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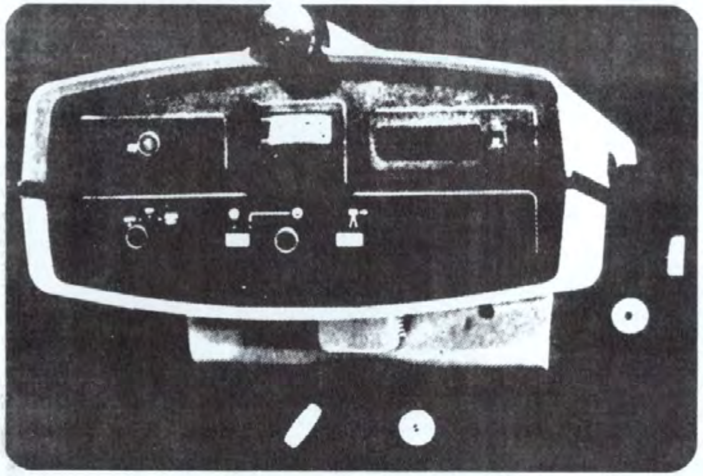


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ASSOCIATION OF SURVEYORS OF PAPUA NEW GUINEA

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A Merry Xmas

and

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COUNCIL NEWS

At the eighty-first Meeting of Council, the results of the postal ballot were declared.

Consequently: A General Meeting now requires a quorum of twenty-five Corporate Members.

Amendments to the Constitution and Bylaws may be made at the Annual General Meetings.

The official title of the Association is now "The Association of Surveyors of Papua New Guinea".

Membership Applications: Mr. J.N. Van As has advanced to Corporate Membership. Mr. R.J. Prikulis has been admitted to Corporate Membership.

Annual Subscriptions for 1975 have been determined by Council, as follows:

Corporate Member	$30
Corp. Member (non resident)	$15
Associate	$15
Student	$ 5

Members are reminded that subscriptions are due on January 1, each year.

Members are reminded that nominations to the grade of Fellow may be made at any time. A proposal for election to the Grade of Fellow must be signed by six Corporate Members and forwarded to Council.

Proposed Technician Association

Council is assisting in the initial stages of development of "The Association of Technician Surveyors of Papua New Guinea".

Scale of Fees Structure

Council has engaged the services of the Accounting Systems Research Centre headed by Prof. G.G. Meredith, to assist in determining professional fees.

URBAN SUBDIVISION MARKING

By: A.E. Naughton

Abstract

The Survey Directions 1970 are quite specific regarding the tolerances to be sought in the placing of ground marks at section and lot corners in urban subdivisions. 1.

Nevertheless the need for millimetre tolerances on all ground marks is questioned, and a balanced system of precise Permanent Survey Marks combined with approximate indicators at all other points is offered as the alternative.

Introduction

Every student of Surveying is taught that basically one should commence a survey with an overall control scheme, quite often roughly placed but precisely surveyed, rigidly checked by various techniques such as independent closed loops and mathematically adjusted by some method adequate for the purpose.

(The Bowditch method of adjustment is currently viewed with some disdain but, for subdivisions of the order of one hundred hectares in total area, it would seem adequate provided that resultant adjustments to angles and distances are sufficiently small in magnitude.)

This initial control scheme is subsequently broken down progressively until all minor points of detail are either surveyed (in the case of existing features) or marked. The actual placing of minor marks, whether they are boundary pegs or engineering design points, may well be slightly in error. In practice it is common for even pre-cast cement blocks to move slightly during settlement, introducing errors of some centimetres.[2]

Basic Requirements

Lot corners in even a first class area with maximum use of commercial lots rarely need to be placed with an accuracy better than about ten millimetres, while for a residential area landowners and lending authorities would, in the vast majority of cases, accept an error in the location of a corner of up to ten centimetres from the "true" position.

This statement presupposes that the error in the corner location is not subject to a further compounding effect of creep throughout some areas of the subdivision, caused by insufficient attention to accumulating errors in the initial setting out.

Anthony E. Naughton, Assoc. R.M.I.T. (Land Surveying), M.A.S.P.N.G., Lecturer Surveying Department, Papua New Guinea University of Technology, Lae.

It is appreciated that even P.S.M's set in concrete poured in situ are subject to quite severe disturbance during construction and that inevitably ground slump will occur near large open drains. The following proposal may well be worthy of consideration.

Proposal

A low cost survey carried out in the initial stages of a proposed subdivision would be adequate for control of levels and for the location of features to within plotting accuracy; rough chaining or even stadia could well serve this purpose.

Upon completion of the design of road and drainage systems, control points would be established, to fall approximately a half road width outside road reserves, that is, inside the proposed lots and on the side remote from any proposed drainage works.

This network of marks would then provide the control framework for the establishment of all other points at all stages of development and, in the penultimate stage for the connection to Permanent Survey Marks placed strategically throughout the subdivision and given adequate time to settle. The P.S.M. network would finally be checked back against the initial control points.

This network of Permanent Survey Marks, intelligently located to a density of the order of one P.S.M. per four hectares as recommended by the Surveyor General [3], appears adequate in that it imposes no great financial hardship on the developer while allowing the future surveyor to adopt a sound datum for re-establishment by checking one angle and measuring two lines of the order of 200 metres each.

The apparent duplication of control would possibly be unpopular with the developer; his savings, however, would be evident in the following considerations.

The only high precision envisaged would occur in the survey and adjustment of the initial control points, in the connection to town datum and in the survey and reverse checking of the P.S.M. network.

All other boundary points including section corners (except in commercial areas) would be indicated, rather than marked. A one metre star picket firmly driven into the ground and left to protrude sufficiently or a non-regulation wooden peg or some other acceptable indicator would serve this purpose.

A marked reduction in time and hence in cost is envisaged in this consideration, provided that each indicator need only be within a maximum of ten centimetres from the relevant point and provided also that intersections of section boundaries can be left unmarked with no reference marks other than the P.S.M's.

On occasion it may be necessary to define a corner in a precise manner, and adjacent corner indicators would obviously be of little or no value for this purpose. It would seem obvious that the original control framework or the P.S.M. network should be adequate for this operation.

Subsequent lot surveys as required from time to time would also benefit from a significant savings in time and cost. It is envisaged that an Identification Survey would require the following field work: the occupation of one P.S.M., the verification of the angle subtended by adjacent P.S.M's, the verification of the distance between two P.S.M's only, and the temporary marking of one front corner of the relevant lot by a checked radiation, the occupation of this corner, the verification of frontage, depth, and the angle subtended at the instrument station, and, in the case of a quadrilateral, the verification of the length of the other boundaries.

Instructions to mark the corners, as distinct from indicating them, would be more reasonably interpreted as an instruction to re-establish, thus necessitating a more refined approach, still a relatively simple exercise with an adequate datum.

Conclusion

Economics demand that in the survey of urban subdivisions only control marks (including Permanent Survey Marks) merit the refinement of millimetre accuracy [4], and sound survey principles favour sufficient precise permanent monuments with other points indicated rather than marked.

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1. Survey Directions 1970, 4.1, 5.1-4, 9.8.

2. Circular Surveyor General to all surveyors, Oct 15, 1971 re placing of cement pegs.

3. Circular Surveyor General to all surveyors, Dec 8, 1971, re density of P.S.M. networks.

4. M.D. Kellock: The quest for the millimetre, Australian Surveyor, March 1973.

RE-ESTABLISHMENT BY RESECTION TO UNKNOWN POINTS

By: G.J. Andrews.

1.0 Introduction

1.01 The problem of the permanence of survey marks and position is a universal one. We have a limited repertoire to call upon when a particular position has to be fixed, irrespective of whether the purpose of the survey is cadastral, engineering or geodetic. Traditional standard marks are placed at the actual position or corner, and auxiliary marks placed as insurance against disturbance of the actual mark. Thus for cadastral surveys in Queensland, wooden pegs placed at the corner of cadastral blocks are supplemented by reference marks in the form of iron pins, permanent marks set in concrete, reference trees, pointer pegs, fence posts, etc. I would imagine that with minor variations this is the approach used in most countries.

Probably the ideal solution to fixing position permanently, thus allowing unambiguous re-establishment of the original position, is the implementation of a coordinate system based on a dense, adjusted network of control points together with a realistic error ellipse. The coordinates of the new station would be the legal definition of the corner equally with the physical marks so long as the physical marks fell within the error ellipse.

However, even this proposed "ideal" solution relies to some extent on the permanence of physical marks so that we come back again to the problem of preservation of position for re-establishment purposes. It is intended that the proposal, which is the subject of this paper, should give surveyors an extra tool in their repertoire. It probably cannot be applied to every point whose position needs to be fixed, but it has some distinct advantages over traditional methods of fixing and re-establishing position.

1.02 The Proposal

A typical example will be used to illustrate the proposal. A surveyor effecting an original survey, wishes to establish a new corner, and preserve that position for future reference. He places the traditional mark at the corner. He notes that three (preferably four) fixed points can be seen from the new corner. These points may be lamp posts, houses, fence posts, windmills, sign posts, etc. He observes a round of angles to the points. It is not necessary to relate these angles to the meridian of his survey although this is preferable. He estimates the distances to each of the points from his position.

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A second surveyor at some future time has the task of re-establishing the original position. Before proceeding into the field he prepares a prediction chart based on the observed angles and estimated distances of the original surveyor. The preparation of the chart takes about 10 minutes. In the field, he sets up in the approximate vicinity of the corner, (say within 30 metres) and reads a quick round of angles (once on face left is sufficient) to the original points. The difference between his observed angles and the original angles are computed. By applying these differences to the prepared chart, he obtains an estimate of the position of the original corner. He sets up on this predicted position and repeats the angles. Eventually an accurate round of angles is observed which agrees exactly with the original round of angles. The new surveyor is now occupying the original corner. If the original surveyor had have related the bearings to the points to the meridian of his survey, the new surveyor would now have position plus original meridian after 10 minutes to 15 minutes work by one man. If the original surveyor had have observed to four points (ie. observed three independent angles), the new surveyor would now have position, meridian and check on position with an estimate of the error in that position.

2.0 Theory

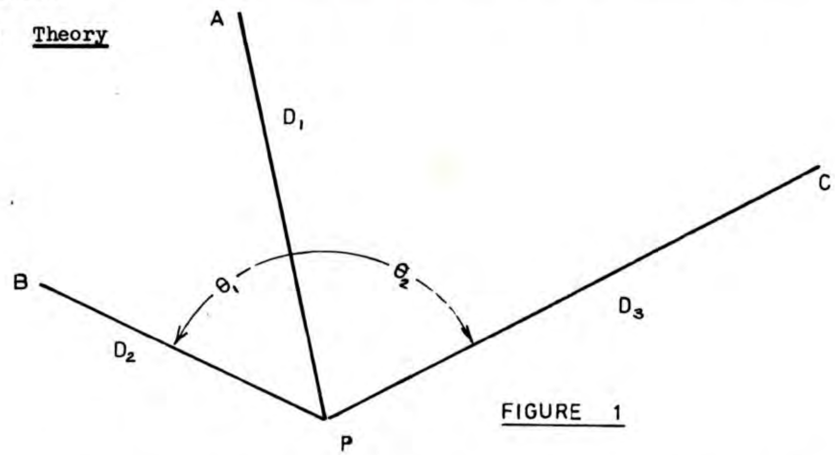

FIGURE 1

2.01 Figure 1 shows a typical situation the original surveyor at P, has observed the angles θ_1 and θ_2 and estimated the distances D_1, D_2 and D_3 to three fixed points whose positions are otherwise unknown. It will be seen later that it is convenient to always choose the middle point as D_1, and to consider the angle θ_1 and θ_2 as being measured from D_1, ie. the positive direction of θ_1 is anticlockwise and that of θ_2 clockwise as indicated by the arrows in Figure 1.

2.02 Since A and B are fixed points, (despite the fact that their position is not known, or only known approximately), there is only one unique circle which passes through them to which they subtend θ_1. The bearing of the tangent to this circle at P_1 can be divided by calculating the angle between D_1 and the radius O_1P (ϕ_1), and adding 90^o. (See Figure 2). Since, in most cases, the radius of the circle is very large; and we are only concerned with the area in the immediate vicinity of P, this tangent can be considered as approximately the actual arc of the circumscribing circle. Thus a straight line could be drawn on the ground through P, on the above deduced bearing exactly representing the circle through A and B. Any point on this line will subtend θ_1 to A and B. A similar line could be drawn on the ground representing the circle through A and C.

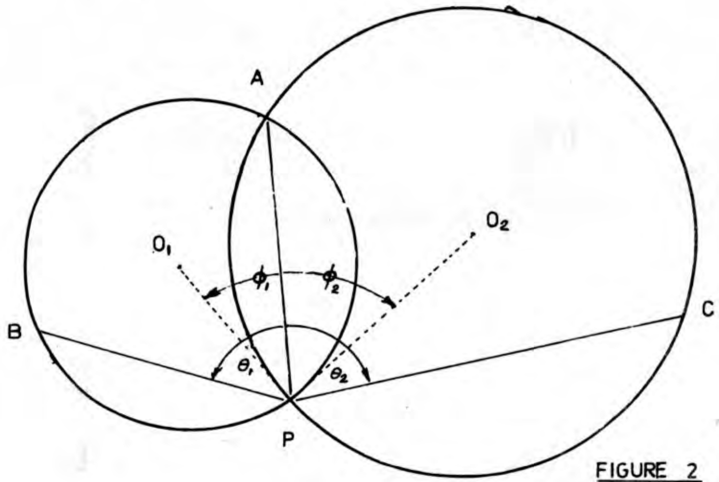

FIGURE 2

2.03 The angles ϕ_1 the angle measured anticlockwise from D_1 to the radius O_1P and ϕ_2 the angle measured clockwise from D_1 to the radius O_2P, can be deduced three ways:

(1) from the formula -

$$\tan \phi = \frac{D_2 \sec \theta - D_1}{D_1 \tan \theta} \qquad \text{—— (1)}$$

(See Appendix I for the deduction of this formula.)

(2) by closing the figure PMON in Figure 3, on an assumed meridian, for ON, whence -

$$\phi = \tan^{-1} 2 \frac{ON}{D_1}$$

(3) by graph. Graph I shows a family of curves for various ratios of $\frac{D_1}{D_2}$. ϕ can be read off for various values of θ, the observed angle. It will be seen that for practical purposes Graph I is adequate for all cases and that ϕ or bearing need only be interpolated to the nearest degree.

2.031 As an example, suppose an original surveyor observes the angles θ_1 and θ_2 as 68^o and 132^o and estimates the distances as shown in Figure 4. For the 68^o angle $\frac{D_1}{D_2} = 3$, interpolating, from the graph $\phi_1 = -3^o$. For the second angle $\frac{D_1}{D_3} = 0.8$ giving $\phi_2 = 69^o$. Assuming D_1 to have a bearing of 0^o, the bearing of the tangents can now be plotted as 87^o and 159^o respectively.

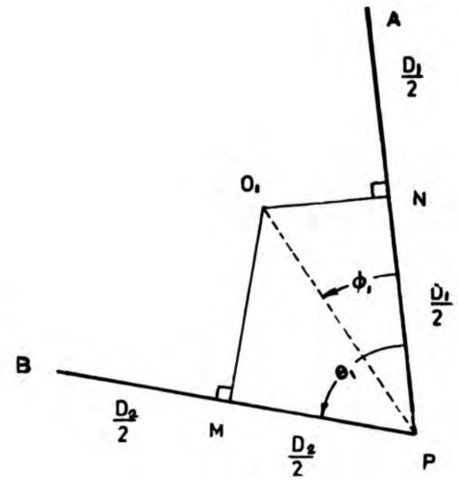

FIGURE 3

2.04 Shift Distances

In 2.02, it was shown that lines could be drawn on the ground representing exactly the circle through A, B and P. A and B subtend θ_1 to any point on this circle. If, instead of observing θ to A and B, the observer had have observed an angle $(\theta_1 + \chi)$, where χ is very small, the observer must be on a different circle to the one whose position was deduced in 2.02. In the vicinity of P, this new circle can be represented by a straight line, parallel to the original circle and a distance "s" towards the points A and B (since the new angle is greater than the first).

"s" in metres is given by the formula -

$$s = \frac{D_1 \operatorname{Cos} (\theta - \phi) \chi' \sin 1'}{\operatorname{Sin} \theta} \qquad \text{———— (2)}$$

where χ is now measured in minutes of arc.

(See Appendix II for proof.)

Equation 2 has been used to produce Graph II which gives shift distance in metres against the ratio $\frac{D_1}{D_2}$ for various values of subtended angle θ. For convenience, D_1 has been taken as 1,000 m. As with Graph I, Graph II can be used with sufficient accuracy for all cases of re-establishment. Referring again to Figure 4 and the example in 2.031 $\frac{D_1}{D_2} = 3$. Since the graph only accommodates ratios of $\frac{D_1}{D_2}$ less than unity, the ratio will be temporarily inverted, thus $\frac{D_2}{D_1} = 0.33$. Interpolating from the graph for observed angle $\theta = 68^\circ$, s = 0.313. Since the numerator of the ratio is 400, the actual value of shift is therefore $(\frac{400}{1000} \times 0.313) = 0.124$. Thus, a line drawn parallel to, and 0.124 from the original 87° tangent line, will represent the position line of an observer reading an angle of $68^\circ 01'$ between A and B in the vicinity of P. For $\theta_2 = 132^\circ$ and $\frac{D_1}{D_3} = 0.8$, from Graph II, "s" is interpolated as 0.175. Since $D_1 = 1200$ the shift to be applied is thus $(\frac{1200}{1000} \times 0.175) = 0.210$.

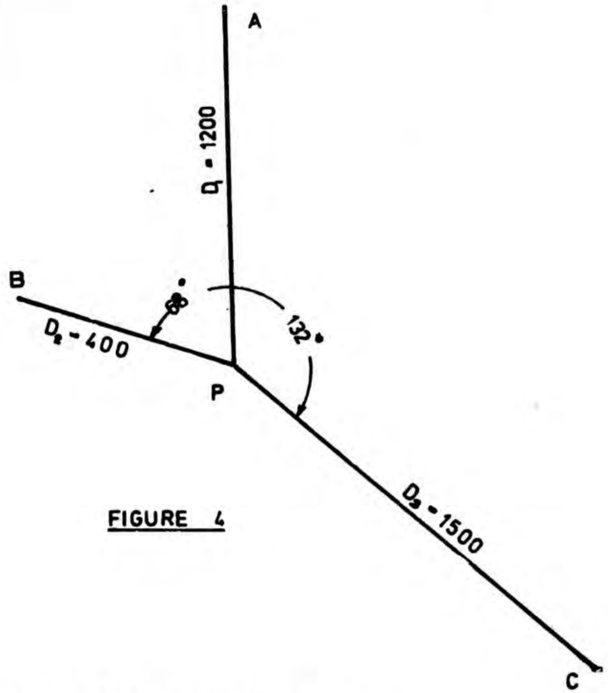

FIGURE 4

3.0 Preparation of the Prediction Chart

Using the information deduced in 2.031 and 2.04, a re-establishment chart can now be constructed at any convenient scale. Natural scale is often possible, but in this case a scale of 1:10 has been chosen for convenience.

Through a central point representing the original station; lines bearing 87° and 159° are drawn. These represent the circles at which 68° and 132° (the original observed angles) are subtended respectively. The meridian has been arbitrarily defined to coincide with the distance D_2. A family of lines 13 mm. apart and parallel to the former (87°) circle are drawn and numbered +1, +2, +3 etc. for lines on the near scale, and -1, -2, -3 for those lines away from A and B. These represent position lines at which $68^\circ 01'$, $68^\circ 02'$, $68^\circ 03'$ etc. would be subtended to A and B, or in the case of the negative values $67^\circ 59'$, $67^\circ 58'$, $67^\circ 57'$ etc. A similar family of lines 21 mm. apart and parallel to the second "circle" (159°) would be ruled and labelled +1', +2', +3' etc. The final result would be as shown in Figure 5.

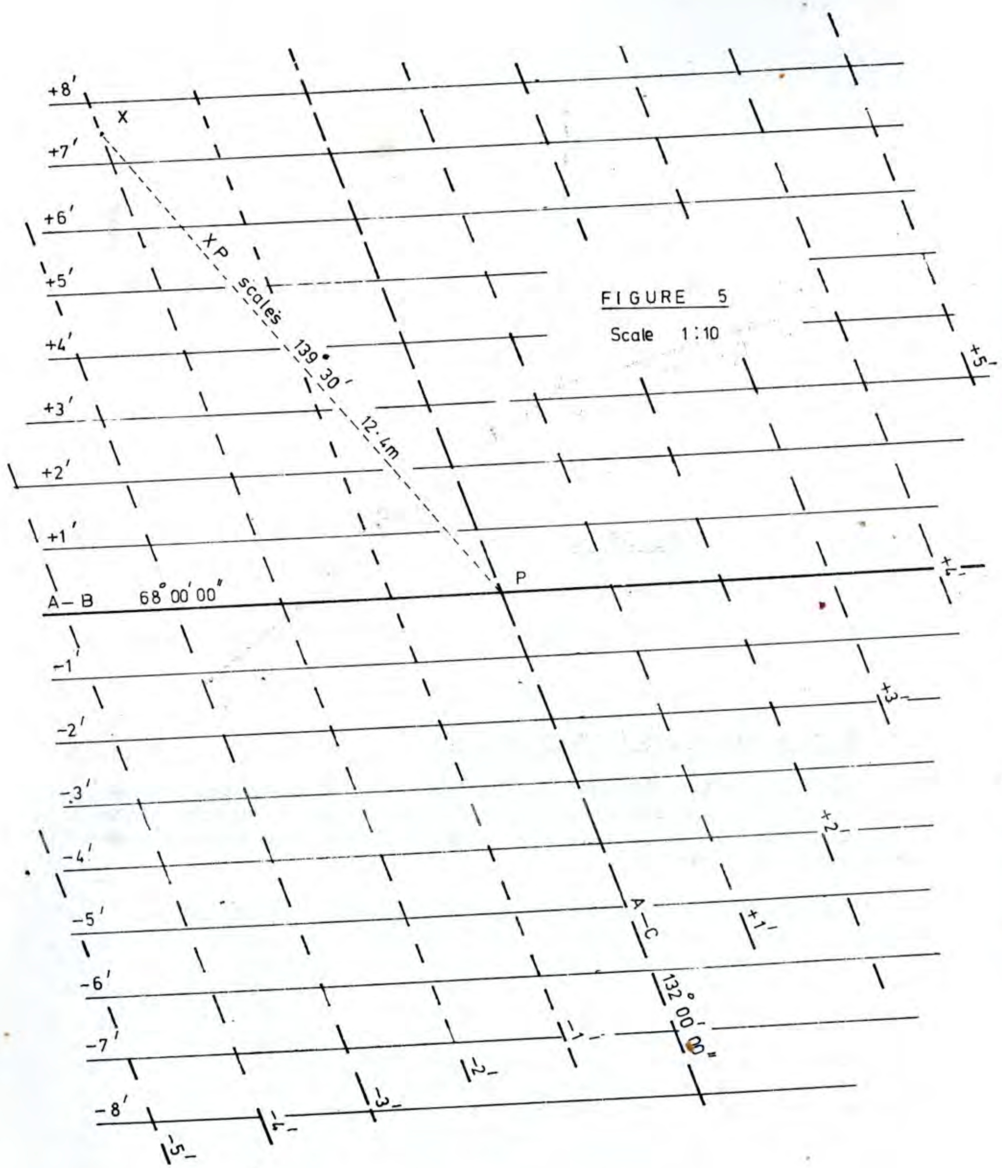

FIGURE 5

Scale 1:10

4.0 Application in the Field

With the prediction chart in Figure 5 a surveyor can now re-establish the original station P. The instrument is set up in the vicinity of P, and a quick round of angles observed on one face to the nearest minute of arc. Suppose the results are:

$$\theta_1 = 69^\circ 15'$$

$$\theta_2 = 131^\circ 40'$$

This gives differences of $+1^\circ 15'$ and $-20'$ with the original angles respectively. Since the observed differences are large, the scale of the prediction chart may be temporarily considered to be 10 times less than plotted ie. new scale is 1:100, and each position line now represents a change of 10' of arc instead of 1' as plotted. The observed differences of +75' and -20' place the observer at point X in Figure 5. The scaled bearing and distance is $139^\circ 30'$ for 12.4 m. allowing for the 1:100 scale change. The surveyor sets out this bearing and distance using say the bearing of D_3 of 132° as an R.O. (Since D_3 is the greatest distance.)

A rapid round of angles at the new position is repeated, and if the differences observed are small as is the usual case the round of angles is repeated using a more accurate system of observation, say by repetition.

If the observed differences are very small, the scale of the diagram can be enlarged to accommodate these small ground distances ie. at natural scale, the chart in Figure 5 represents position lines of $0^\circ 00' 06''$ differences $\left(\frac{0^\circ 01'}{10}\right)$. Figure 5 is now a record of the position of P which is as "permanent" as the distant reference marks. Note that the scale changes are temporary and require no extra plotting. A suitable scale can always be chosen to ensure that the observer's position can be plotted in any prepared chart.

5.0 Accuracy and Optimum Conditions

5.01 The accuracy of the final re-establishment of position is dependent directly on the accuracy with which the angles can be observed by both original and re-establishing surveyor. If each angle could be reproduced with an accuracy of $\pm$ 20", this discrepancy could be translated to position lines either side of the position lines for accepted value of each angle. Thus a parallelogram can be constructed representing the bounds inside which the true position of the original corner must lie. (See Figure 6.) It is obvious that the accuracy of the re-established position on the ground is enhanced for a configuration of observed objects that gives "s" as <u>small</u> as possible; and the angle between position lines between say 60° and 120° but preferably 90°.

Re-establishment by Resection to Unknown Points

5.02 Before proceeding on with a discussion on the optimum position for observed objects, it may be instructive to note that when this system is used to re-establish position, inaccuracies in the estimation of distances to these reference objects cause no error in the final re-establishment of the station. If it were possible to obtain the exact distances to the reference objects, it would only be necessary to read one accurate round of angles in the vicinity of the true position in order to exactly locate the original station. Inaccurate distance estimates only generate an imperfect prediction chart. This necessitates a couple of quick rounds which generally causes a rapid convergence on the true position. For example suppose $D_1 = 500$ m., $D_2 = 1,000$ m., and $\theta = 40^\circ$. From Graphs I and II: $\phi \doteq 63^\circ$, and $s \doteq 0.21$ m. However, if D_1 were estimated at 400 m. and $D_2 = 1200$ m., the values of ϕ and s actually used would be 0.15 and 76°. Although these incorrect values, when used, would cause inaccurate estimates of the true corner with respect to the existing instrument position, a subsequent round of angles at the new position would generate a further shift in the approximate direction of the true position. Thus if subsequent predicted positions do not converge rapidly within say three rounds of angles, a check on the accuracy of the estimation of distances would be indicated. The final test of correct re-establishment will always be an exact reproduction of the original angles.

5.03 Optimum positions of reference marks. As previously indicated, the most desirable configuration of reference marks is one which gives small values of "s" for each family of lines which should intersect within the range, (say) $60^\circ - 120^\circ$.

5.031 Minimization of "s" can be achieved by -

(1) keeping D_1 in the ratio $\frac{D_1}{D_2}$ as small as possible. The formula for "s" is directly proportional to D_1.

(2) selecting subtended angles in the range $70^\circ - 180^\circ$. Once again referring to Graph II, values of "s" lower than 0.29 occur for any subtended angle between 70° and 180°, for all ratios of $\frac{D_1}{D_2}$. The most favourable configuration for a small value of "s" is to be midway between two reference marks ie. $\frac{D_1}{D_2} = 1$, subtended angle $\theta = 180^\circ$. Where the subtended angle is less than about 50°, "s" can be minimized by keeping the ratio $\frac{D_1}{D_2}$ less than say 0.25.

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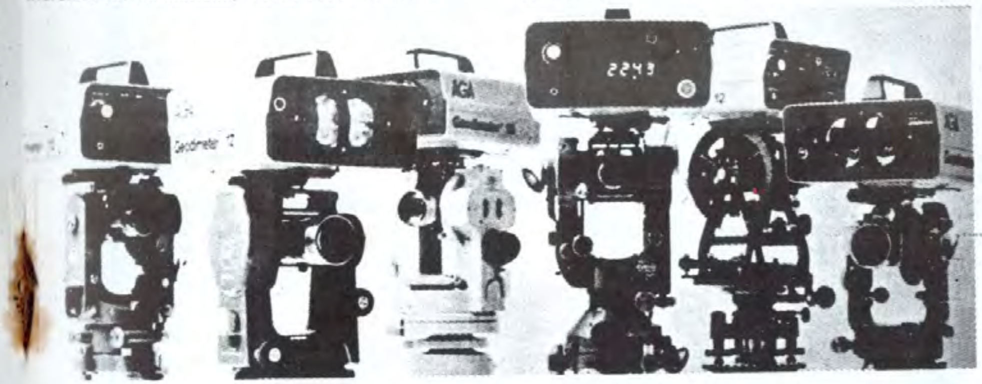

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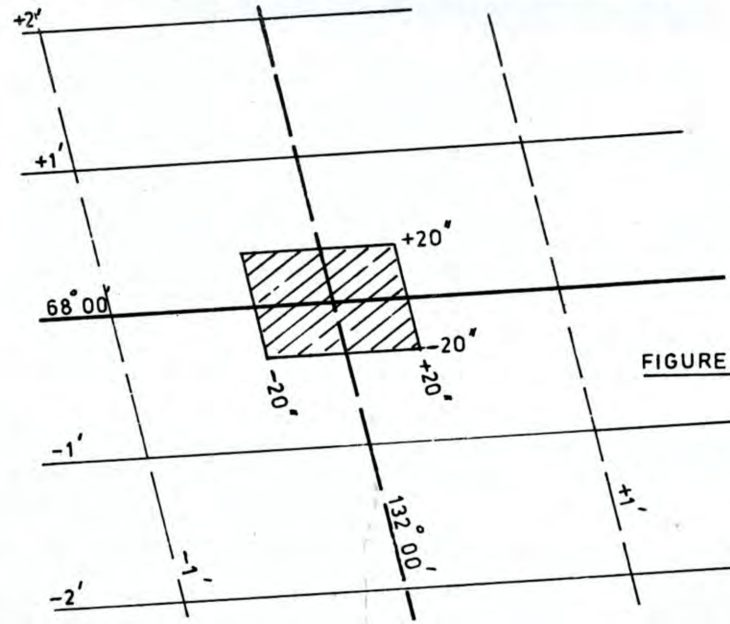

FIGURE 6

5.032 Optimization of the angle between position lines is assisted by referring to Graph I. If there are n reference marks, the number of angles possible are ${}^{n}C_2$; each of these angles generating its own position line, thus if n = 3, the number of available angles to choose from is 3; if n = 4, the number of angles available is 6. When n is 4 or more, it may be necessary to examine each possibility separately in order to choose a suitable compromise between angle between position lines and minimization of "s". From Graph I it can be seen that for small values of $\frac{D_1}{D_2}$ and irrespective of subtended angle, $\phi \doteq 90^{\circ}$. Thus the circle is approximately in the direction of D_1 the smaller distance. Therefore an ideal configuration would have the two smaller distances perpendicular to each other with the third very large distance in any direction.

This deduction leads in fact to the interesting proposition that the sun may be used as one of the referring objects, in combination with two close reference objects approximately perpendicular to each other. This system would be applicable where only two reference objects are available. The original surveyor would observe the angle between these two marks, and take a normal observation on the sun for azimuth using one of the reference marks as R.O. for the observation. The true azimuth of both reference marks can thus be deduced and a prediction chart prepared from Graphs I and II in the normal way. A re-establishing surveyor would then be required to repeat the process, deducing new true azimuth values of the two reference marks by observation.

In his article in the Australian Surveyor, Vol 26 No. 1 March 1974, Bennett shows that the most consistently accurate method of determining true azimuth by solar observation is by observing hour angle. This method allows observations outside the normal limitations of the observed zenith distance method. With calculators such as Compucorp, HP45, and HP65, true azimuth can be obtained in a matter of minutes. Using this system, two reference marks 300 m. from the original station subtending 90° at that station would allow its re-establishment within a 30 mm. square, assuming $\pm 10''$ error in angle observation. The same prediction chart would be applicable to both original and re-establishing surveyor since $\frac{D_1}{D_2} = 0$ for any position of the sun. From Graph II $s = 0.2909$ when $\frac{D_1}{D_2} = 0$.

5.033 Movement of marks. When traditional monument or reference marks are disturbed the error in the adopted position bears a relationship to the amount of disturbance. With the proposed system of re-establishment movement of reference marks can cause a wide variety of movement of the original station. However it is possible to improve of the normal 1:1 ratio of movement of mark to error in adopted position with suitably positioned reference marks. In some cases, movement of reference marks cause no error in the re-established position. This would occur when the movement does not generate a change in observed angle. However, if a reference mark moves such as to cause a change in the observed angle at the true position the error in the re-establishment of the original position will be directly proportional to the angular change, and "s". Thus movements in the more distant reference marks have less effect than movements of closer marks. For an object say 1 km. from a station to move $\pm$ 100 mm. would generate a maximum error of only 20" or $\frac{1}{3}$ of the value of s from Graph II. Now this value of s may arise from a configuration of marks that further decreases the effects of the movement of reference mark - ie. an observed angle of 70° or more gives values of s less than the "normal" 0.2909/1,000 m. values. Thus if the 1,000 m. line in the above example happens to be used in conjunction with a D_1 of say 500 m., with a subtended angle of say 130°, from Graph II $s = 0.215/1{,}000$ m., giving a true value of $s = 0.107$. The original 100 mm. movement has now become a 36 mm. error on the ground, since the actual angular change is only 20".

6.0 Applications

6.01 The proposed method has been tested in the field and found to be a practical solution to re-establishment of position to cadastral accuracies using a normal 20" theodolite and it is in this capacity that the method can be most profitably applied taking advantage of marks that can be seen from a particular corner.

6.02 However in estate development the principle can be refined by deliberately placing marks at random throughout the estate such that any three or four can be seen from any point on the development. Section corners could be fixed using this method, then re-established at any subsequent time irrespective of now much earthworks or other disturbance has occurred in the vicinity of the corner. Note that no formal connection of reference marks is required ie. control survey is not necessary.

6.03 Should an accurate control survey have been effected however, and these stations are visible, the method can be used to set out any new position by resection.

The coordinates of the required position are computed and from these, the angles that the various control points subtend at the required point. A prediction chart similar to Figure 5 can now be prepared based on these angles. Since the chart is based on known distances, the chart will be without error. The determination of the required position will therefore require one, perhaps two rounds of angles, depending on how close the initial set up is to the required position.

6.04 It is hoped that the system will be applied in the near future to the control of the construction of a large rock-fill dam. Suppose centre line of the structure has to be set out at 20 m. intervals, and each position has to be re-established every 2 m. vertical rise. Six permanent targets could be set out in the configuration shown in Figure 7. The proposed centre line of the structure would be established initially in the ground and angles read to any four (or all) of the marks from each centre line position with say a second order theodolite. The distances could be estimated with some degree of precision to make the prediction chart accurate.

Each centre line position would now have its own prediction chart prepared. All positions would use the stations B and E as one angle and any two (or more) other marks. During the construction period any point can now be re-established by one instrument man only. The accuracy required in this stage is not great so a 20" theodolite would be sufficient. When greater accuracy is required say for positioning concrete form work, the second order theodolite can be substituted.

6.05 EDME can be incorporated easily into the system if necessary since the prediction chart is a scaled plan of the area in the vicinity of the original station. Therefore any true distance from the original position to a reference mark can be represented on the chart by a line through P, perpendicular to the bearing of the reference mark. Observed differences with this original distance can be represented on the prediction chart by a line parallel to and the observed differences from, this original line.

FIGURE 7

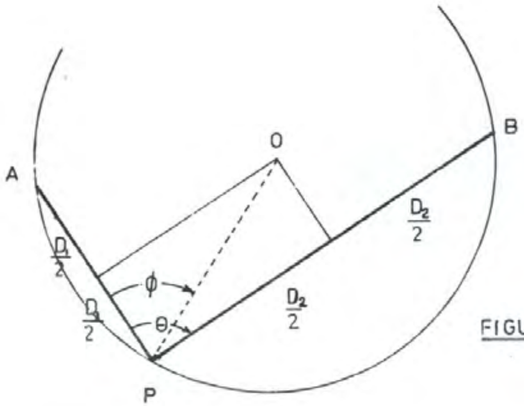

FIGURE A

APPENDIX I

Proof of the Formula $\phi = \tan^{-1}\left(\frac{D_2 \operatorname{Sec}\theta - D_1}{D_1 \tan\theta}\right)$

In Figure A, the points A and B are fixed but otherwise unknown points subtending at P. O is the centre of the circle through A, B and P.

$$OP = \tfrac{1}{2} D_1 \sec\phi = \tfrac{1}{2} D_2 \sec(\theta - \phi)$$

$$\text{ie.} \quad D_1 \operatorname{Cos}(\theta - \phi) = D_2 \operatorname{Cos}\phi \qquad \text{———— (1)}$$

$$D_2 \operatorname{Cos}\phi = D_1 (\operatorname{Cos}\theta \operatorname{Cos}\phi + \sin\theta \sin\phi)$$

$$D_2 = D_1 \operatorname{Cos}\theta + D_1 \sin\theta \tan\phi$$

$$\therefore \quad D_1 \operatorname{Sin}\theta \tan\phi = D_2 - D_1 \operatorname{Cos}\theta$$

$$\therefore \quad \tan\phi = \frac{D_2 - D_1 \operatorname{Cos}\theta}{D_1 \operatorname{Sin}\theta}$$

$$\therefore \quad \phi = \tan^{-1}\left(\frac{D_2 - D_1 \operatorname{Cos}\theta}{D_1 \operatorname{Sin}\theta}\right)$$

$$\underline{\text{OR}} \quad \phi = \tan^{-1}\left(\frac{D_2 \operatorname{Sec}\theta - D_1}{D_1 \tan\theta}\right)$$

$$\underline{\text{OR}} \quad \phi = \tan^{-1}\left(\frac{D_2}{D_1} \operatorname{Cosec}\theta - \operatorname{Cot}\theta\right)$$

APPENDIX II

Proof of the Formula $s = \dfrac{D_1 \operatorname{Cos}(\theta - \phi)x}{\operatorname{Sin}\theta}$

In Figure B, O is the centre of the circle through A, P and B. P^1 lies on a second circle through A and B. $PP^1 = s$, and is measured radially towards O.

$\angle APB = \theta$, the original angle

$\angle AP^1B = \theta + x$, when x is a small difference (in radians).

$$x = \alpha + \beta$$

$$= \frac{P^1L}{D_1} + \frac{P^1K}{D_2} \text{ (for small angles)}$$

$$= \frac{s \sin\phi}{D_1} + s\,\frac{\sin(\theta - \phi)}{D_2}$$

$$= s\left(\frac{D_2 \sin\phi + D_1 \sin(\theta - \phi)}{D_1 D_2}\right) \qquad (2)$$

From Equation 1 in Appendix I.

$$D_1 \sec\phi = D_2 \sec(\theta - \phi)$$

ie. $D_2 = D_1 \sec\theta \operatorname{Cos}(\theta - \phi)$

Substituting for D_2 in (2) above,

$$x = s\left[\frac{D_1 \sec\theta \operatorname{Cos}(\theta - \phi)\sin\phi + D_1 \sin(\theta - \phi)}{D_1 \sec\theta \operatorname{Cos}(\theta - \phi)}\right]$$

$$= s\left[\frac{\operatorname{Sec}\theta \operatorname{Cos}(\theta - \phi)\sin\phi + \sec\theta \operatorname{Cos}\theta \sin(\theta - \phi)}{D_1 \sec\theta \operatorname{Cos}(\theta - \phi)}\right]$$

$$x = s\left[\frac{\sin(\theta - \phi + \phi)}{D_1 \operatorname{Cos}(\theta - \phi)}\right]$$

$$\therefore s = \frac{D_1\, x \operatorname{Cos}(\theta - \phi)}{\operatorname{Sin}\theta}$$

$$= \frac{D_1 \operatorname{Cos}(\theta - \phi)\ x^1 \sin 1^1}{\operatorname{Sin}\theta}$$

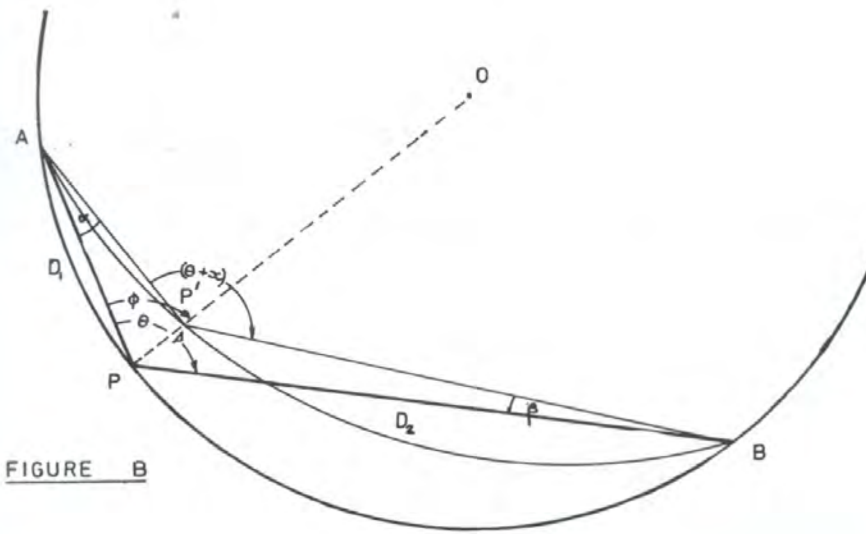

FIGURE B

CALCULATION OF WATER VAPOUR PRESSURE

For those surveyors using programmable calculators and involved in e.d.m. measurements and barometric levelling, it is frequently more desirable to calculate water vapour pressure as part of an overall programme rather than referring to psychrometric tables.

From the Smithsonian Meteorological Tables 1971 are quoted the formulae:-

$$e = e_w + de$$

$$de = - 0.000660 \, (1 + 0.00115 \, t') \, p \, (t - t')$$

$$\begin{aligned} \log 10 \; e_w = & - 7.90298 \, (T_s/T' - 1) \\ & + 5.02808 . \log_{10} (T_s/T') \\ & - 1.3816 . 10^{-7} \, (10^{11.344 \, (1 - T'/T_s)} - 1) \\ & + 8.1328 . 10^{-3} \, (10^{-3 \; 49149 \, (T_s/T' - 1)} - 1) \\ & + \log_{10} 1013.246 \end{aligned}$$

e = required water vapour pressure in millibars.
e_w = saturation water vapour pressure at the wet bulb temperature.
de = correction to e_w to obtain e.
p = atmospheric pressure in millibars.
t = dry bulb temperature in ^{o}C.
t' = wet bulb temperature in ^{o}C.
T_s = 373.16 ^{o}K.
T' = $(t' + 273.16)$ ^{o}K.

CALCULATION OF VERTICAL ATMOSPHERIC REFRACTION

As with the above, surveyors may programme a formulae for vertical atmospheric refraction into their astronomy reductions. From Roelofs, Astronomy Applied to Land Surveying:-

$$r'' = \frac{P}{1013.25} \cdot \frac{273.2}{273.2 + t} \, (60.10 \, \mathrm{Tan} \, z - 0.072 \, \mathrm{Tan}^3 \, z)$$

r'' = atmospheric refraction in secs. of arc.
P = atmospheric pressure in millibars.
t = dry bulb temperature ^{o}C.
z = observed zenith distance.

THE HISTORY OF THE BOUGAINVILLE COPPER PROJECT

By: J.D. Erickson

Edited extracts from a talk delivered at the Ninth Survey Congress, Bougainville, by Mr. Erickson, Assistant General Manager, Production, Bougainville Copper Ltd.

Bougainville Copper was officially discovered in 1963 when some of the records of the geological investigations were found and evaluated. As a result, Bougainville was picked as a prospecting target and a prospecting authority was taken out by C.R.A. Exploration.

Ken Phillips, a C.R.A. geologist, started exploration in 1964. He and his assistants hiked over the mountain into the Panguna Valley and commenced their soil samples. The initial exploration and sampling indicated that there was copper in the area which was of enough interest to start drilling programmes. Initially, small drills were back-packed over the mountain into the valley and drilling was commenced on a small scale. It was soon evident however, that it was necessary to bring in larger drills. These were brought in by helicopter support. It became an extremely expensive proposition to support an exploration effort of the size we had; so we had to put in an access road.

Moving on into 1966, we realized that we had moved into an area where there could be some significant disturbance. It was noted that the current mining rights at the time made no provision for any compensation to the land owners. We were instrumental in getting the legislation changed so that compensation could be paid to the people who were disrupted by the mining operations.

In 1967, it became more apparent that there was more copper in the island. We already spent a considerable amount of money and there was still no guarantee that we were going to keep the lease. At that time, we were operating under a prospecting authority which had to be renewed on a periodic basis and all the government had to say was "I'm sorry, we can't renew it," and a considerable investment would have been lost. So the Bougainville copper agreement was negotiated with the Government and ratified by the House of Assembly in 1967. This became the base document for all the work involved in the establishment of Bougainville Copper.

With the agreement, the underground development started. As the development proceeded, it was obvious by the beginning of 1969 that this was a mineral deposit which was economically viable. The base document was essentially a collection of the information we had, the opinions of experts and an economical evaluation and assessment of the mine's potential. This information was the basis on which we were able to borrow the amount of money that we needed and to establish the sales contracts that we had.

In 1970, the construction effort started full blast. By mid 1971, construction reached its peak, where we had over 10,000 workers on site.

In early 1972, we had the final commissioning of the plant and commercial operation commenced on 1st April, 1972. The Bougainville Copper operation, that is the truck and shovel operation, officially commenced with its pre-stripping operation in November 1970. So, the mine as such has been operating since 1970; but it was predominantly a stripping operation until ore drilling started in 1972.

As at the 30th of June, 1972, we have mined 66 million tonnes of ore which yielded 1,400,000 tonnes of concentrate containing 1,397,000 tonnes of copper, 42,000 kilograms of gold and 96,000 kilograms of silver. So,we have got off to a flying start from a humble beginning back in 1963.

The initial financial outlay was A$21,000,000 and this was essential before we were in a position to say whether or not we have a mine. This amount invested initially is significant in any type of venture.

A question was asked on how investigations were carried out. Initially, geologists came in and their first mode of attack was through soil samplings. Because of the dense jungle and the volcanic ash that overlay the area, we don't have exposure of outcroppings which are usually good signs and targets of exploration. The method employed here was to get into the stream beds which were obviously eroded as far as any of the surrounding area. Samples are then taken and measured for counts of copper parts per million. The areas where the copper counts were significantly higher became areas of interest which then needed to be penetrated. This was done using small drills. These drills were able to give us an indication as to what kind of minerals are present. Information on the geological structure is likewise obtained during the drilling operation.

Larger drills which have the capability of going down about 2000 feet were then brought in. A 400-foot grid pattern was established and we started drilling holes on the 400-foot grid. Gradually, we continued to increase the amount of mineral bearing material and we were able to establish that we did have, in fact, a deposit that was large enough to economically support a mine.

The economics of an operation like this depend on mass-volume efficiency. Large tonnage operations are the only way it can become economic, considering the large infrastructure required and the cost of supporting it.

The bulk of our product - copper concentrate which contains copper, gold and silver - is sold to a consortium of Japanese smelters, a German smelter and Rio Tinto, a Spanish smelter. The bulk of our product is

committed to these customers over a ten-year period. There is very little left for external sales. We had a bit of excess last year. We sold a small quantity to a smelter in Mexico and some to a smelter in Yugoslavia.

We have no control over the price of our product. It is set by the London Metal Exchange. The price we receive for our copper, gold and silver is as quoted on a particular day from the date of delivery of the product to the smelter. So, we are not in any position to negotiate special contracts.

You may ask what are the benefits of our operations to Papua New Guinea. Besides the economic benefit to the country, we have a significant impact on several areas. One area is the training aspect. Because of the requirement for skilled people, we have had an extensive training program from the very beginning. Recognizing that we were going to need a staff of operators in this particular mine, as many Papua New Guineans as possible were trained. We started, way back in the early days of 1968/69, an apprentice training program which was affiliated with the regular Apprentice Board. We are approximately 77% run by Papua New Guineans.

The road that we put in here has been a tremendous advantage to the people of Bougainville. It has opened up the south to the east coast and west coast of Bougainville. It has given people access to markets, thus enabling them to get their cocoa and copra out. It has likewise provided a more reasonable access to the availability of supplies.

We are very heavily engaged in agriculture extension work. We have established a cattle station, a pig breeding station and a vegetable farm. We've likewise been heavily involved in business development from the very beginning. We made people understand that we were in the copper mining business and we didn't particularly want any other business. So, people became involved in developing regular business to support us. Today there are a considerable number of these small businesses.

The final question of course is, "What are the detrimental effects of a large mining project?" I think it is fair to say that when you bring an operation the size of Bougainville Copper into a place like Bougainville, which was relatively under-developed before that time, there is a certain amount of social impact that occurs where you have competition between a sophisticated operation and the villages. This has caused problems from time to time; but I think we now have a reasonable understanding with the community.

The other detrimental part is the tailings or waste disposal, which is a problem in mining. The wasted tailings disposal which is going down the river was developed after consultation and discussion with the government.

The History of the Bougainville Copper Project

It might be worthwhile looking at the profits a little more in detail because of all the publicity and controversy we've had on Bougainville Copper profits. It is an interesting situation where you sometimes mistake profits with cash. I have a few figures here that might be of interest to you.

We have profits of 144.4 million dollars, plus an exchange scale of almost 14 million dollars to give us a consolidated net earnings of 158.4 million. I think it is worthwhile converting this into cash availability. Other items plus the exchange gain which is cash you don't necessarily receive, amount to an additional cash flow of 11.5 million. Add share subscription of 2.5 million to give us in hand 172.4 million dollars in cash.

I will now separate the profits from cash and examine where all the money went. The banks were the first to get their cut of 53.1 million as loan repayments. 17.9 million went to capital expenditure and this will be with us for the life of the mine. 21.2 million is the working capital. This leaves cash available for the dividend of 80.2 million.

This shows how the dividend was paid out. The ownership of Bougainville Copper is such that C.R.A. owns 53.6% of the company, the public owns 26.4% and Papua New Guinea owns 20%. These are share holdings and this is the percentage of dividend that they received from the 80.2 million.

The government received 16 million dollars plus the withholding tax. From the dividend withholding tax, the government received an extra 9.5 million, making a net of 25.5 million dollars. C.R.A's dividend of 43 million dollars, less withholding tax of 6.5 million, left them with 36.5 million dollars and the public ended up with 18.2 million.

CHAIR IN SURVEYING AT THE P.N.G. UNIVERSITY OF TECHNOLOGY

Members will be interested to know that Dr. H.L. Oswal has accepted the position of Professor of Surveying at the University of Technology.

Dr. Oswal arrives in March 1975 to take up his new position. His previous appointment was Dean and Professor of Civil Engineering at the University of Roorkee, India. He began his academic career in 1948 as a Lecturer in Mine Surveying at the Banaras Hindu University.

Dr. Oswal is Indian by birth, and is married with five children.

Academic qualifications are as follows:

- B. Sc (Honours), 1944, College of Science, Nagpur.
- B. Sc (Mining), 1948, Banaras Hindu University.
- Diploma of Photogrammetry, 1954, University College, London.
- Master of Science (Engineering), 1955, University College, London.
- Ph. D (Photogrammetry research), 1970, Roorkee University.

During this period Dr. Oswal has published 27 Research and Technical Papers.

He spent two months earlier this year teaching at the Royal Melbourne Institute of Technology, and was a guest lecturer briefly at the University of New South Wales.

PERSONAL NOTES

Mike Kellock, Fay and the kids will be off to Melbourne shortly. Congratulations to Mike who is to be an assistant registrar at La Trobe University. It was claimed that there was a stiff competition for the post. Colleagues, friends and foes will certainly miss them. SP is definitely losing two loyal and dedicated customers in PNG, but Lae's loss will be Melbourne's gain.

Ian Johnson has recently joined the Overseas Development Ministry of the British Government and is based at the British Embassy in Khartoum. Ian is a special adviser to the Sudanese on development of survey education.

Mick Larmer and Peter Jolley are buying one-way tickets to PNG.

Sue George presented Bob with yet another girl (an error in prediction) weighing $6\frac{3}{4}$ lbs.

Gil Hole and David Forster have actually paid their 1975 subscriptions in advance. A sign of prosperity.

Geoff and Liz Brown recently commenced a Kombi-bus tour of the Continent.

Ivon Bryning was seen leaving in a hurry.

Rumour has it that Tony Quick tried his luck in the stock market. He bought Bougainville Copper shares for a dollar-eighty a share which, unfortunately for Tony, is down to ninety cents. A very sound investment indeed. He is off to New Ireland soon to recoup his losses, one way or the other.

Ossie Dent could be nominated for the title of 'Grandfather of the Year' after arrival of triplets - all girls.

Roy Lea, King of the Dartboard, is pre-occupied in preparing fifty kegs of SP to mark his half century of good living. All the best!

John and Faye Hastings are back from an around-the-world-in-10-days shopping spree. John was seen happily waving a newly acquired mini-calculator from Hong Kong.

SURVEYING EDUCATION CORNER

Mike Kellock, Head of the Surveying Department, produced a valuable document which he submitted in partial fulfilment of the conditions for admission to the Degree of Master of Surveying. The thesis is titled "Design Considerations in Close-Range Mirror-Stereo Photogrammetry". The document proposes the simultaneous production of a stereo-pair of photographs using one camera and one or more mirrors. This will be suited to a number of close-range photogrammetric projects, particularly in the medical and biological field.

The Surveying degree course produced a record number of graduates this year. The Chief Minister, Mr. Michael Somare, spoke to the congregation during the graduation ceremony and excerpts from his address follow:

"This afternoon I wish to discuss briefly what self-government and Independence should mean for graduates, students and staff of the University of Technology. I wish to quote a few figures which should clearly illustrate the future role of the University and its graduates.

At present, one of our scarcest resources is the skilled worker. At this moment, probably more than eighty percent of the executive, managerial and professional positions are held by people not born in this country. This is despite the fact that those not born in Papua New Guinea represent only two percent of our population. Although the figures have improved since, the 1971 census showed that more than seventy percent of the sub-professional positions were also taken by expatriates. This figure represents a real challenge to this University.

Students graduating from this University will be the engineers who will build the roads; others will maintain airports or build and maintain the harbours and jettys that will allow our people to develop their cash crops and other industries to an export level. Other graduates will install and operate new micro-wave equipment for the development of Posts and Telegraphs, or the Department of Civil Aviation.

As I have already pointed out in the figures I quoted earlier, Accountancy and Business Studies graduates, Surveyors, Architects, Cartographers, Valuers, Chemical Technologists and every other type of graduate from this University are not only needed, but are essential for our country and its successful development.

Self-government and the coming of Independence are a challenge to this University and its graduates. The challenge is yours and our people are the beneficiaries. I feel sure that you will make full use of these opportunities."

Congratulations to the department of Surveying graduates who received their degrees, diplomas and certificates.

Surveying Education Corner

The question has been raised a number of times and in various quarters as to whether the length of the course for the first degree in surveying should be raised to contain more training of a practical nature. There have been favourable comments on the idea of including a one-year period of practical work (preferably after the third year) within the normal degree course of a surveyor.

The fundamental issues involved are:

1. Should the University be the place where the under-graduate receives practical training as well as his academic education.

2. Should the practical training be the responsibility of the private sector and government towards their intake of newly-graduated recruits.

3. Should the responsibility be shared between the University, private sector and government.

It is doubtful whether any university-based institution could cover all aspects of practical training needed by the private sector and the government. What the private sector frequently requires is the ability to handle a situation promptly and effectively as this may often be a matter of judgment and experience rather than demanding a high degree of technical knowledge or practical skill.

In most developed countries, it has been long recognized that the academic graduate still requires training of a practical nature, in industry, before he can be considered as an embryonic engineer/surveyor. Large firms require new graduates to undertake apprenticeship during which they would spend two to three months in a variety of sections. However, many of the smaller firms were, understandably, unable or unwilling to organise such planned training and often recruited graduates who had received such training elsewhere with one of the larger organizations.

Any suggestion to extend the length of the surveying degree course for the purpose of providing practical training, in addition to academic instruction, should be thoroughly discussed with representatives of the private sector, the university and the government before being implemented both with regard to the type and extent of the training to be provided.

UNIVERSITY OF TECHNOLOGY GRADUATES - 1974