TAYLOR'S UNIVERSITY

Wisdom · Integrity · Excellence

SCHOOL OF

ARCHITECTURE, BUILDING AND DESIGN

BACHELOR OF QUANTITY SURVEYING (HONOURS)

SITE SURVEYING [QSB 60103]

FIELD WORK REPORT 2

TRAVERSING

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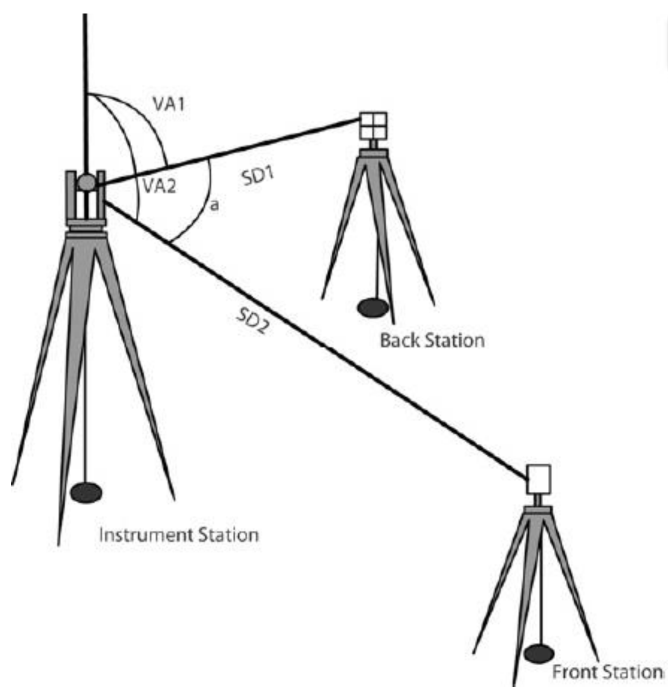
1. INTRODUCTION TO TRAVERSING

WHAT IS TRAVERSING?

Survey is the methodology of modern or local ways of gathering the information about the civil engineering to enable the engineer to make right decision. Traversing is the process of measuring the length and direction of the sides of a traverse to establish a network of both vertical and horizontal control points on or near the ground in the vicinity. A traverse consists of a series of lines, whose lengths and directions are measured, connecting points whose positions are to be determined. The route of the traverse line can be adjusted for obstacles such as rough or timbered terrain, swampy land, buildings and areas of heavy traffic. It is perhaps the most common type of control survey performed by surveyors in private practice or employed by local governmental agencies.

USES OF TRAVERSING

1. Locate topographic detail for the preparation of plans.
2. Lay out the engineering works.
3. Process and order earthwork and other engineering quantities.



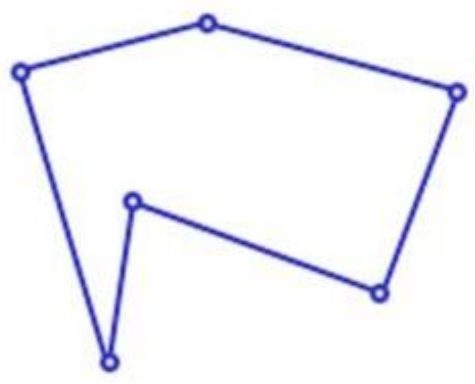
TYPES OF TRAVERSE

1. **CLOSED TRAVERSE** - When the lines form a circuit which ends at the starting point, it is known as closed traverse. The errors during measurement are minimized and adjusted to get accurate data. Closed traverse is the primary method used in checking surveying field work.

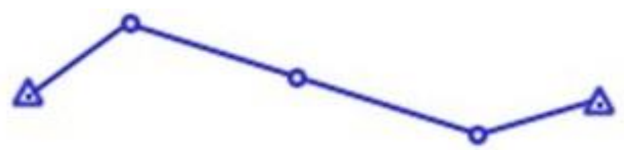
   There are two types of traverse:

   - **Loop Traverse** – a loop traverse starts and ends on a station of assumed coordinates and azimuth without affecting the computations, area or relative position of the stations.
   - **Connecting Traverse** – It looks like an open traverse, except that it begins and ends at points or lines of known position and direction at each of the traverse.

2. **OPEN TRAVERSE** - When the lines form a circuit ends elsewhere except starting point and it does not form a closed polygon, it is said to be an open traverse. This type of traverse is frequently used for preliminary surveys of a long narrow strip of country such as the roads, canals, railways or highways.



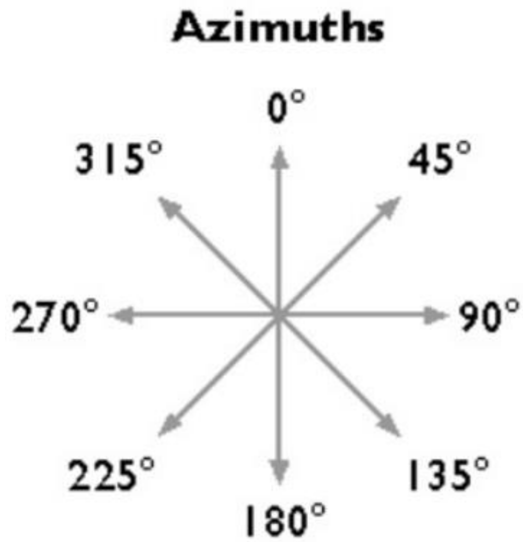
LOOP TRAVERSE



CONNECTING / OPEN TRAVERSE

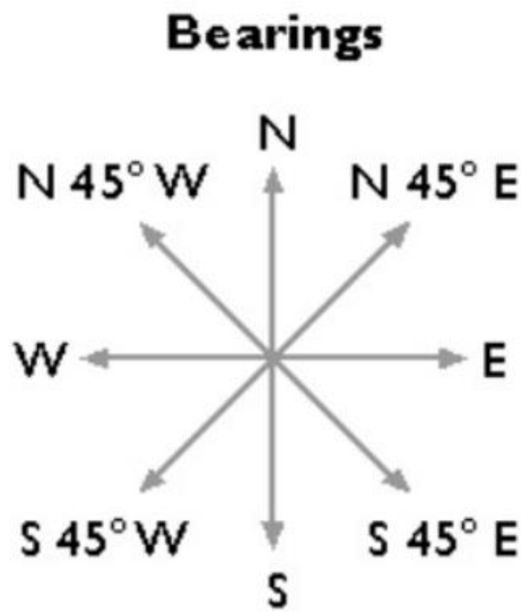
AZIMUTHS

Horizontal angles measured clockwise from a reference meridian with angle between 0^o to 360^o. Generally, the angles should be measured only from north.



BEARINGS

Horizontal angles measured from the meridian either east or west with angle less than 90^o. It can be measured both either clockwise or anticlockwise. It can belong to one of four quadrants: northeast (NE), northwest (NW), southeast (SE), and southwest (SW).



2. OBJECTIVES

- To develop and enhance student’s knowledge in the traversing procedure.
- To enable student to have the knowledge on how to set- up the instrument such as tripod and theodolite using the correct method.
- To enable student to know what precautions must be taken to prevent the instruments being damage.
- To allow student to know the importance of teamwork while carrying the fieldwork in order to complete it on time.
- To allow student to apply theories that have been taught and learnt in class to a hand on situation.
- To ensure student are familiar with the various type and method of traverse surveying used for detailing.
- To allow student identify the error and make some adjustment to the data using the correct formula provided.
- To enable student to learn how to compute the traverse properly and adjust the measured value of a close traverse properly in order to achieve mathematical closure.

3. OUTLINE OF APPARATUS

Theodolite

Theodolite is a most accurate instrument mainly used for measuring both horizontal and vertical angles. It is used in geo-location work and triangulation networks. Besides that, it is also a tool used in land surveying or other engineering industry. Theodolite also consist of a telescope mounted to swivel both horizontally and vertically. The horizontal and vertical axes of the theodolite must be perpendicular to avoid horizontal axis error. However, when the telescope is pointed at the specified object, the angle of the axes can be measured with great precision. The tripod also enable the instrument to rotate 360 degree. The calculation is also based on the principle of trigonometry. Lastly, it can also be used for locating points on line, prolonging survey lines, establishing grades, determining difference in elevation, setting out curves, aligning tunnels etc.

Tripod

A tripod is an instrument that consists of three legged used as platform to support the weight or maintain the stability of the object that are going to place on top of it. A tripod provides stability against downward forces and horizontal forces and movements about horizontal axes. The positioning of the three legs away from the vertical centre allows the tripod better leverage for resisting lateral forces. Most of the modern tripod are made of aluminium although wood can still be used. Aluminium is completely resistant to conditions of high humidity, light-weight, providing convenience when setups are often changed. The feet are either aluminium tripped with a steel point or steel. The tripod is normally placed in the location where it is needed, indicated mark or benchmark. We are required to press down the leg platform of the tripod to secure the leg. After the tripod is secured, the instrument will then place on the flat surface on the head and the mounting screw is used to tighten the instrument when it is correctly positioned to prevent it to fall.

Ranging Rod

A ranging rod also known as the theodolite rod which is a surveying instrument used to mark areas and to set up straight lines in the field. Besides that, it is also used to mark points that must be seen from a distance .Back in the olden days, the rod are made out of bamboo or well-seasoned wood but nowadays it is made out of metallic materials only due to the durability. The total length of the rod is usually 2-3m long and 3cm in diameter. However, it is normally painted alternatively with red and black or white in color.

Plumb Bob

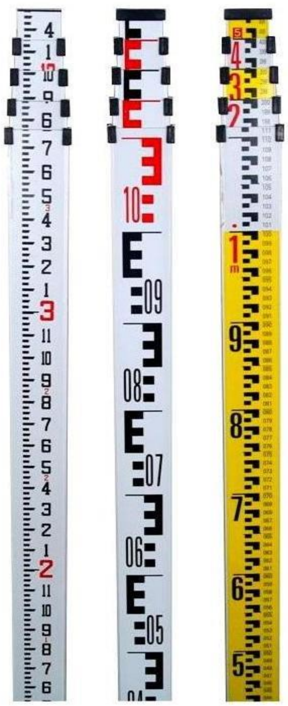
A plumb bob also known as a plummet is an equipment with pointed tip at the bottom or a cone shaped metal weight. It is suspended from a string normally used to establish vertical line or as a sighting references to a surveyor's transit. It is also used in surveying to establish the nadir with respect to gravity of a point in space. In addition, plumb bob is used with the theodolite to set the instrument on the indicated mark or benchmark or to transcribe position onto the ground for placing a new marker. Plumb bob were also used to provide vertical datum in building measurement used for most of the tall structures.

Optical Plummet and Bull's Eye Level

An optical plummet is an instrument used to attach the theodolite to a tripod. The optical plummet are attached with an optical bulls eye bubble. The optical plummet allow instrument to be repeatedly placed in the same position with sub-millimeter precision by just loosening or tighten a locking handle. The plummet is also used to hold the theodolite to prevent it from falling which can caused damage to it. Sometimes, the plummet can also be used to perform angular or distance measurement from the survey point. Tribrach are precision instruments and should be checked regularly to ensure they do not become a source of error. While for the bull's eye level, it is necessary to keep the bubble with the dark circle to ensure that the instrument is levelled. Other than that, accumulative error would occurred if the instrument is not stable before undergo the survey.

Levelling Staff



Levelling staff also called the levelling rod is a levelling instrument that can be made from wood or aluminium. The features of these rod is that it can be extended up to 5 meters vertically. It is used to determine the difference in height between points or height of points above a datum surface. Some rods are graduated on one side only while others are marked on both sides. If marked on both sides, the markings can be identical or can have imperial units on one side and metric on the other. However, without using this instrument the surveyor is unable to get the proper reading of that particular point. In addition, to ensure that the levelling is rod is vertical, we must ensure that the spirit bubble with are within the black circle at all the time.

4. TRAVERSING FIELDWORK

The picture attached above show the area and the location of points of the traversing at Taylor's University Lakeside Campus, Car Park Area.

5. FIELD DATA (1st)

Theodolite Station	Station sighted	Instrument Height	Top stadia readings	Mid stadia readings	Bot Stadia readings
A	**B**	1.38	1.48	1.38	1.28
	D	1.38	1.565	1.38	1.20
B	**A**	1.368	1.468	1.368	1.268
	C	1.368	1.505	1.368	1.235
C	**B**	1.452	1.588	1.452	1.318
	D	1.452	1.536	1.452	1.369
D	**A**	1.428	1.611	1.428	1.245
	C	1.428	1.512	1.428	1.344

Theodolite Station	Station sighted	Vertical Angle from horizontal line		Horizontal Angle from vertical line	
A	**B**	89° 59' 00"	270° 1' 00"	60° 3' 40"	299° 59' 40"
	D	90° 0' 00"	270° 00' 00"		
B	**A**	90° 1' 40"	269° 58' 20"	118° 44' 00"	241°24’20’’
	C	90° 04' 20"	269° 56' 20"		
C	**B**	89° 56' 20"	270° 4' 0"	89° 56' 20"	270° 7' 20"
	D	89° 52' 20"	270° 08' 00"		
D	**A**	90° 0' 0"	269° 59' 40"	91° 15' 40"	268° 38' 40"
	C	90° 6' 40"	269° 53' 20"		

Compute the Angular Error and Adjust the Angles

The sum of all interior angles = (n-2) x 180°

= (4-2) x 180°

= 360°

Station	1st Reading	2nd Reading	Adjustment	Average
A	60°03’40’’	299°59’40’’	360° - 299°59’40’’ = 60°00’20’’	60°02’10’’
B	118°44’00’’	241°24’20’’	360° - 241°24’20’’ = 118°35’40’’	118°39’50’’
C	89°56’20’’	270°07’20’’	360° -270°07’20’’ = 89°52’40’’	89°54’30’’
D	91°15’40’’	268°38’40’’	360° - 268°38’40’’ = 91°21’20’’	91°18’30’’
SUM				359°54’50’’

Total angular error = 360° - 359°54’50’’ = 0°05’10’’

Error per angle = 0°05’10’’ / 4 = 0°01’17.5’’

Station	Average	Correction	Adjusted Angles
A	60°03’40’’	+ 0°01’17.5’’	60°03’17.5’’
B	118°44’00’’	+ 0°01’17.5’’	118°41’07.5’’
C	90°10’20’’	+ 0°01’17.5’’	89°55’47.5’’
D	90°59’20’’	+ 0°01’17.5’’	91°19’47.5’’
SUM			360°00’00’’

Distance A – B

1St reading	
Top	1.48
Mid	1.38
Bot	1.28

2nd Reading	
Top	1.468
Mid	1.368
Bot	1.268

Distance A - B = (K x s x cos^2 (θ)) + (C x cos(θ))

Distance A – B = (100 x (1.48 – 1.28) x cos^2 ($\frac{(90°-89°59') + (270°-270°1')}{2}$)) + (0 x cos(($\frac{(90°-89°59') + (270°-270°1')}{2}$))

Distance A – B = 100 x 0.20 x cos^2(0) + 0

Distance A – B = 20

Distance B – A = (K x s x cos^2 (θ)) + (C x cos(θ))

Distance B – A = (100 x (1.468 – 1.268) x cos^2 ($\frac{(90°-90°01'40'') + (270°-269°58'20'')}{2}$)) + (0 x cos(($\frac{(90°-90°01'40'') + (270°-269°58'20'')}{2}$)))

Distance B – A =100 x 0.20 x cos^2(0) + 0

Distance A – B = 20

Average distance = $\frac{20+20}{2}$ = 20

Distance B – C

1st reading	
Top	1.505
Mid	1.368
Bot	1.235

2nd Reading	
Top	1.588
Mid	1.452
Bot	1.318

Distance B – C = (K x s x $\cos^2$ (θ)) + (C x cos(θ))

Distance B – C = (100 x (1.505 – 1.235) x $\cos^2 \left(\frac{(90°-90°04'20'')+(270°-269°56'20'')}{2}\right)$) + (0 x $\cos\left(\frac{(90°-90°04'20'')+(270°-269°56'20'')}{2}\right)$

Distance B – C = 100 x 0.27 x $\cos^2$(-0°00'20'') + 0

Distance B – C = 26.9999

Distance C – B = (K x s x $\cos^2$ (θ)) + (C x cos(θ))

Distance C – B = (100 x (1.505 – 1.235) x $\cos^2 \left(\frac{(90°-89°56'20'')+(270°-270°04'00'')}{2}\right)$) + (0 x $\cos\left(\frac{(90°-89°56'20'')+(270°-270°04'00'')}{2}\right)$

Distance C – B = 100 x 0.27 x $\cos^2$(-0°00'10'') + 0

Distance C – B = 26.9999

Average distance = $\frac{26.99999 + 26.99999}{2}$ = 26.99999 ≈ 27

Distance C – D

1St reading	
Top	1.536
Mid	1.452
Bot	1.369

2nd Reading	
Top	1.512
Mid	1.428
Bot	1.344

Distance C – D = (K x s x $\cos^2$ (θ)) + (C x cos(θ))

Distance C – D = (100 x (1.48 – 1.311) x $\cos^2 \left(\frac{(90° - 89°52'20'') + (270° - 270°08'00'')}{2}\right)$) + (0 x $\cos\left(\left(\frac{(90° - 89°52'20'') + (270° - 270°08'00'')}{2}\right)\right)$

Distance C – D = 100 x 0.167 x $\cos^2$(-0°00'10'') + 0

Distance C – D = 16.69999 ≈ 16.7

Distance D – C = (K x s x $\cos^2$ (θ)) + (C x cos(θ))

Distance D – C = (100 x (1.464 – 1.297) x $\cos^2 \left(\frac{(90° - 90°06'40'') + (270° - 269°53'20'')}{2}\right)$) + (0 x $\cos\left(\left(\frac{(90° - 90°06'40'') + (270° - 269°53'20'')}{2}\right)\right)$

Distance D – C =100 x 0.168 x $\cos^2$(0°00'00'') + 0

Distance D – C = 16.8

Average distance = $\frac{16.7+16.8}{2}$ = 16.75

Distance D – A

1St reading	
Top	1.611
Mid	1.428
Bot	1.245

2nd Reading	
Top	1.565
Mid	1.381
Bot	1.199

Distance D – A = (K x s x $\cos^2$ (θ)) + (C x cos(θ))

Distance D – A = (100 x (1.564 – 1.197) x $\cos^2$ ($\frac{(90°-90°)+(270°-269°59'40'')}{2}$)) + (0 x cos(($\frac{(90°-90°)+(270°-269°59'40'')}{2}$))

Distance D – A = 100 x 0.366 x $\cos^2$(0°00'10'') + 0

Distance D – A = 36.59999 ≈36.6

Distance A – D = (K x s x $\cos^2$ (θ)) + (C x cos(θ))

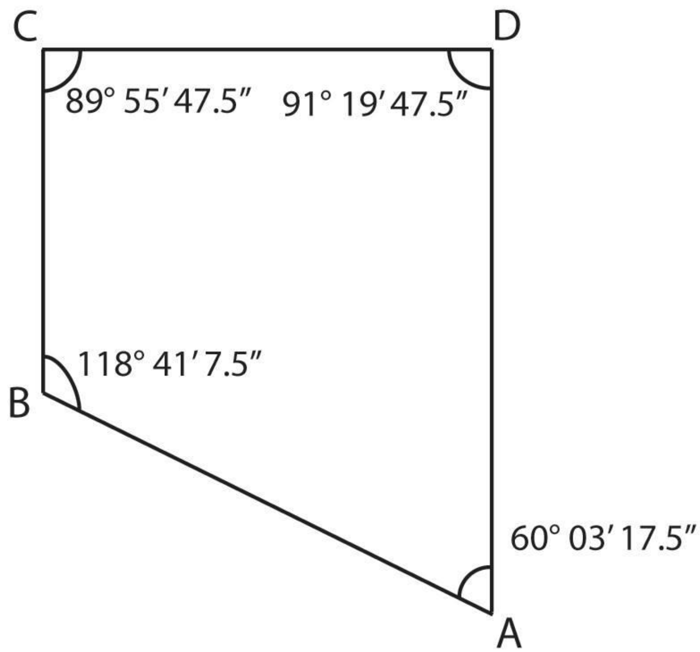
Distance A – D = (100 x (1.564 – 1.197) x $\cos^2$ ($\frac{(90°-90°00'20'')+(270°-269°53'20'')}{2}$)) + (0 x cos(($\frac{(90°-90°00'20'')+(270°-269°53'20'')}{2}$)))

Distance A – D =100 x 0.366 x $\cos^2$(0°3'10'') + 0

Distance A – D = 36.59997

Average distance = $\frac{36.6+36.59997}{2}$ = 36.59999

Compute course azimuth and bearing



Clockwise method

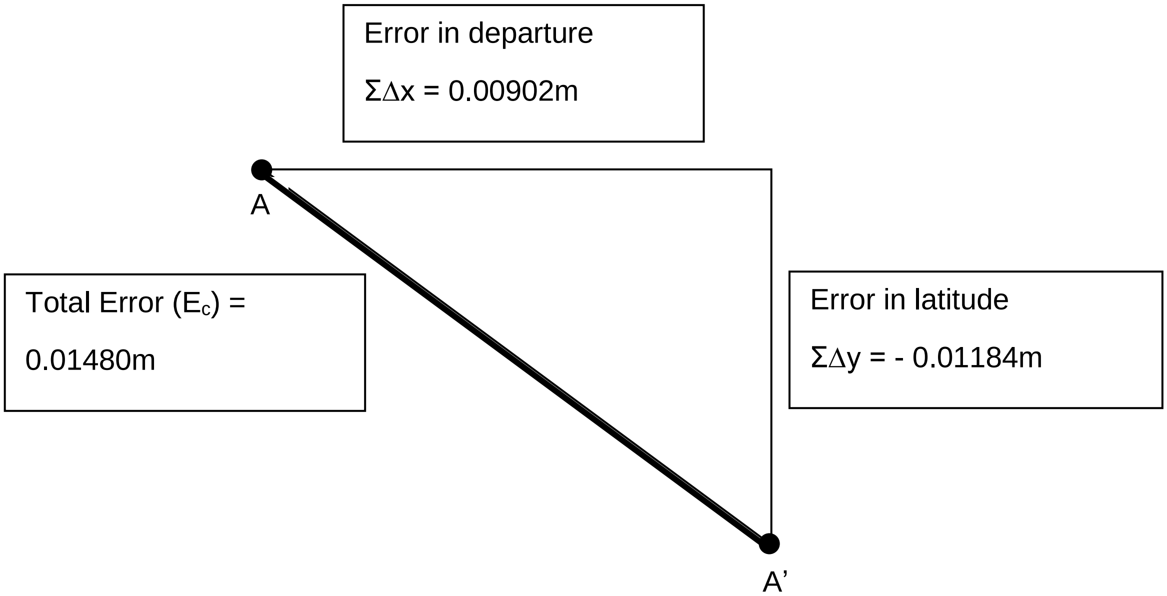
Assumed A-D is at N 00°00'00'' W

	Azimuths	Bearing
D – A	180°00'00''	S 00°00'00'' W
A – B	180° + 180° + 60°3'17.5'' = 299°56'42.5''	N 60°3'17.5'' W
B – C	299°56'42.5'' + 180° - 118°41'7.5'' =1°15'35''	N 1°15'35'' E
C – D	181°15'35'' + 180° - 89°55'47.5'' = 91°19'47.5''	S 60°3'17.5'' E
D – A	91°19'47.5'' + 180° - 91°19'47.5'' = 180°00'00''	S 00°00'00'' W

Computations for Latitude and Departure

			cos β	sin β	L cos β	L sin β
Station	**Bearing, β**	**Length**	**Cosine**	**Sine**	**Latitude**	**Departure**
A						
	S 60°03’17.5’’ E	20	0.499171	0.866504	9.98341	-17.33008
B						
	S 1°15’35’’ W	27	0.999758	0.021985	26.99347	0.5936
C						
	N 88°40’12.5’’ W	16.75	0.023208	0.999731	-0.38873	16.7455
D						
	N 00°00’00’’ W	36.59999	1	0	-36.59999	0
A						
	Perimeter(P) = 100.34999		Sum of latitudes = ΣΔy = - 0.01184			
			Sum of departure = ΣΔx = 0.00902			

Determine the error of the closure



Accuracy = 1 : (P/E_c)

For average land surveying an accuracy of about 1:3000 typical

An accuracy of a least 1:5000 would be required for third-order control traverse surveys

$E_c = \sqrt{(sum\ of\ latitude)^2 + (sum\ of\ departure)^2}$

$E_c = \sqrt{(-0.01184)^2 + (0.00902)^2}$

E_c = 0.01488m

Accuracy = 1 : (100.34999/ 0.01488)

= 1 : 6741.94766

∴ The traversing is acceptable.

Adjust Course Latitude and Departure

Station	Length (m)	Correction	
		Latitude	Departure
A			
	20	0.01184 / 100.34999 x 20 = 0.00236	- 0.00902/ 100.34999 x 20 = - 0.00180
B			
	27	0.01184 / 100.34999 x 27 = 0.00318	- 0.00902/ 100.34999 x 27 = - 0.00243
C			
	16.75	0.01184 / 100.34999 x 16.75 = 0.00198	- 0.00902/ 100.34999 x 16.75 = - 0.00150
D			
	36.59999	0.01184 / 100.34999 x 36.59999 = 0.00432	- 0.00902/ 100.34999 x 36.59999 = - 0.00329
A			
		+ 0.01184	- 0.00902

	Unadjusted		Corrections		Adjusted	
Station	Latitude	Departure	Latitude	Departure	Latitude	Departure
A						
	9.98341	-17.33008	+ 0.00236	- 0.00180	9.98577	-17.33188
B						
	26.99347	0.59360	+ 0.00318	- 0.00243	26.99665	0.59117
C						
	-0.38873	16.74550	+ 0.00198	- 0.00150	-0.38675	16.74400
D						
	-36.59999	0	+ 0.00432	- 0.00329	-36.59567	-0.00329
A						
Σ	-0.01184	0.00902	+ 0.01184	- 0.00902	0.0	0.0
			Check		Check	

Compute station coordinates

Assume that Station A with a latitude of 420 and Station D with a departure of 420

Station	N coordinate(Latitude)	E Coordinate(Departure)	
A	420.00000	419.99671	Start/ return here for Lat. check
	9.98577	-17.33188	
B	429.98577	402.66483	
	26.99665	0.59117	
C	456.98242	403.256	
	-0.38675	16.744	
D	456.59567	420.00000	Start/ return here for Dep. check
	-36.59567	-0.00329	
A	420.00000	419.99671	

6. FIELD DATA (2nd)

Theodolite Station	Station sighted	Instrument Height	Top stadia readings	Mid stadia readings	Bot Stadia readings
A	B	1.484	1.585	1.484	1.384
	D	1.484	1.666	1.484	1.300
B	A	1.448	1.548	1.448	1.348
	C	1.448	1.583	1.448	1.313
C	B	1.501	1.635	1.501	1.365
	D	1.501	1.585	1.501	1.417
D	A	1.473	1.659	1.473	1.290
	C	1.473	1.557	1.473	1.387

Theodolite Station	Station sighted	Vertical Angle from horizontal line		Horizontal Angle from vertical line		
A	B	89° 59' 40"	270° 2' 20"	60° 06' 40"	240° 07' 00"	179° 54' 00"
	D	89° 59' 40"	270° 0' 00"			
B	A	90° 04' 20"	269° 55' 40"	118° 46' 20"	298° 45' 40"	180° 00’ 20’’
	C	90° 00' 00"	269° 59' 40"			
C	B	89° 56' 00"	270° 04' 40"	90° 04' 20"	270° 05' 40"	180° 00 40"
	D	89° 53' 20"	270° 07' 00"			
D	A	90° 00' 40"	269° 59' 40"	91° 13' 20"	271° 13' 20"	180° 00' 20"
	C	89° 59' 40"	270° 00' 00"			

Compute the Angular Error and Adjust the Angles

The sum of all interior angles = (n-2) x 180°

= (4-2) x 180°

= 360°

Station	1st Reading	2nd Reading	3rd Reading	Adjustment	Average
A	60° 06' 40"	240° 07' 00"	179° 54' 00"	240° 07' 00" - 179° 54' 00" = 60° 13’ 00’’	60° 06' 40" + 60° 13’ 00’’ = **60° 09’ 50’’**
B	118° 46' 20"	298° 45' 40"	180° 00’ 20’’	298° 45' 40" - 180° 00’ 20’’= 118°45’ 20’’	118° 46' 20" + 118° 45’ 20’’ = **118° 45’ 50’’**
C	90° 04' 20"	270° 05' 40"	180° 00 40"	270° 05' 40" - 180° 00 40"= 90° 05 ’00’’	90° 04' 20" + 90° 05 ’00’’ = **90° 05’ 10’’**
D	91° 13' 20"	271° 13' 20"	180° 00' 20"	271° 13' 20" - 180° 00' 20" = 91° 13’ 00’’	91° 13' 20" + 91° 13’ 00’’ = **91° 13’ 10’’**
SUM					360° 14’ 00’’

Total angular error = 360° - 360° 14’ 00’’ = 0° 14’ 00’’

Error per angle = 0° 14’ 00’’/ 4 = 0° 03’ 30’’

Station	Average	Correction	Adjusted Angles
A	60° 09’ 50’’	- 0° 03’ 30’’	60° 06’ 20’’
B	118° 45’ 50’’	- 0° 03’ 30’’	118° 42’ 20’’
C	90° 05’ 10’’	- 0° 03’ 30’’	90° 01’ 40’’
D	91° 13’ 10’’	- 0° 03’ 30’’	91° 9’ 40’’
SUM			360°00’00’’

Distance A – B

1St reading	
Top	1.585
Mid	1.484
Bot	1.384

2nd Reading	
Top	1.548
Mid	1.448
Bot	1.348

Distance A - B = (K x s x cos^2 (θ)) + (C x cos(θ))

Distance A – B = (100 x (1.585 – 1.384) x cos^2 ($\frac{(90°-89°59'40'')+(270°-270°2'20'')}{2}$)) + (0 x cos(($\frac{(90°-89°59'40'')+(270°-270°2'20'')}{2}$))

Distance A – B = 100 x 0.201 x cos^2(-0°01'00'') + 0

Distance A – B = 20.1

Distance B – A = (K x s x cos^2 (θ)) + (C x cos(θ))

Distance B – A = (100 x (1.548 – 1.348) x cos^2 ($\frac{(90°-90°01'40'')+(270°-269°58'20'')}{2}$)) + (0 x cos(($\frac{(90°-90°01'40'')+(270°-269°58'20'')}{2}$)))

Distance B – A =100 x 0.20 x cos^2(0) + 0

Distance A – B = 20

Average distance = $\frac{20+20.1}{2}$ = 20.5

Distance B – C

1St reading	
Top	1.583
Mid	1.448
Bot	1.313

2nd Reading	
Top	1.635
Mid	1.501
Bot	1.365

Distance B – C = (K x s x cos^2 (θ)) + (C x cos(θ))

Distance B – C = (100 x (1.583 – 1.313) x cos^2 ($\frac{(90°-90°)+(270°-269°59'40'')}{2}$)) + (0 x cos($\frac{(90°-90°)+(270°-269°59'40'')}{2}$)

Distance B – C = 100 x 0.27 x cos^2(0°00'10'') + 0

Distance B – C = 26.99999

Distance C – B = (K x s x cos^2 (θ)) + (C x cos(θ))

Distance C – B = (100 x (1.635 – 1.365) x cos^2 ($\frac{(90°-89°56'00'')+(270°-269°53'20'')}{2}$)) + (0 x cos($\frac{(90°-89°56'00'')+(270°-2769°53'20'')}{2}$)

Distance C – B = 100 x 0.27 x cos^2(0°05'20'') + 0

Distance C – B = 26.9997

Average distance = $\frac{26.99999+26.99997}{2}$ = 26.99999 ≈ 27

Distance C – D

1st reading	
Top	1.585
Mid	1.501
Bot	1.417

2nd Reading	
Top	1.557
Mid	1.473
Bot	1.387

Distance C – D = (K x s x $\cos^2$ (θ)) + (C x cos(θ))

Distance C – D = (100 x (1.585 – 1.417) x $\cos^2 (\frac{(90° - 89°53'20'') + (270° - 270°07'00'')}{2})$) + (0 x $\cos((\frac{(90° - 89°53'20'') + (270° - 270°07'00'')}{2}))$

Distance C – D = 100 x 0.168 x $\cos^2$(-0°00'10'') + 0

Distance C – D = 16.79999 ≈ 16.8

Distance D – C = (K x s x $\cos^2$ (θ)) + (C x cos(θ))

Distance D – C = (100 x (1.557 – 1.387) x $\cos^2 (\frac{(90° - 90°07'00'') + (270° - 269°53'20'')}{2})$) + (0 x $\cos((\frac{(90° - 90°07'00'') + (270° - 269°53'20'')}{2}))$

Distance D – C =100 x 0.17 x $\cos^2$(-0°00'10'') + 0

Distance D – C = 16.99999 ≈ 17

Average distance = $\frac{16.8+17}{2}$ = 16.9

Distance D – A

1St reading	
Top	1.659
Mid	1.473
Bot	1.290

2nd Reading	
Top	1.666
Mid	1.484
Bot	1.300

Distance D – A = (K x s x $\cos^2$ (θ)) + (C x cos(θ))

Distance D – A = (100 x (1.659 – 1.290) x $\cos^2$ ($\frac{(90°-90°)+(270°-269°59'40'')}{2}$)) + (0 x cos(($\frac{(90°-90°)+(270°-269°59'40'')}{2}$))

Distance D – A = 100 x 0.369 x $\cos^2$(0°00’10’’) + 0

Distance D – A = 36.89999 ≈ 36.9

Distance A – D = (K x s x $\cos^2$ (θ)) + (C x cos(θ))

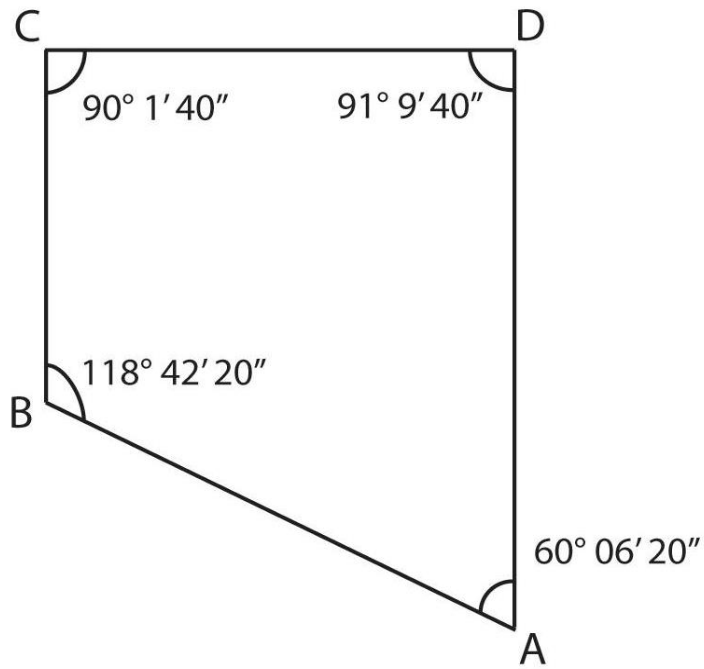
Distance A – D = (100 x (1.666 – 1.300) x $\cos^2$ ($\frac{(90°-89°59'40'')+(270°-270°0'0'')}{2}$)) + (0 x cos(($\frac{(90°-89°59'40'')+(270°-270°0'0'')}{2}$)))

Distance A – D =100 x 0.366 x $\cos^2$(0°0’10’’) + 0

Distance A – D = 36.59999 ≈ 36.6

Average distance = $\frac{36.9+36.6}{2}$ = 36.75

Compute course azimuth and bearing



Counter clockwise method

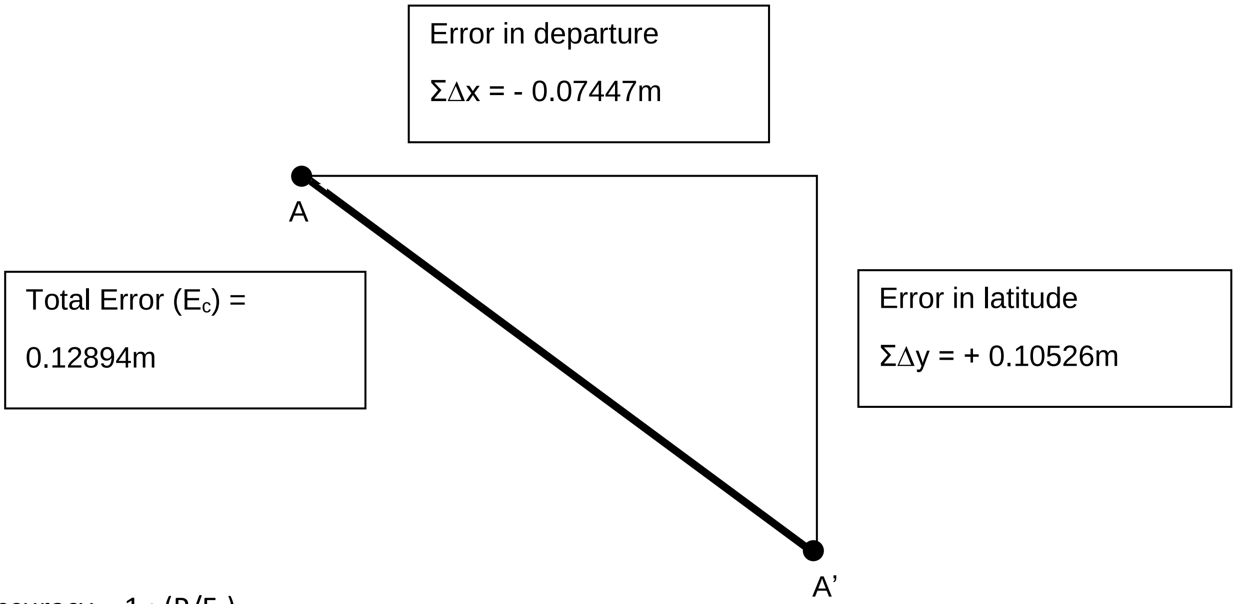
Assumed A-D is at N 00°00’00’’ W

	Azimuths	Bearing
A – D	00°00’00’’	N 00°00’00’’ W
D – C	180° + 91°09’40’’ = 271°09’40’’	N 88°50’20’’ W
C – B	271°09’40’’ – 180° + 90°01’40’’ =181°11’20’’	S 01°11’20’’ W
B – A	181°11’20’’ – 180° + 118°42’20’’ = 119°53’40’’	S 60°6’20’’ E
A – D	119°53’40’’ + 60°6’20’’ + 180° - 360° = 00°00’00’’	N 00°00’00’’ W

Computations for Latitude and Departure

			cos β	sin β	L cos β	L sin β
Station	**Bearing, β**	**Length**	**Cosine**	**Sine**	**Latitude**	**Departure**
A						
	S 60°06’20’’ E	20.05	0.498404	0.866945	-9.993	17.38225
B						
	S 1°11’20’’ W	27	0.999785	0.020749	-26.9942	-0.56022
C						
	N 88°50’20’’ W	16.9	0.020264	0.999795	0.34246	-16.8965
D						
	N 00°00’00’’ W	36.75	1	0	36.75	0
A						
	Perimeter(P) = 100.70		Sum of latitudes = ΣΔy = + 0.10526			
			Sum of departure = ΣΔx = - 0.07447			

Determine the error of the closure



Accuracy = 1 : (P/E_c)

For average land surveying an accuracy of about 1:3000 typical

An accuracy of a least 1:5000 would be required for third-order control traverse surveys

$E_c = \sqrt{(sum\ of\ latitude)^2 + (sum\ of\ departure)^2}$

$E_c = \sqrt{(0.10526)^2 + (-0.07447)^2}$

E_c = 0.12894m

Accuracy = 1 : (100.70/ 0.12894)

= 1 : 780.983

∴ The traversing is **not acceptable** and need to remeasure to reduce random error

∴ **The adjustment for latitude and departure cannot be done** as the reading is not reach the minimum of accuracy.

7. Comparison of distance using auto level and theodolite

Auto level Station	Station sighted	Top stadia readings	Mid stadia readings	Bot Stadia readings	Distance (m)
A	**B**	1.473	1.373	1.273	20.0
	D	1.555	1.371	1.190	36.5
B	**A**	1.448	1.349	1.249	19.9
	C	1.513	1.378	1.243	27.0
C	**B**	1.448	1.313	1.178	27.0
	D	1.400	1.317	1.233	16.7
D	**A**	1.500	1.319	1.135	36.5
	C	1.435	1.352	1.269	16.6

Distance	Distance counted using auto level	Average distance (m)	Distance of 1st data using theodolite	Distance of 2nd data using theodolite
A - B	20.0	19.95	20	20.05
	19.9			
B - C	27.0	27	27	27
	27.0			
C - D	16.7	16.65	16.75	16.9
	16.6			
D - A	36.5	36.5	36.59999	36.7
	36.5			

8. GRAPH

9. DISCUSSION

After compiling our results, we have discussed our data with other groups. In the discussion, we were able to point out certain elements that have affected our data. These elements can help increase the accuracy of our data.

1. **Distance**
   - In certain cases, if the points were set up too close within each other, accuracy may decrease drastically. This means that the points must be farther away from each other to maximize the accuracy.

2. **Accuracy**
   - In our 2nd reading, we had a major human error where all 3 crosshairs (top, middle & bottom) were increased by 0.03. This caused our distance to be increased by 0.3.
   - This can be prevented by taking multiple readings at the same station.

3. **Sunlight**
   - Certain groups which were doing their surveying in the morning/afternoon got higher accuracy as the crosshairs were slightly more visible

4. **Angle**
   - When the angle of the instrument and staff is around 90°, there is a slight chance where the accuracy would be less than when the angle is slightly higher.

5. **Cross-checking**
   - Upon consultation, we were told by our lecturer to use an auto-level to calculate the distance. This can help cross-check with our distance to maintain accuracy.

10. CONCLUSION

From this fieldwork, we were given a chance to use the theodolite. The theodolite is a very complex instrument which require multiple tries to be able to set-up. It has affected our group the first time we were using it as it was late in the evening. However, after using the theodolite for multiple times we have been able to get the hang of it and was able to set-up in a short amount of time.

In our first reading, we had some error in our data for our station C & D. This has also affected our data and thus we had to redo our readings for said stations.
We also found out the use of the theodolite and how it can be very useful for calculating area. This was a very great experience for all of us as we were able to take readings from it and compute our data to find the angles.