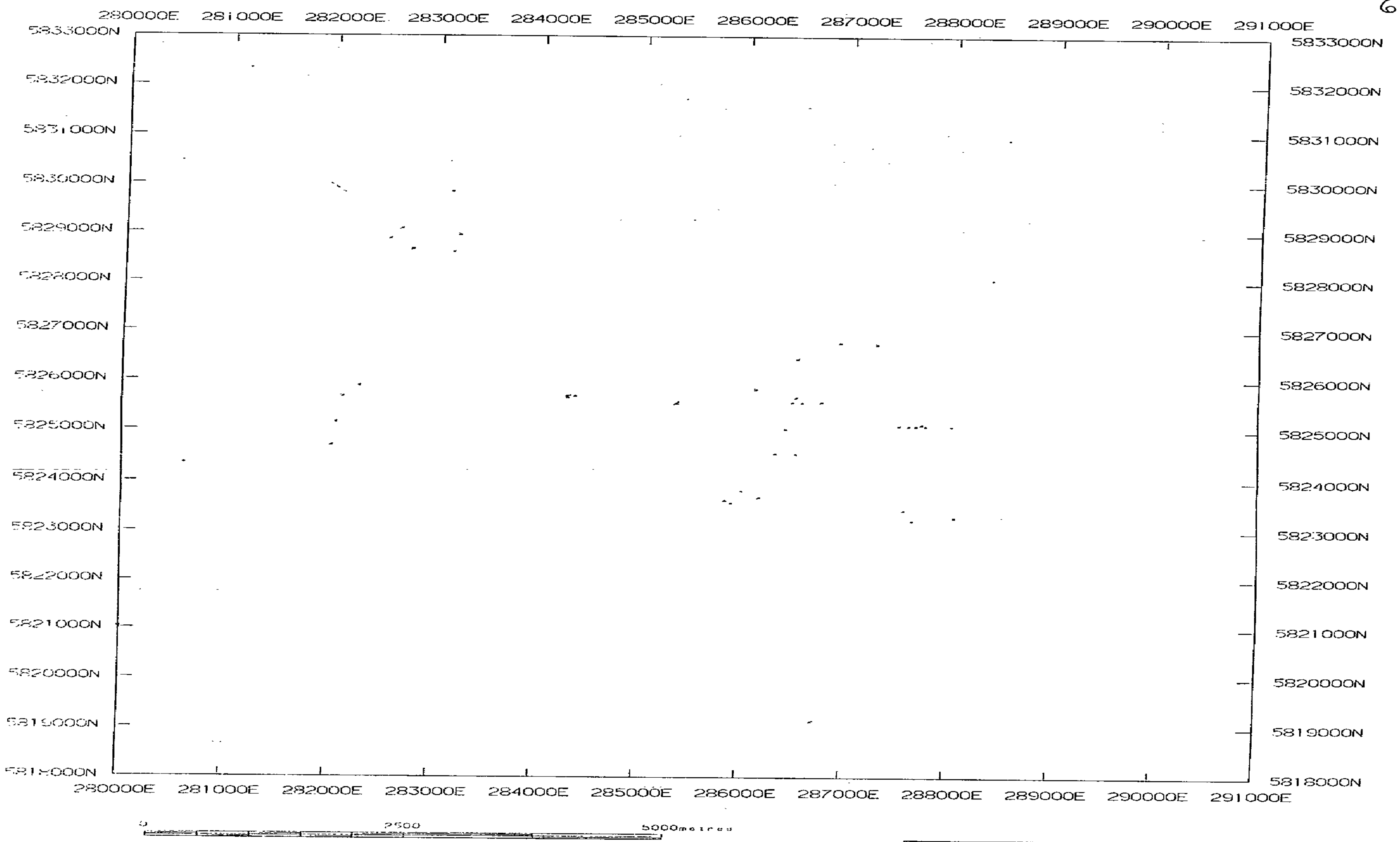
SOIL DEPTH	
PROJ NO.	DATE: 15-JAN-86



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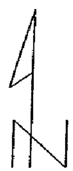
LEGEND

- SOIL DEPTH DATA POINT WHERE DEPTH IS BETWEEN FOUR AND FIVE METRES



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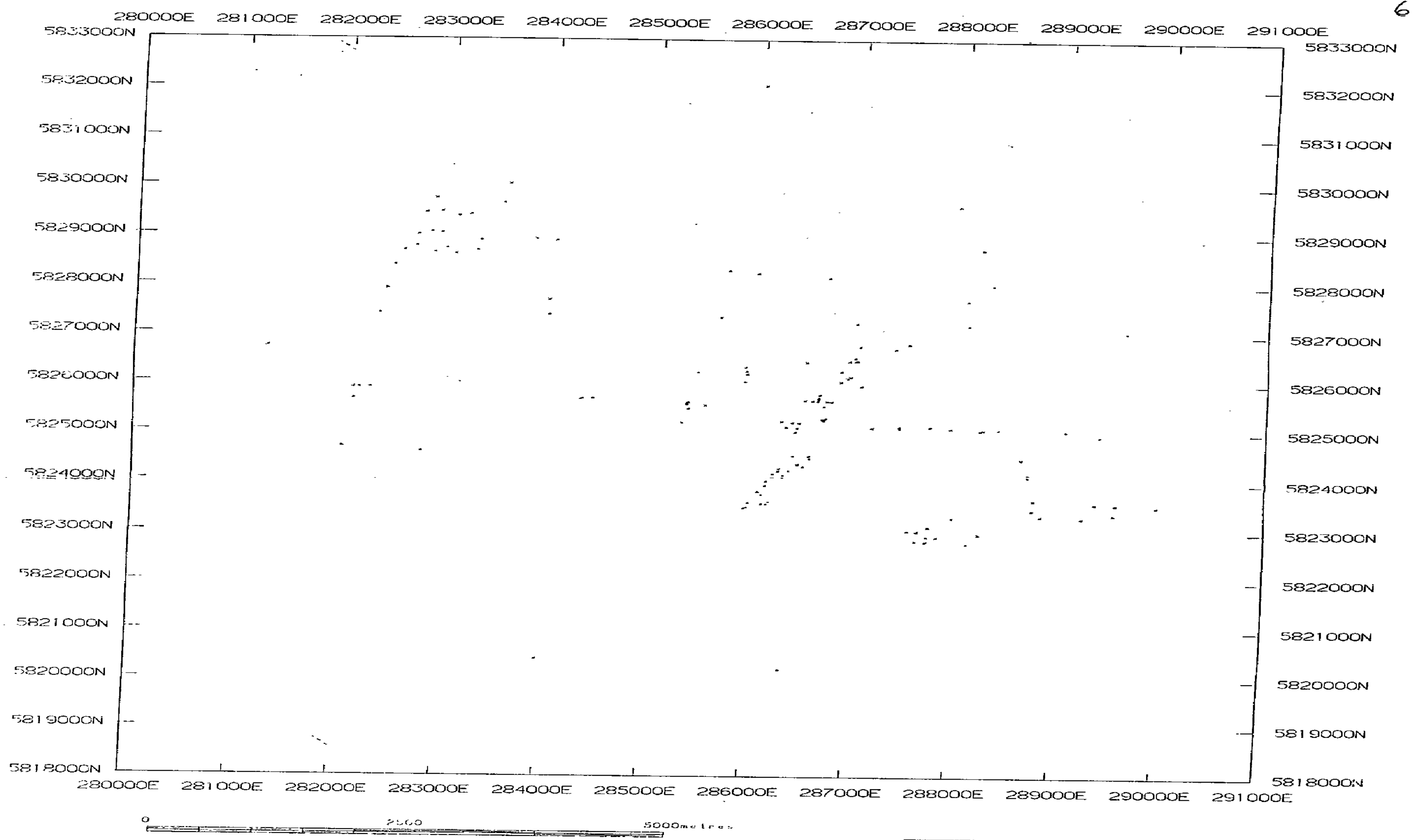

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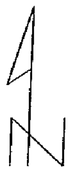
• SOIL DEPTH DATA POINT WHERE DEPTH IS BETWEEN THREE AND FOUR METRES



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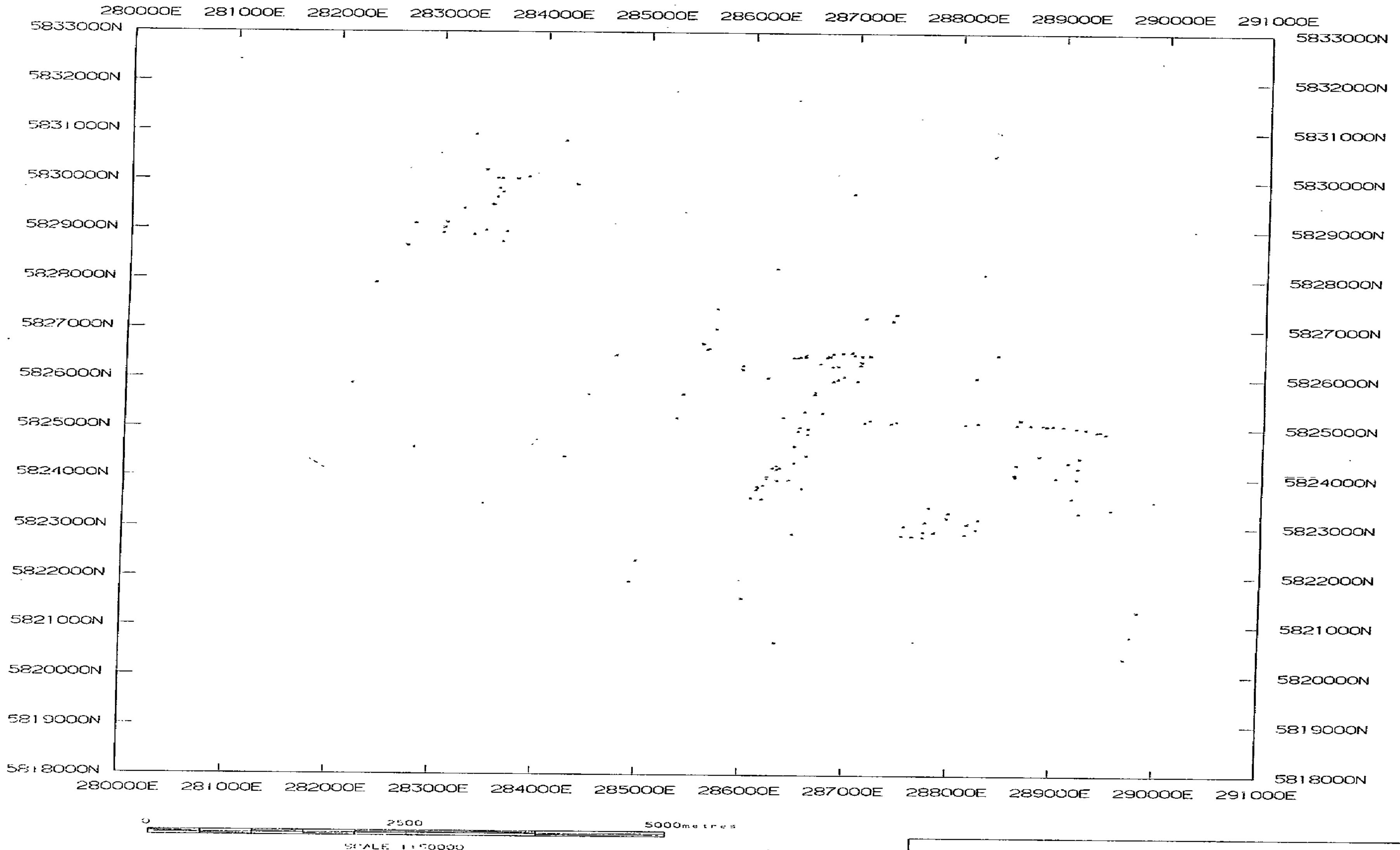

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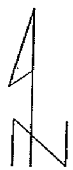
- SOIL DEPTH DATA POINT WHERE DEPTH IS BETWEEN TWO AND THREE METRES



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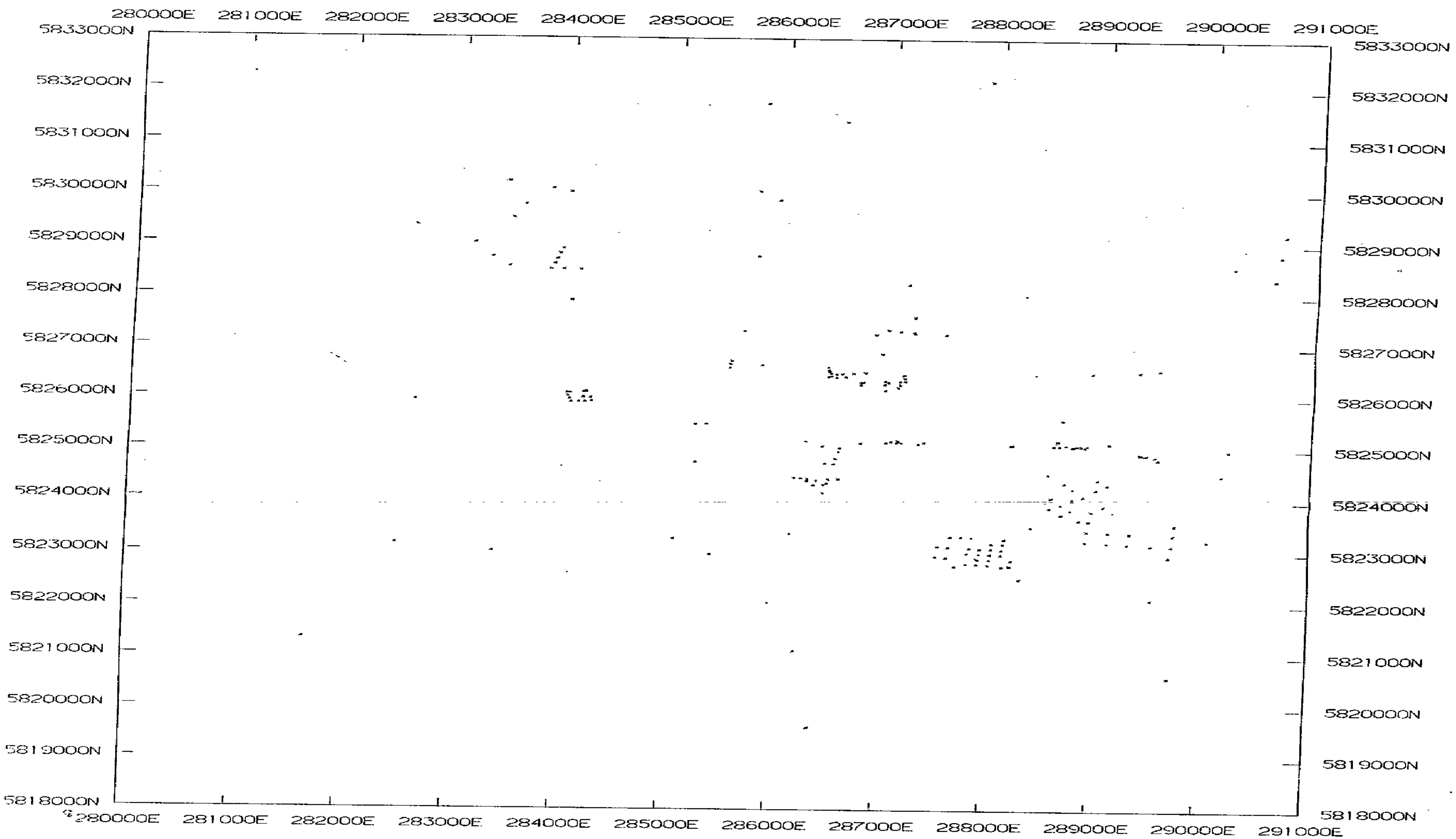

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- SOIL DEPTH DATA POINT WHERE DEPTH IS BETWEEN ONE AND TWO METRES

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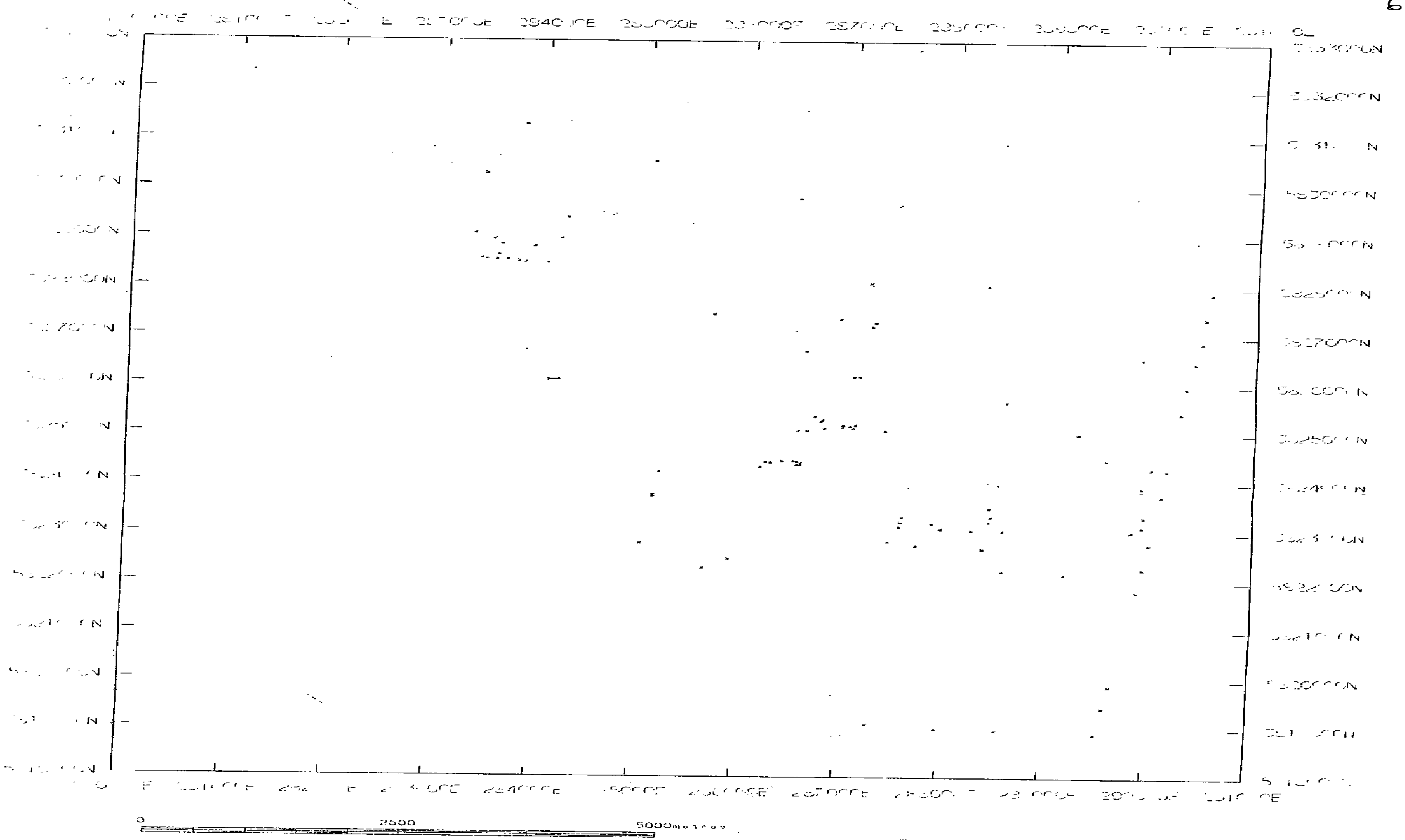

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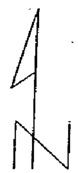

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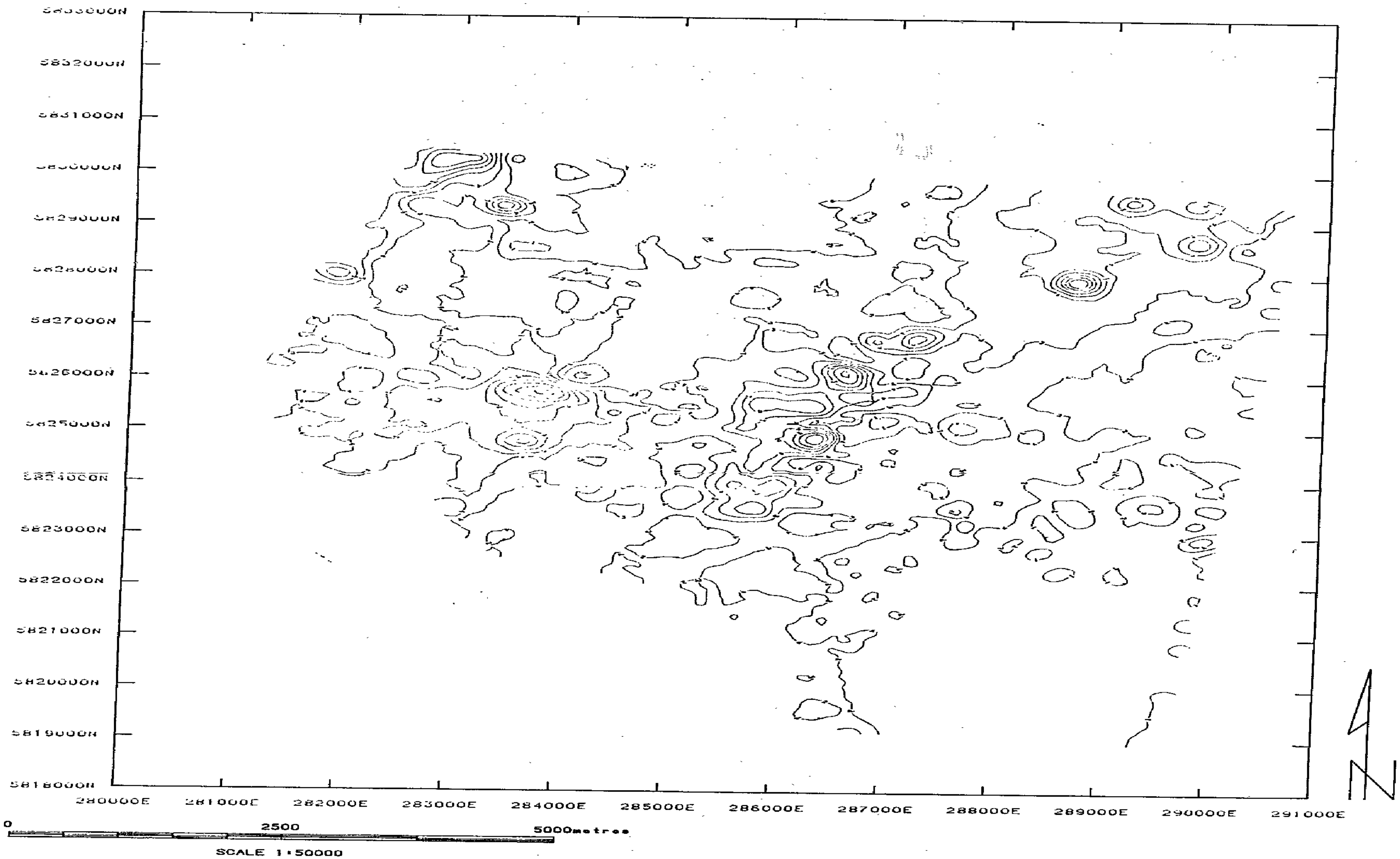

Numeric surface generated
using block kriging -
100 X 100 m. blocks.

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APPENDIX III

BUILDING REGULATIONS FOR FOOTINGS AND FOUNDATIONS EXTRACT FROM VBR (1983)

GROUP VI—STRUCTURAL PROVISIONS

PART 32—FOUNDATIONS

32.1 * * * * *

FOUNDATIONS: ASSESSMENT OF ADEQUACY

32.2 The adequacy of *foundations* shall be based on *approved*—

(*a*) well established and relevant local knowledge and experience of *foundation* conditions in the vicinity of the proposed building; or

(*b*) tests on the *foundation* materials.

32.3 * * * * *

ALLOWABLE BEARING PRESSURES—GENERAL

32.4 The bearing pressure on the *foundation* of a building shall not exceed the values given in Regulation 32.5 unless—

(*a*) an investigation of the *foundations* has been conducted and the *building surveyor* is satisfied in the light of the report on that investigation, that higher bearing pressures are justified; or

(*b*) an investigation of the *site* has been conducted under AS 1726 and the bearing pressures are based on the information obtained from that investigation.

ALLOWABLE BEARING PRESSURES

Application of Regulation

32.5 (1) This Regulation shall only apply where the class and description of the soil or rock adopted for the purposes of this Regulation and the allowable bearing pressures adopted for the purposes of this Regulation are stated on the plans submitted for a *building approval*.

Reference to Tables

(2) The allowable bearing pressures for use pursuant to this Regulation shall be those prescribed in—

(*a*) Table 32.5A;

(*b*) Table 32.5B; or

(*c*) Table 32.5C—

as *required* to be construed in accordance with the Notes to those Tables.

Determination of soil description

(3) In determining an appropriate soil description for use pursuant to this Regulation, the designer shall take account of seasonal moisture conditions.

Pad or strip footings near boundaries

(4) Where any pad or strip footing is on or within 1 m of the boundary of the *allotment* other than a *street alignment*, the allowable bearing pressure shall be two-thirds of the value otherwise prescribed in this Regulation.

TABLE 32.5A

FOOTINGS ON COHESIVE SOIL

Description	*Maximum Allowable Bearing Pressures for Footings at Ground Surface (kPa)*	
	Strip Footings	*Pad Footings (Square or Circular)*
(1)	(2)	(3)
Very soft clay and silt	20	30
Soft clay and silt	40	60
Firm clay	95	110
Stiff clay	180	210
Very stiff clay	350	430
Hard clay	520	650

Notes:

A. Rectangular footings with width to length proportions in the ratio 1 : 5 or greater shall be deemed to be strip footings.

B. For rectangular footings with a width to length ratio between 1 : 1 and 1 : 5 the allowable bearing pressure may be interpolated between those prescribed for strip footings and pad footings.

C. Where a footing is located below ground surface the allowable bearing pressure may be increased by 5 kPa for each 300 mm in distance which the base of the footing is below the ground surface.

D. (1) For the purposes of this Table the following interpretations shall apply:

(*a*) 'Very soft clay and very soft silt' means soil which may be readily penetrated to a depth of 100 mm by the clenched fist.

(*b*) 'Soft clay and soft silt' means soil which may be easily penetrated to a depth of 50 mm by the thumb.

(*c*) 'Firm clay' means soil which may with moderate effort be penetrated to a depth of 50 mm by the thumb.

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(*d*) 'Stiff clay' means soil which may readily be indented by the thumb, but penetrated by the thumb only with great effort.

(*e*) 'Very stiff clay' means soil which may be readily indented by the thumbnail.

(*f*) 'Hard clay' means soil which may be indented by the thumbnail but only with great difficulty.

(2) For the purposes of these interpretations clay shall include silty or sandy clays.

TABLE 32.5B

FOOTINGS ON NON-COHESIVE SOILS

Description	*Allowable Bearing Pressure in kPa for a Footing located at Ground Surface*	*Increase in Allowable Bearing Pressure in kPa for every 300 mm of Depth of Base of Footing Below Ground Surface*	*Maximum Allowable Bearing Pressure in kPa Under any Conditions*
(1)	(2)	(3)	(4)
Loose sand or gravel	50w	15	100
Medium sand or gravel	150w	40	250
Dense sand or gravel	350w	100	550
Very dense sand or gravel	600w	150	700

Notes:

A. For the purpose of this Table, w is the least plan dimension of the footing in metres.

B. If, in the opinion of the building surveyor, the water table is likely to rise to a level the distance of which below the base of the footing is not more than w, the allowable bearing pressure and maximum allowable bearing pressure shall be one half of that otherwise prescribed.

C. For the purposes of this Table, the following interpretations shall apply:

(*a*) 'Loose sand or gravel' means sand deposits readily removable by shovelling only and into which a sharp pointed wooden post 50 mm square can easily be driven with a hammer not exceeding 5 kg.

(*b*) 'Medium sand or gravel' means sand or gravel deposits removable by vigorous shovelling and into which a sharp pointed wooden post 50 mm square can be driven with a hammer not exceeding 5 kg with some difficulty.

(*c*) 'Dense sand or gravel' means sand or gravel deposits requiring picking for removal, and offering high resistance to penetration by excavating tools.

(*d*) 'Very dense sand or gravel' means gravel deposits requiring hard picking for removal, and offering hard resistance to disturbance by excavating tools.

TABLE 32.5C

FOOTINGS ON ROCK

Description	*Maximum Allowable Bearing Pressures for Rock Foundations in Various Conditions of Weathering in kPa*		
	Highly Weathered	*Moderately Weathered*	*Fresh to Slightly Weathered*
(1)	(2)	(3)	(4)
Soft limestone and similar porous rocks	100 to 400	300 to 1000	800 to 1500
Sandstone, mudstone and similar sedimentary rocks	200 to 600	500 to 1500	1200 to 2000
Slate, schist and similar metamorphic rocks	200 to 600	600 to 2000	1500 to 3000
Basalt, granite and similar igneous rocks	200 to 600	500 to 2000	1500 to 4500

Notes:

A. (*a*) The lower end of each range shall be used for rock foundations of the category to which the range applies and which are highly jointed or contain obvious defects.

(*b*) The upper end of each range shall be used for massive and consistent rock foundations of the category to which the range applies.

B. A bearing pressure greater than 600 kPa shall not be imposed by a footing resting on a basalt rock foundation unless a qualified engineer—

(*a*) establishes the condition of the basalt rock foundation to a depth of not less than 1½ times the width or diameter of the footings; and

(*b*) decides in accordance with good engineering practice and the condition of the basalt rock foundation that it is safe to rest the footing on that foundation.

C. For the purposes of this Table:

(*a*) 'Highly Weathered Rock' means rock of predominantly earthy colours (particularly yellows, reds and browns) with numerous clay seams and pieces of which can generally be broken by hand.

(*b*) 'Moderately Weathered Rock' means rock showing some earthy colour predominantly surrounding the joints with some clay seams and pieces of which can generally be broken by hand;

(*c*) 'Fresh to Slightly Weathered Rock' means rock predominantly of a mineral colour or lustre with only minor discolouration adjacent to joints and pieces of which can only be broken with difficulty using hand tools.

D. Where rock is in an extremely weathered form so that, although rock texture and appearance are mainly preserved, the rock substance has the strength properties of soil and may be readily disintegrated by gentle agitation in water, it shall be deemed to be soil for the purposes of this Regulation and classified in accordance with interpretations contained in Tables 32.5A and 32.5B.

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GROUP VI—STRUCTURAL PROVISIONS

PART 33—FOOTINGS NOT ON PILING OR CAISSONS

PROVISION OF FOOTINGS

33.1 Suitable footings shall be provided where necessary to reduce the intensity of the pressure of the building on the *foundations*.

DESIGN OF FOOTINGS

33.2 Footings, including slab-on-ground footings, shall be designed and *constructed* so that any relative movements of separate footings and of different parts of any one footing under loading, or of a footing and any other element of the substructure will not impair the stability of or cause significant structural damage to the superstructure.

DEEMED TO COMPLY

Scope

33.3 (1) This Regulation shall apply to concrete strip or pad footings where—

(*a*) the building does not contain more than 4 *storeys*;

(*b*) the area of any *storey* of the building is not greater than 600 m²; and

(*c*) the bearing pressure exerted by the footing does not exceed the value prescribed in Regulation 32.4.

Construction and proportions of footings designed by prescribed allowable bearing pressure

(2) A footing designed and *constructed* in accordance with this Regulation shall be deemed to comply with Regulation 33.2 if it—

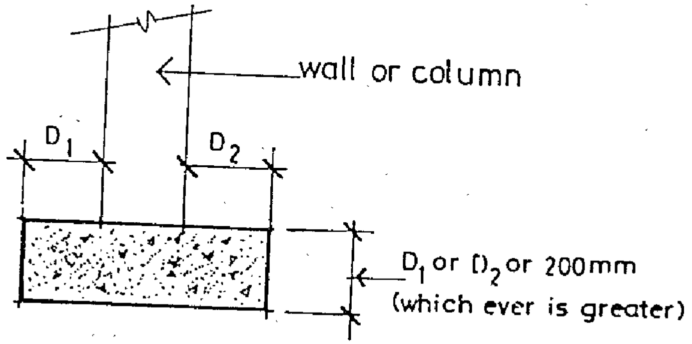
(*a*) is of reinforced concrete *constructed* having a compressive strength at 28 days of not less than 20 MPa, as determined in accordance with AS 1480;

(*b*) has a depth of not less than—

(i) the horizontal projection of the footing at right angles to the face of the wall or the column it supports (as illustrated in Figure 33.3); or

(ii) 200 mm—

whichever is the greater;



Horizontal Projection of footing

FIGURE 33.3

(*c*) in the case of a pad footing, contains not less than 0·15 per cent of the cross-sectional area of the footing, as designed, as reinforcement in each direction near the bottom face of the footing, with a minimum cover of 50 mm;

(*d*) in the case of a strip footing, contains not less than 0·15 per cent of the cross sectional area of the footing as designed as longitudinal reinforcement with a minimum cover of 50 mm, half of which shall be placed in the top third and half in the bottom third of the footing; and

(*e*) has reinforcement in strip footings and pad footings lapped for continuity—

(i) at splices—for a distance of not less than 500 mm;

(ii) at T intersections—for the full width of the layer;

(iii) at corners where fabric strips are used as reinforcement—for the full width of the fabric layer; and

(iv) at corners where bars are used as reinforcement—by a bent lap bar of 500 mm each leg, placed in each layer of reinforcement near the outer face of the corner.

DEEMED TO COMPLY

Application of Regulation

33.4 (1) Any footing of a Class I building or a building of another class which has a uniformly distributed live load not exceeding 3 kPa when calculated in accordance with AS 1170 which is *constructed* in accordance with the relevant provisions of this Regulation and

Regulations. 33.5 to 33.9 shall be deemed to comply with the requirements of Regulation 33.2 except where—

(*a*) by reason of—

(i) the nature of the *foundation*; or

(ii) the design of the building; or

(iii) any other relevant considerations—

the *building surveyor* is of the opinion that any such footing would not be adequate in the particular case; or

(*b*) the building—

(i) contains more than 2 *storeys*; or

(ii) has a wall, which, excluding any gable, exceeds 7·2 m in height; or

(iii) will contain a concrete floor other than a slab-on-ground.

Concrete strength

(2) Concrete used in footings shall have a compressive strength at 28 days of not less than 20 MPa, determined in accordance with AS 1480.

Foundation classification

(3) The classification of the *foundation* of any *site* on which footings are to be *constructed* shall be in accordance with the "Classification of Expansive Behaviour of Melbourne Soils for Domestic Construction" published by CSIRO and AEBIRA.

Areas not covered by publication

(4) In areas outside those shown in Figure 3 of the publication referred to in sub-regulation (3) the *site* on which footings are proposed to be *constructed* shall be similarly classified by adopting, where practicable, the principles stated in that publication and taking into account experience or knowledge of local or traditional building construction practices.

Drawings to include foundation classification

(5) The drawings referred to in Regulation 8.2 (2) shall include the *foundation* classification adopted pursuant to this Regulation and shall be confirmed to the satisfaction of the *building surveyor* on the *site* on which the footings are proposed to be *constructed*.

FOOTINGS FOR STUMPS

General requirements

33.5 (1) Every footing for stumps shall comply with the following:

(*a*) The size of concrete footings for stumps shall be in accordance with AS 1684.

(*b*) The bearing areas of footings for stumps which support a wall sheathed with cement-sand facing tiles, shall be double those prescribed for a timber-framed wall in AS 1684.

(*c*) The footings shall be founded at a depth of—

(i) in the case of sites classified as stable in accordance with Regulation 33.3, not less than 450 mm; or

(ii) in the case of sites classified as intermediate in accordance with Regulation 33.3, not less than 700 mm; or

(iii) in the case of sites classified as unstable in accordance with Regulation 33.3, not less than 1 m.

Concession

(2) Notwithstanding paragraph (*c*) of sub-regulation (1), the *building surveyor* may permit the footings for stumps to be founded at a depth of less than 450 mm if he is satisfied by reason of experience or local knowledge that such a depth would be adequate for the structural stability of the building in the case of—

(i) the re-stumping of or *alterations* to an existing building;

(ii) rock *foundations*; or

(iii) a building in which walls of stud-framed and sheeted construction are supported on stumps.

Excavations

(3) Excavations for footings for stumps shall be—

(*a*) properly backfilled with *approved* material; and

(*b*) compacted in an *approved* manner.

STRIP FOOTINGS

Reinforcement

33.6 (1) Reinforcement in strip footings shall—

(*a*) be equally distributed in two layers, one near the top and one near the bottom of the footing;

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(*b*) have a concrete cover of not less than 50 mm at any part; and

(*c*) be laid continuously, each layer being lapped—

(i) at intersections—for its full width;

(ii) at splices—for not less than 500 mm;

(iii) at corners where fabric strips are used as reinforcement—for the full width of the fabric layer;

(iv) at corners where bars are used as reinforcement—by a bent lap bar of 500 mm each leg, placed in each layer of reinforcement near the outer face of the corner; and

(v) at steppings, as shown in Figure 33.6.

Design generally

(2) Strip footings constructed of concrete shall—

(*a*) have a width and depth in accordance with Table 33.6;

(*b*) if stepping is necessary, comply with one or more of the methods shown in Figure 33.6 and have level bottoms between steppings;

(*c*) be reinforced in accordance with Table 33.6 and Figure 33.6; and

(*d*) be founded on soil or rock having an allowable bearing pressure of not less than 100 kPa.

Reduced footing depth permissible

(3) Where a strip footing designed and *constructed* pursuant to this Regulation is to rest wholly or partly on a floater or rock outcrop, the depth of the strip footing in the vicinity of the floater or rock outcrop may, subject to sub-regulation (4) be reduced to not less than two-thirds of the depth otherwise prescribed by this Regulation.

Reinforcement in reduced footing depth

(4) Where the depth of a strip footing is reduced pursuant to sub-regulation (3), the reinforcement in the section of the strip footing of reduced depth—

(*a*) shall be double the amount of that prescribed by Table 33.6; and

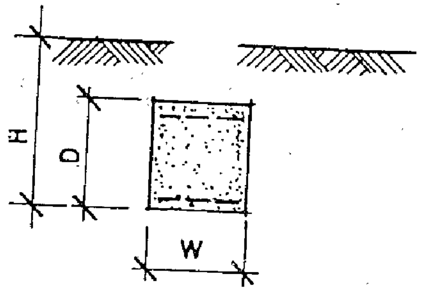
(*b*) shall extend at least 500 mm beyond the section of strip footing of reduced depth.

TABLE 33.6

MINIMUM DIMENSIONS AND REINFORCEMENT FOR STRIP FOOTINGS FOR DIFFERENT TYPES OF CONSTRUCTION

Foundation Classification	*Number of Storeys*	*Footings*		*Excavated Depth to Base of Footings Below Ground Surface*	*Alternative Reinforcement*	
		Width *W (mm)*	*Depth* *D (mm)*	*H (mm)*	*Number of C12, S16 or Y12 Bars or Main Wires of F11TM at Top and Bottom*	*Number of Main Wires of F8TM at Top and Bottom*
(1)	(2)	(3)	(4)	(5)	(6)	(7)
MASONRY VENEER, TIMBER FRAMED, METAL FRAMED OR SINGLE LEAF MASONRY WALLS						
Stable	One	300	375	450	2	3
	Two	375	375	450	3	4
Intermediate	One	300	525	600	3	6*
	Two	375	525	600	3	6*
Unstable	One	300	675	750	3	6*
	Two	375	675	750	3	6*
CAVITY OR DOUBLE LEAF MASONRY WALLS						
Stable	One	350	375	450	2	3
	Two	450	375	450	3	6*
Intermediate	One	350	525	600	3	6*
	Two	450	525	600	4	8*
Unstable	One	350	675	750	3	6*
	Two	450	675	750	4	8*

*Reinforcement is to be provided in two equal layers.



STRIP FOOTING

ILLUSTRATION TO TABLE 33.6

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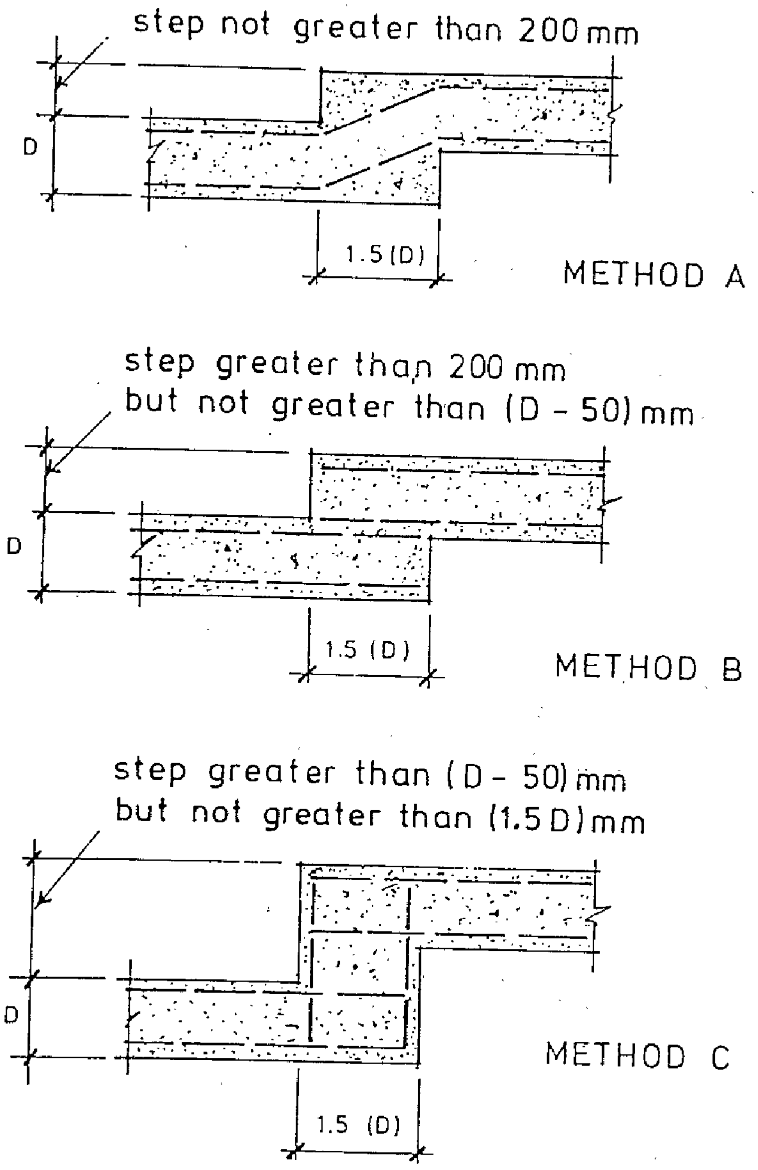

FIGURE 33 6 STEPPED FOOTINGS

SLAB-ON-GROUND

Reinforcement in the slab-on-ground beams

33.7 (1) Reinforcement in the beams in slab-on-ground shall—

(*a*) be placed near the bottom of each beam;

(*b*) have a concrete cover of not less than 50 mm at any part; and

(*c*) be lapped—

(i) at intersections—for its full width;

(ii) at splices—for not less than 500 mm;

(iii) at corners where fabric strips are used as reinforcement—for the full width of the fabric layer;

(iv) at corners where bars are used as reinforcement—by a bent lap bar of 500 mm each leg, placed in each layer of reinforcement near the outer face of the corner; and

(v) at steppings—as shown in Figure 33.6.

Reinforcement fabric in slab-on-ground

(2) Reinforcement fabric in slab-on-ground shall—

(*a*) be placed in the upper half of the slab, with a concrete cover of not less than 25 mm at any part;

(*b*) be lapped for a distance of not less than 225 mm; and

(*c*) be supported by bar chairs at spacings of not more than 1·2 m in either direction.

Requirements generally

(3) Every slab-on-ground shall comply with the following:

(*a*) Top-soil containing significant amounts of organic matter shall be removed from the area on which the slab is to rest.

(*b*) Edge beams of the slab shall be founded on soil or rock having an allowable bearing pressure of not less than 100 kPa.

(*c*) The slab shall be founded on soil or rock having an allowable bearing pressure of not less than 30 kPa.

(*d*) The slab shall be provided with a vapour barrier which shall—

(i) consist of a sheet of polyethylene not less than 0·2 mm in thickness;

(ii) be placed beneath the slab so that the bottom surface of the slab is entirely underlaid;

(iii) be continued around the edge beams to at least ground level or to the bottom of the edge recess, whichever is the lower;

(iv) be lapped at all joints for a distance of not less than 200 mm; and

(v) be taped around pipes which penetrate the slab.

(*e*) The dimensions and reinforcement of the edge beams shall be not less than those prescribed in Table 33.7A and as illustrated in Figure 33.7A.

(*f*) Edge recesses shall be provided for a *masonry* cavity wall or *masonry* veneer construction and shall—

(i) have a depth of not less than 50 mm and any part of the edge beam below any such recess shall have a depth of not less than 150 mm; and

(ii) be *constructed* in the manner illustrated in Figure 33.7A.

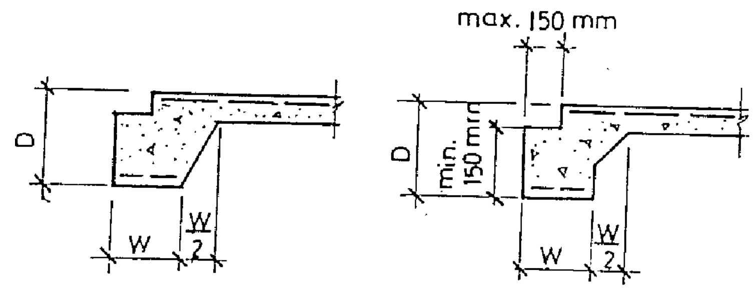

FIGURE 33.7A

TABLE 33.7A
MINIMUM DIMENSIONS AND REINFORCEMENT OF EDGE BEAMS

Foundation Classification and Building Height	*Size (W × D) in mm*	*Alternative Bottom Reinforcement*	
		Number of C12, S16 or Y12 Bars or Main Wires of F11TM Fabric	*Number of Main Wires of F8TM Fabric*
(1)	(2)	(3)	(4)
Stable			
—one storey	300 × 300	2	3
—two storeys	400 × 400	3	4
Intermediate			
—one storey	300 × 400	3	6*
—two storeys	400 × 400	4	8*

TABLE 33.7A—*continued*
MINIMUM DIMENSIONS AND REINFORCEMENT OF EDGE BEAMS

Foundation Classification and Building Height	*Size (W × D) in mm*	*Alternative Bottom Reinforcement*	
		Number of C12, S16 or Y12 Bars or Main Wires of F11TM Fabric	*Number of Main Wires of F8TM Fabric*
(1)	(2)	(3)	(4)
Unstable			
—one storey	300 × 600	3	6*
—two storeys	400 × 600	4	8*

* Reinforcement is to be provided in two equal layers.

(*g*) On completion of the building the top surface of the slab shall be at a height above the adjoining ground level of—

(i) 75 mm, in the case of a slab located adjacent to a drained and paved area;

(ii) 100 mm, in the case of a slab located on a sandy, well drained *site*; or

(iii) 150 mm, in any other case.

(*h*) Stiffening beams shall—

(i) be *constructed* in accordance with the dimensions prescribed in Table 33.7B and in the manner illustrated in Figure 33.7B;

(ii) be reinforced in accordance with the provisions of Table 33.7B; and

(iii) be founded on soil or rock having an allowable bearing pressure of not less than 30 kPa;

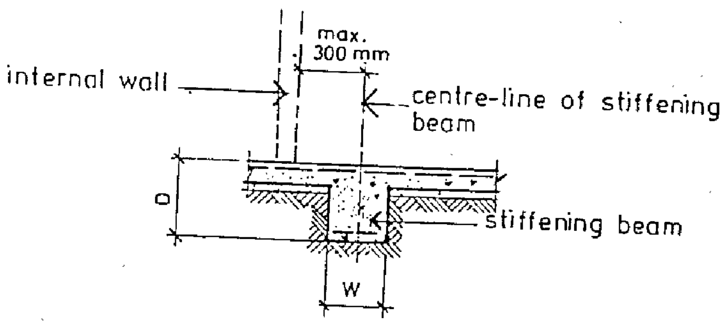

FIGURE 33.7B

TABLE 33.7B

MINIMUM SIZE, SPACING AND REINFORCEMENT OF STIFFENING BEAMS

Foundation Classification	*Beam Size (W × D)*	*Alternative Bottom Reinforcement*		*Maximum Spacing of Stiffening Beams from Edge Beams or other Stiffening Beams in any direction in metres*	
		Number of C12, S16 or Y12 Bars or Main Wires of F11TM Fabric	*Number of Main Wires of F8TM Fabric*	*Timber or Metal Framed Internal Walls*	*Masonry Internal Walls*
(1)	(2)	(3)	(4)	(5)	(6)
Intermediate	300 × 400	3	6*	4.5	3.5
Unstable	300 × 600	3	6*	4	3

* Reinforcement is to be provided in two equal layers.

(*i*) A support shall be provided under any *internal wall* in the manner prescribed by Part 1 of Table 33.7C if the wall is not located within 300 mm of the centre-line of a stiffening beam.

(*j*) A beam providing support for an *internal wall* in accordance with Part 1 of Table 33.7C shall be—

(i) *constructed* in accordance with Part 2 of Table 33.7B for stiffening beams and in the manner illustrated in Figure 33.7B;

(ii) if the wall is a *loadbearing* wall, be founded on soil or rock having an allowable bearing pressure of not less than 100 kPa; and

(iii) if the wall is a non-*loadbearing* wall, be founded on soil or rock having an allowable bearing pressure of not less than 30 kPa.

TABLE 33.7C

PART 1—SUPPORTS FOR VARIOUS TYPES OF INTERNAL WALL CONSTRUCTION

Number of Storeys	*Type of Wall Construction*	*Internal Wall*	
		Loadbearing	*Non-loadbearing*
(1)	(2)	(3)	(4)
one	timber or metal	no requirement	no requirement
two	timber or metal	beam required	additional slab reinforcement required
one	masonry	additional slab reinforcement required	no requirement
two	masonry	beam required	beam required

PART 2—MINIMUM SIZE AND REINFORCEMENT OF INTERNAL WALL SUPPORT BEAMS

Foundation Classification	*Beam Size (W × D)*	*Alternative Bottom Reinforcement*	
		Number of C12, S16 or Y12 Bars or Main Wires of F11TM Fabric	*Number of Main Wires of F8TM Fabric*
(1)	(2)	(3)	(4)
Stable	300 × 300	2	3
Intermediate	300 × 400	3	6*
Unstable	300 × 600	3	6*

*Reinforcement is to be provided in two equal layers.

(*k*) additional slab reinforcement providing support for an *internal wall* in accordance with Part 1 of Table 33.7C shall—

(i) be not less than 800 mm wide;

(ii) be positioned centrally under the wall in the lower part of the slab with a concrete cover of not less than 30 mm;

(iii) be placed in the manner illustrated in Figure 33.7C; and

(iv) comply with the provisions of paragraph (*l*);

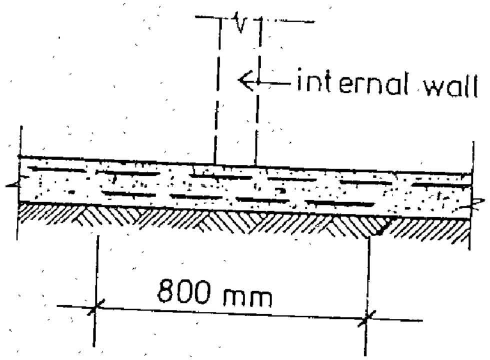

FIGURE 33.7C

(*l*) every slab shall—

(i) in the case of a slab resting on soil classified in accordance with Regulation 33.3 as stable, be reinforced with F72 mesh or bars of equivalent strength; or

(ii) in any other case, be reinforced with F82 mesh or bars of equivalent strength;

(*m*) A slab shall be not less than 100 mm thick.

(*n*) Pipes providing heat to a slab shall not be embedded in a slab less than 125 mm thick.

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(*o*) Where pipes are to be embedded or recesses provided in the slab, the slab shall be thickened in an *approved* manner to ensure that there is no loss of strength.

Reduced footing depth permissible

(4) Where a beam of a slab-on-ground, designed and *constructed* pursuant to this Regulation, is to rest wholly or partly on a floater or rock outcrop, the depth of the beam in the vicinity of the floater or rock outcrop may, subject to sub-regulation (5), be reduced to not less than two-thirds of the depth otherwise prescribed by this Regulation.

Reinforcement in reduced footing depth

(5) Where the depth of a beam is reduced pursuant to sub-regulation (4), the reinforcement in the section of the beam of reduced depth—

(*a*) hall be double the amount of that prescribed by this Regulation; and

(*b*) shall extend at least 500 mm beyond the section of strip footing or beam of reduced depth.

FOOTING SLABS

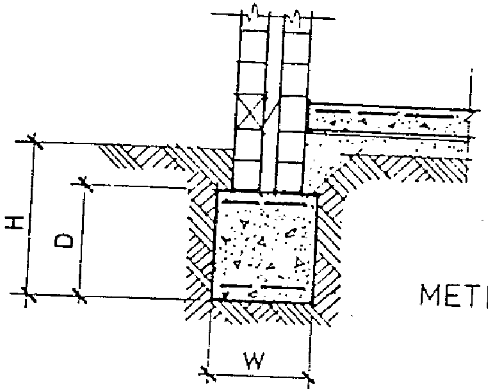
33.8 A footing slab system designed and *constructed* pursuant to this Regulation shall comply with the following:

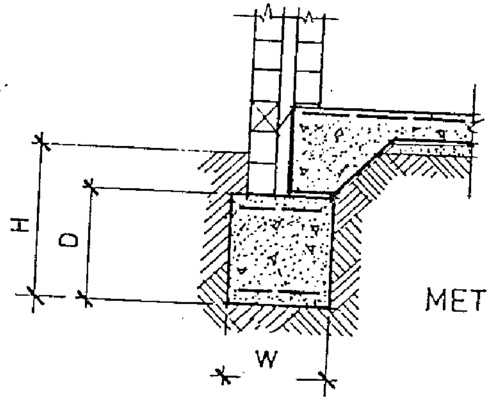
(*a*) Except where by reason of experience or local knowledge the *building surveyor* permits otherwise, the footing slab shall be founded on a *site* classified in accordance with Regulation 33.3 as stable.

(*b*) The configuration of the system shall conform with—

(i) one of the methods illustrated in Figure 33.8; or

(ii) any other method not less effective than the methods so illustrated;





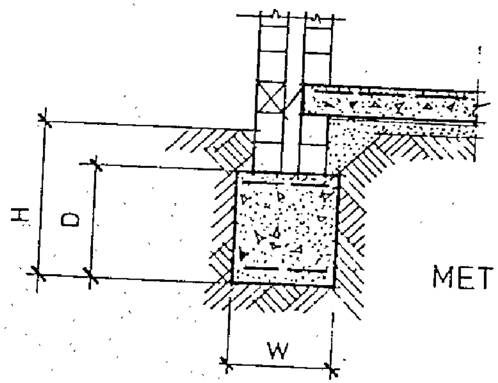


FIGURE 33.8

(*c*) The footing part of the system shall comply with the provisions of Regulation 33.6 as if it were a strip footing.

(*d*) The slab part of the system shall comply with the provisions of Regulation 33.7 (3) (other than paragraphs (*b*), (*d*) (iii), (*e*) and (*f*)) as if it were a slab-on-ground.

(*e*) Where, in the design and *construction* of a footing slab system, filling is restrained by an *external wall* and the filling is greater than 600 mm in depth, the *external wall* shall be designed by a *qualified engineer* and *constructed* in accordance with that design.

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FOOTINGS ADJOINING BOUNDARIES: PERMISSIBLE PROJECTIONS

33.9 Notwithstanding anything in Part 15, a footing may—

(*a*) support a *party wall*; and

(*b*) extend beyond the boundaries of a *street alignment*—

(i) to a distance of not more than 300 mm where the highest projecting part of the footing is at a depth of not less than 450 mm but is less than 3 m below the ground level; or

(ii) to a distance of not more than 1 m where the highest projecting part of the footing is at a depth of 3 m or more below the ground level.

APPENDIX IV

SITE CLASSIFICATION METHOD
EXTRACT FROM AS2870-1986

SECTION 2. SITE CLASSIFICATION

2.1 DESIGNATION. Site classes shall be designated as follows:

Foundation	Character	Class
Sand and rock Silt and some clay	Stable	A S
Moderately reactive clay Highly reactive clay Extremely reactive clay	Reactive	M H E
Sand Material other than sand	Controlled fill	A A to P
Mine subsidence Uncontrolled fill Landslip Soft Collapsing soils	Problem	P

2.2 CLASSIFICATION PROCEDURE. All site classifications shall be based on one or more of the following:

(a) Assumption of soil type without any site investigation from a classification map or from well established local knowledge provided that soils are known to be consistent over large areas. The soil type and site conditions shall be checked by a site visit before construction.

(b) Site investigation to identify soil profile using one or more boreholes or test pits in the site or a number distributed over a subdivision.

(c) Site investigation using a penetrometer, for sand sites.

(d) Site investigation including soil sampling and appropriate tests.

(e) Clause 7.2 for South Australia.

Where the Building Authority has designated a presumed site classification or simplified system based on a map of site classifications, this may be used but shall not preclude the adoption of a less severe classification if supported by a site investigation and a classification in accordance with this Section.

2.3 STABLE AND REACTIVE SITES.

2.3.1 Sand or rock site. Sand sites (to depths in excess of the depths of influence as defined in Appendix D or to rock) or rock sites, as defined in the standard, shall be classified as Class A.

2.3.2 Silt sites. Silt sites as defined in the standard or mixtures of sand and silt (to depths of influence as defined in Appendix D or to rock) shall be classified as Class S.

2.3.3 Clay sites. In addition to the general requirements of Clause 2.2, the procedure for the classification of a clay site shall include one or more of the following methods:

(a) Visual assessment of the site and interpretation of knowledge of existing masonry house walls on light strip footings which have existed for not less than 15 years in a similar soil assessed in accordance with Table 2.1.

(b) Identification of the soil profile and a classification in accordance with Appendix C or from established data on the performance of the soil profile.

(c) Computation of the predicted surface movement, y_s, in accordance with Appendix D, with the following limits:

Surface Movement	*Class*
$y_s \leqslant 20$ mm	S
$20 \text{ mm} < y_s \leqslant 40$ mm	M
$40 \text{ mm} < y_s \leqslant 70$ mm	H
$y_s > 70$ mm	E

2.3.4 Reduction of reactive site classification. The effect of the treatments below may be taken into account to improve the site classification:

(a) Removal and replacement of reactive clay with a non-reactive material and protection of any remaining reactive clay from moisture changes; or

(b) Covering the site with a layer of compacted stable material preferably well in advance of construction.

2.3.5 Soft foundations. Soft foundations are classified as Class P where the allowable bearing pressure at foundation level is less than the following values as appropriate:

(a) Under strip or pad footings ....100 kPa.

(b) Under beams and slab panels for all slabs, except that 100 kPa is required under the edge footing of footing slabs without ties......... 50 kPa.

Allowable bearing pressures shall be assessed in accordance with Appendix B.

NOTE: Inadequate allowable bearing pressure is not common except for silt sites.

TABLE 2.1

SIMPLE CLASSIFICATION OF CLAY SITES

(Damage categories are given in Appendix A)

Characteristic performance of masonry (veneer or full) house on light strip footings	Classification of site
Rare Category 0 or 1 damage	S
Often Category 1 damage but rarely Category 2 damage. Category 3 damage is very rare even in extreme environmental conditions. (The site may show surface cracking in dry periods.)	M
Often Category 1 or 2 damage with occasional examples of Category 3 damage or more severe. (Ground surface cracking is common in dry periods.)	H
Often Category 3 or more severe and area is usually well known for damage to houses and structures. (Deep ground surface cracking occurs in dry spells.)	E

2.4 CONTROLLED FILL SITES.

2.4.1 Controlled sand fill on sand sites. Controlled sand fill over sand site may be classified as a Class A site.

2.4.2 Shallow controlled fill. The effect of controlled fill up to 800 mm deep for sands and gravels and up to 450 mm deep for clay may be disregarded in the site classification.

2.4.3 Other controlled fill sites. Other controlled fill sites may be classified as Class S sites provided that the settlement and reactivity of both the fill and the underlying natural soil complies with Clause 2.3.3(c).

2.5 PROBLEM SITES. Where the site includes mine subsidence, uncontrolled fill, landslip conditions or soft soil (see Clause 2.3.5), the site shall be classified as a problem site (Class P) and a footing system shall be designed in accordance with Section 5.

APPENDIX C

SITE CLASSIFICATION BY SOIL PROFILE IDENTIFICATION

In some areas, where sufficient data has been established, an assessment of the reactivity of a clay profile may be associated with the descriptions in Tables C1, C2 and C3 provided that the clay on the site is identified as belonging to the profile described in the table. In neighbouring regions, the table may be used if soil types and climates are similar.

Where a range of classifications is given in the tables generally, the typical values should be used. A higher or lower classification may be required if the moisture conditions or soil profile on the site differ markedly from those normally expected.

NOTES:

1. Depth of clay layer refers to the thickness of the clay in the profile. Shallow may be taken to mean less than 0.6 m depth of reactive clay.
2. The tables can only be used in conjunction with a site investigation where variable soil conditions are expected such as Adelaide. Where the soils are consistent, such as Melbourne, geological or pedological maps may be used but the soil type should be checked by a site visit before construction.
3. Only a limited number of profiles are included. If the soil profile is not listed in the table then some alternative classification procedure should be used.
4. Where a range is given the classification may be based on the depth of clay, the depth of the water table, the drainage of the site and a visual assessment of the reactivity of the soil.
5. The soil type notation in Table C2 is taken from Bulletin 46 'Geological Survey of South Australia'. A classification from the table shall not be based solely on the maps given in that report on soil types. The Category E1 has been introduced primarily for those profiles which represent a transition from highly to extremely reactive.

TABLE C1
CLASSIFICATION BASED ON LOCATION AND TYPICAL PROFILE—VICTORIA

Examples	Classification
Melbourne and District	
Basaltic clays—	
≤ 0.6 m depth of clay layer	M
> 0.6 m depth of clay layer	H
Non-basaltic clays— (Including silurian and devonian residual clays and quaternary alluvial clays)	
≤ 0.6 m depth of clay layer	S
> 0.6 m depth of clay layer	M
Tertiary sediments	
≥ 1 m sand over clay	A
< 1 m sand over clay, assess on the basis of depth of clay layer—	
≤ 0.6 m	S
> 0.6 m	M
Westernport	
Alluvial clays	S to H
Horsham and district	
Grey brown cracking clays	H to E
Geelong	
Basaltic clays—	
≤ 0.6 m depth	M
> 0.6 m depth	H
Waurn Ponds formation	M to E
Tertiary sediments	S to M
Phillip Island	
Basaltic clays	H
Alluvial clays	M
Shepparton and District	
Quaternary alluvial clay	S to H

NOTE: The reactivity of the tertiary and silurian clays is variable, and some areas of high reactivity have been identified. Whilst the above classifications have generally been shown as satisfactory, particularly when combined with the requirements of Appendix A, if testing is not carried out, local experience should be considered when classifying a site.