University of Toronto
Department of Civil Engineering

GULL LAKE SURVEY CAMP

CAMP NOTES

2009

SURVEYING

INTRODUCTION:

As stated by Dr. Adam Chrzanowski, "Surveying is the field of applied science and engineering that deals with spatial information about positions of points on, above or below the earth's surface. The spatial information may be displayed as maps, digital terrain models, or just a list of coordinates of the points of interest in a selected coordinate system."

The purpose of this course is to learn; how to make field notes, basic surveying procedures and the instruments used to perform these procedures, survey calculations that apply to the procedures and how the survey procedures apply to mining.

Main Types of Surveys
Plane surveying is that type of surveying in which the surface of the earth is considered to be a plane for all X and Y dimensions. All Z dimensions (height) are referenced to the mean spherical surface of the earth (mean sea level).
Geodetic Surveying is that type of surveying in which the surface of the earth is considered to be spherical (actually an ellipsoid of revolution) for X and Y dimensions. The Z dimensions (height) are referenced to the mean surface of the earth (msl).

Classes of Surveys
Preliminary: to locate and gather survey data, physical features and vertical information.
Layout: marking on the ground the features shown on the design plan ie; roads, pipelines.
Control: used to reference preliminary and layout surveys. Horizontal control is tied to the property boundaries or other horizontal control points and vertical control is determined by benchmarks with known elevation amsl.

Surveys - Definitions
Topographic surveys: preliminary surveys used to tie in the natural and man-made surface features of an area, related relative to one another and a control framework.
Hydrographic surveys: preliminary surveys that are used to tie in underwater features to a surface control line, ie; shorelines, marine features, water depths, etc.
Route surveys: preliminary, layout and control surveys of highways, railways and transmission lines, etc.
Property surveys (Cadastral surveys): preliminary, layout and control surveys that determine existing property boundaries or create new ones.
Aerial Surveys:photogrammetric techniques are employed to convert the aerial photographs into scale maps and plans.
Final or "As Built" Surveys: final surveys to provide a final record of location to make sure it is according to plans.
Mining Surveys: preliminary, layout and control surveys that are used in underground mining, tunneling and open-pit mining.
Global Positioning System (GPS) Surveys: positioning surveys in which the coordinates (northings, eastings and elevations) of survey stations are determined by NAVSTAR satellites and receivers.

Evolution of Surveying

- historical records dating back almost 5000 years show evidence of surveyors in China, India, Babylon and Egypt - mostly for property boundaries
- surveyors used ropes with knots tied at set gradations to measure distances
- ropes used to lay out right angles using 3-4-5 method
- crude, but accurate levels used by Egyptians and Romans (e.g., aqueducts)

in mid 1500s

- surveyor's chain (1 chain = 100 links = 66 ft) first used in the Netherlands
- term theodolite used to describe an instrument, graduated in 360°, to measure angles

1609

- telescope

first half of 1700s

- dumpy levels developed - combine telescope with a bubble level - measure differences in elevation

1831

- American repeating transit - telescope revolves (transits) on its axis

early 1900s

- theodolite much improved

1950s and 1960s

- European and Japanese repeating instruments with micrometer or scale readouts and optical plummets

1950s

- electronic distance measurements (EDM) for control-survey distance measurements

after WWII

- aerial surveys - large-scale topographic surveys

late 1980s

- total stations - electronic data collection of angles, distances and descriptive data with transfer to a computer and plans drawn by digital plotter

today

- Global Positioning System (GPS)

Survey Geographic Reference

- most broad reference system defined by lines of latitude and longitude
- latitude - east/west lines parallel to the equator
- longitude - north/south lines converging at poles; 0° longitude is arbitrarily placed through Greenwich, England
- latitude/longitude not used for plan surveying
- plane surveying uses coordinate grid system or original township fabric as reference basis

Survey Grid Reference

- all Provinces and States use a grid system best suited to their needs; grid limited in size so no serious errors accumulate due to curvature of the earth
- advantages include ease of calculation
- grids are referenced to geographic reference so that translation can be done as needed

Survey Legal Reference

- Canada and US Townships were originally laid out in 6-mile squares; a wide variety of patterns emerged, reflecting English/French heritage
- Townships subdivided into sections and concessions
- sections subdivided into real estate developments
- all developments referenced to the original Township fabric

Survey Vertical Referencing

- by international agreement, mean seal level (MSL) is defined and used
- actual sea level varies, which is why the definition is required
- benchmarks are available in most areas for survey use

Units of Measurement

- Imperial (foot) system was used (still used in the U.S. and Canada to deal with old surveys)
- SI (System International) units are used for new work in Canada (metric)

1.1 UNITS OF MEASUREMENT

Although the foot system of measurement has been in use in the United States from the early settler days up to the present, the metric system has been making steady inroads. The Metric Conversion Act of 1975 made conversion to this system largely voluntary, but subsequent amendments and government actions made the use of the metric system mandatory for all federal agencies as of September 1992. By January 1994, the metric system was required in the design of all federal facilities; additionally, many states' Departments of Transportation have already begun switching to the metric system for field work and highway design.

Table 1.1 MEASUREMENT DEFINITIONS AND EQUIVALENCIES

Linear measurements			Foot units		
1 mile	=	5280 feet	1 foot	=	12 inches
	=	1760 yards	1 yard	=	3 feet
	=	320 rods	1 rod	=	$16\frac{1}{2}$ feet
	=	80 chains	1 chain	=	66 feet
			1 chain	=	100 links
1 acre	=	43,560 ft^2 = 10 square chains			

Linear measurement		Metric (SI) units
1 kilometer	=	1,000 meter
1 meter	=	100 centimeter
1 centimeter	=	10 millimeter
1 decimeter	=	10 centimeter
1 hectare (ha)	=	10,000 m^2
1 square kilometer	=	1,000,000 m^2
	=	100 hectares

Foot-to-metric conversion		
1 ft =	0.3048 m (exactly)	1 inch = 25.4 mm (exactly)[a]
1 km =	0.62137 miles	
1 hectare (ha) =	2.471 acres	
1 km^2 =	247.1 acres	

[a] Prior to 1959, the United States used the relationship 1 m = 39.37 in. This resulted in a U.S. survey foot of 0.3048006 m (approx.).

Angular measurement	
1 revolution	= 360°
1 degree	= 60′
1 minute	= 60″ (seconds)

Precision and Accuracy

Precision: the degree of perfection **used** in the survey. The level of refinement and care when performing the survey. Can be a function of the equipment used. In a series of independent measurements of the same quantity, the closer each measurement is to the average value, the better is the precision.
Accuracy: the degree of perfection **obtained** in the survey. In a series of independent measurements the closer the average measurement is to the true value is the accuracy.

Example:
Three survey crews are asked to measure a known distance of 300m, five times.

	Crew one	Crew two	Crew three
	295.902	299.980	299.997
	295.908	300.060	299.999
	295.905	299.955	300.002
	295.907	300.020	299.998
	295.903	299.990	300.003
Mean	295.905	300.001	300.000
Error	4.905	0.001	0.000

Crew one: precise but not accurate
Crew two: accurate but not precise
Crew three: precise and accurate

Accuracy Ratio
- ratio of the error in a measurement to the actual measurement (relative accuracy)
- used to give a relative measure of how well a measurement was made
- there are specific guidelines for accuracy ratios for different levels of surveying
- may be specified by a client or employer
- equipment and methods of surveying must be chosen to ensure the required accuracy ratio (or better) is achieved
- ratio is always rounded to the nearest 1/100th

- ratio of error to actual distance
  ex. 4.905/295.905=1/60.3=1/60
  ex. 0.001/300.001=1/300 001=1/300 000

Table A.9 TRAVERSE

Classification	First order	Second order Class I	Second order Class II	Third order Class I	Third order Class II
Recommended spacing of principal stations	Network stations 10–15 km. Other surveys seldom less than 3 km.	Principal stations seldom less than 4 km except in metropolitan area surveys where the limitation is 0.3 km.	Principal stations seldom less than 2 km except in metropolitan area surveys where the limitation is 0.2 km.	Seldom less than 0.1 km in tertiary surveys in metropolitan area surveys. As required for other surveys.	
Horizontal directions or angles					
Instrument	0".2	0".2 or 1".0	0".2 or 1".0	1".0	1".0
Number of observations	16	8 or 12[a]	6 or 8[a]	4	2
Rejection limit from mean	4"	4" or 5"	4" or 5"	5"	5"
Length measurements					
Standard error	1 part in 600,000	1 part in 300,000	1 part in 120,000	1 part in 60,000	1 part in 30,000
Reciprocal vertical angle observations					
Number of and spread between observations	3D/R-10"	3D/R-10"	2D/R-10"	2D/R-10"	2D/R-20"
Number of stations between known eleva-	4–6	6–8	8–10	10–15	15–20

Table A.9 TRAVERSE

Classification	First order	Second order Class I	Second order Class II	Third order Class I	Third order Class II
Astro azimuths					
Number of courses between azimuth checks	5–6	10–12	15–20	20–25	30–40
Number of observations/night	16	16	12	8	4
Number of nights	2	2	1	1	1
Standard error	0".45	0".45	1".5	3".0	8".0
Azimuth closure at azimuth checkpoint not to exceed	1".0 per station or 2" $\sqrt{N}$	1".5 per station or 3" $\sqrt{N}$. Metropolitan area surveys seldom to exceed 2".0 per station or 3" $\sqrt{N}$.	2".0 per station or 6" $\sqrt{N}$. Metropolitan area surveys seldom to exceed 4".0 per station or 8" $\sqrt{N}$.	3".0 per station or 10" $\sqrt{N}$. Metropolitan area surveys seldom to exceed 6" per station or 15" $\sqrt{N}$.	8" per station or 30" $\sqrt{N}$.
Position closure					
After azimuth adjustment	0.04 m $\sqrt{K}$ or 1:100,000	0.08 m $\sqrt{K}$ or 1:50,000	0.2 m $\sqrt{K}$ or 1:20,000	0.4 m $\sqrt{K}$ or 1:10,000	0.8 m $\sqrt{K}$ or 1:5000

May be reduced to 8 and 4, respectively, in metropolitan areas.

Errors

Systematic Errors: are those errors whose magnitude and algebraic sign can be determined. This allows the surveyor to eliminate them from measurements and thus improve the accuracy. Eg. the effect of temperature on the shortening or lengthening of a steel tape.

Random Errors: also known as accidental errors are associated with the skill and vigilance of the surveyor. Random errors tend to cancel themselves out, eg. one measurement is too long, the next is too short.

Blunders: mistakes made in measurements by human error, eg. transposing figures. Mistakes must be discovered and eliminated. As a rule, **every** measurement is checked or repeated. This also increases the precision of the measurement.

Stationing

- measurements often taken along a baseline and perpendicular to a baseline
- distances along baseline referred to as stations (or chainages for historical reasons)
- beginning of baseline is the zero end
  - in imperial, denoted 0+00 (e.g., 1+00 is 100' from the beginning)
  - a point 131.26' from the beginning is 1+31.21
- metric (SI) still uses 100 unit stations (e.g., a point 376.27 m from the beginning is STN 3+76.27)
- some highway agencies us a 1,000 unit stations (e.g., a point 6,747.36 m from the beginning is STN 6+747.36

Types of Measurements

distances
angles
levels

- all measurements referenced to some defining datum
- e.g., gravity is a reference for vertical; 90° to vertical is level reference
- techniques developed for each type of measurement to eliminate systematic errors and provide better results

Equipment

- tape, chaining pin, plumb bob, range pole, hand level
- transit, theodolite, 2nd order
- total station
- level 3rd and 2nd order, rod, split bubble
- GPS

DISTANCE MEASUREMENTS

Types of Distance measurements (History)

Pacing: useful but not accurate, sufficient for locating survey markers or for rough checks in construction layout, useful for finding midway distances when leveling.
Odometer: automobile odometers can be used to locate lot lines or fence lines in rural areas from known intersections.
Measuring Wheel: a bicycle tire calibrated per revolution. Contractors might use it to check lengths of curbs or for checking pay quantities.
Electronic distance measurement (EDM): function by sending light waves or microwaves to a prism which reflects them back to the instrument, distance is calculated by measuring the phase difference between the transmitted and reflected signals. Now total stations are used.
Stadia: uses a level or transit (theodolite) and a graduated rod. The interval between the upper and lower crosshair reading times a stadia constant (usually 100) gives distance. It was used for topographic surveys until the advent of the total station. It is useful for measuring irregular areas such as an open pit mine or underground room (stope).
Taping: before tapes were developed the device of measurement was Gunter's chain. (It was popular in the time of settlement of North America). It was a chain 66 feet long consisting of 100 links. Listed in units of measurement. Now tapes are made of steel or stylon (nylon covered steel) and come in 100 ft or 30m lengths.

Taping

- determine distance between field points
- establish points in the field

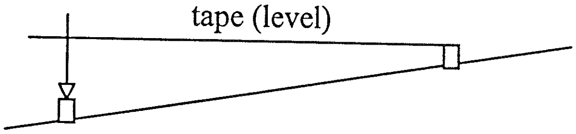
Standard Conditions

	Imperial	Metric
temperature	68°F	20°C
fully supported	yes	yes
tension	10 lb	50 N
		1 lb force = 4.448 N

Taping Accessories

plumb bob

- when suspended by string with gravity as reference and point to centre of earth
- used to transfer positions from tape to ground an vice versa when tape being held off ground to maintain its horizontal (level) alignment (horizontal plane)



chaining pins

- used to set intermediate marks on the ground
- set in at 45° to the ground, perpendicular to the tape alignment

hand level

- used to keep tape level

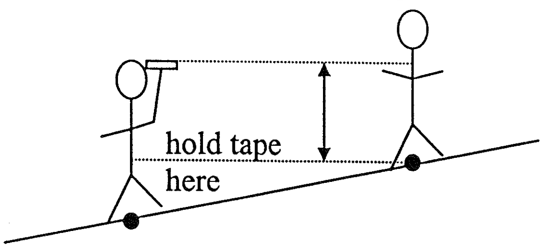
range poles

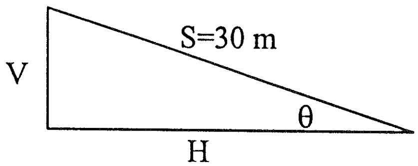
- used to give alignment
- sight along the tape so that tape is in line with pole

NOTE: except for slope taping, all taping is done with the tape as horizontal as possible; always tape in both directions and average

Procedure for Taping on a Slope

- person at lowest elevation sights other person
- estimate difference in elevation from where you sight to the point where they will hold the tape
- go down the same distance from your line of sight and hold the tape at that elevation - level line
- try to get within 0.1 m accuracy
- if chaining downhill, use plumb bob to identify point on ground
- if chaining uphill, use plumb bob to stay on point and say "mark" when aligned





$$\sin\theta = \frac{V}{S} = \frac{0.1\,\text{m}}{30\,\text{m}};\ \theta = 0.191^\circ$$

$H = S\cos\theta = 30\cos(0.191^\circ)$

$H = 29.9998$ m

V	H	error
0.1 m	29.9998 m	0.2 mm
0.5 m	29.9958 m	4.2 mm

Taping Errors

systematic

- magnitude and sign can be determined
- can be eliminated
- e.g., temperature effect on length of steel tape

random

- accidental - reflects level of skill
- cancel out

blunders

- human error
- do not cancel out - must be found

e.g., transposing numbers

Taping Corrections

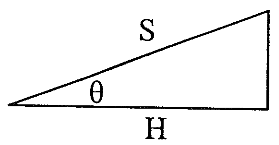
- can correct for systematic errors (i.e., standard conditions not met)

Systematic Errors

1. slope
2. incorrect tape length
3. temperature
4. sag tension

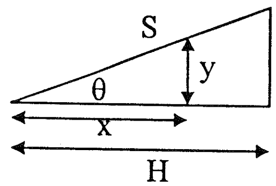
1. Slope Corrections

slope angle - given slope distance (S) and slope angle (θ), find H



$$H = S\cos\theta$$

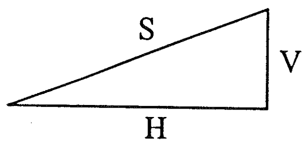
slope gradient - given slope distance (S) and gradient m=y/x=rise/run, find H



$$\tan\theta = y/x = m$$
$$\theta = \tan^{-1}(y/x) = \tan^{-1}(m)$$

$$H = S\cos\theta$$

vertical distance - given slope distance (S) and vertical distance (V), find H



$$S^2 = H^2 + V^2$$
$$H = \sqrt{S^2 - V^2}$$

2. Incorrect Tape Length

- tapes become stretched, kinked, spliced

3. Temperature Correction

$$C_t = a(T - T_s)L$$

T_s=standard temperature; T=temperature; L=length (distance measured); a=thermal coefficient of expansion

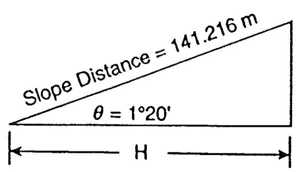
4. Sag Correction

$$C_s = -\frac{w^2L^3}{24P^2} = -\frac{W^2L}{24P^2}$$

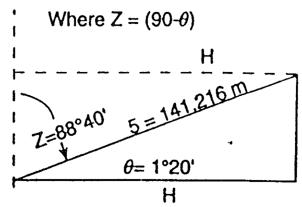
w=weight of tape per unit length; W=weight of tape between two supports; L=length of tape; P=applied tension (10 lb or 50 N)

■ **EXAMPLE 2.3** *Slope Corrections*

(a) Slope angle: Given the slope distance *(S)*, and slope angle (θ), or Zenith angle (90-θ)



or



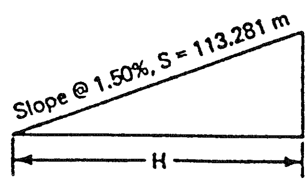
$$\frac{H \text{ (horizontal)}}{S \text{ (slope)}} = \cos \theta \qquad\qquad \frac{H}{S} = \text{Sin Z}$$

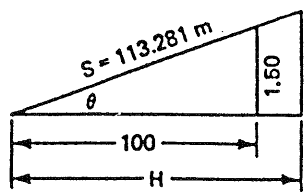
$H = S \cos \theta$ [Eq.(2.1)] $\qquad H = \text{S} \sin \text{Z}$ [Eq.(2.1a)]

$= 141.216 \cos 1°20'$ $\qquad = 141.216 \sin 88°40'$

$= 141.178$ m $\qquad = 141.178$ m

(b) Slope gradient: Given the slope distance and gradient (slope)





for every hundred rise 1.5

ratio $\frac{1.5}{100}$

$= \frac{\text{opp}}{\text{adj}}$ or tan

$$1.50/100 = \tan \theta$$

$$\theta = 0.85937°$$

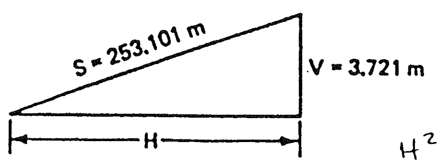
cos θ = $\frac{\text{adj}}{\text{hyp}}$

Horizontal / Slope

$$\frac{H}{113.281} = \cos 0.85937°$$

$$H = 113.268 \text{ m}$$

(c) Slope and vertical distance: Given the slope distance and difference in elevation *(V)*



$H^2 = S^2 - V^2$

$H = \sqrt{S^2 - V^2}$

$= 253.074$

■ **EXAMPLE 2.10**

A tape is weighed and found to be 1.95 lb. The overall length of the 100-ft tape (end to end) is 102 ft. The specific weight of steel is 490 lb/ft³.

$$\frac{102 \text{ ft} \times 12 \text{ in.} \times \text{area (in.}^2)}{1{,}728 \text{ in.}^3} \times 490 \text{ lb/ft}^3 = 1.95 \text{ lb}$$

$$\text{Area} = \frac{1.95 \times 1728}{102 \times 12 \times 490} = 0.0056 \text{ in.}^2$$

Tension errors are usually quite small and as such have relevance only for very precise surveys. Even for precise surveys it is seldom necessary to calculate tension corrections, as availability of a tension spring balance allows the surveyor to apply standard tension and thus eliminate the necessity of calculating a correction.

2.13.3 Sag Corrections

■ **EXAMPLE 2.11**

$$C_s = \frac{-w^2L^3}{24P^2} = \frac{-W^2L}{24P^2}$$

where $W^2 = w^2L^2$

w = weight of tape per unit length

W = weight of tape between supports

L = length of tape between supports

A 100-ft steel tape weighs 1.6 lb and is supported only at the ends with a force of 10 lb. What is the sag correction?

Solution

$$C_s = \frac{-1.6^2 \times 100}{24 \times 10^2} = -0.11 \text{ ft}$$

If the force were increased to 20 lb, the sag is reduced to

$$C_s = \frac{-1.6^2 \times 100}{24 \times 20^2} = -0.03 \text{ ft}$$

■ **EXAMPLE 2.12**

Calculate the length between two supports if the recorded length is 42.071 m, the mass of this tape is 1.63 kg, and the applied tension is 100 N.

Solution

$$C_s = \frac{-(1.63 \times 9.807)^2 \times 42.071}{24 \times 100^2}$$

$$= -0.045$$

Therefore, the length between supports = 42.071 − 0.045 = 42.026 m.

Ordinary taping precision is referred to as being that which can result in 1/5,000 accuracy. The techniques used for ordinary taping, once mastered, can easily be maintained. It is possible to achieve an accuracy level of 1/5,000 with little more effort than is required to

Table 2.2 SPECIFICATIONS FOR 1/5,000 ACCURACY

Source of error	Maximum effect on one tape length	
	100 ft	30 m
Temperature estimated to closest 7°F (4°C)	±0.005 ft	±0.0014 m
Care is taken to apply at least normal tension (lightweight tapes) and tension is known within 5 lb (20 N)	±0.006 ft	±0.0018 m
Slope errors are no larger than 1 ft/100 ft or 0.30 m/30 m	±0.005 ft	±0.0015 m
Alignment errors are no larger than 0.5 ft/100 ft or 0.15 m/30 m	±0.001 ft	±0.0004 m
Plumbing and marking errors are at a maximum of 0.015 ft/100 ft or 0.0046 m/30 m	±0.015 ft	±0.0046 m
Length of tape is known within ±0.005 ft (0.0015 m)	±0.005 ft	±0.0015 m

In the foregoing example, it is understood that corrections due to systematic errors had already been applied.

course	measured distances	sum	mean distance	accuracy	remarks
1-2	102.345				
2-1	102.363		102.354		
~~2-3~~	~~97.545~~				
3-2	97.832				
2-3	97.846		97.839		
3-4	120.000				
	25.000				
	30.000				
	30.000				
	2.789	207.789			
4-3	207.794	207.794	207.792		

$$\text{accuracy ratio} = \frac{|\text{distance 1} - \text{average distance}|}{\text{average distance}} = \frac{|\text{distance 2} - \text{average distance}|}{\text{average distance}}$$

LEVELING

Leveling is the procedure used to determine differences in elevation between points that are remote from each other.
An **elevation** is a vertical distance above or below a reference datum which in surveying is mean sea level (msl).
A **vertical line** is a line from the surface of the earth to the earth's center. It can be referred to as a plumb line or a line of gravity.
A **level line** is a line in a level surface. A **level surface** is a curved surface parallel to the mean surface of the earth.

A **Level** is a cross-hair equipped telescope and an attached spirit level tube which is mounted on a sturdy tripod. If the level is set up properly, the telescope which revolves around the vertical axis will give a line of sight over the horizontal cross-hair that defines a horizontal plane.
Differential leveling or spirit leveling, is used to determine differences in elevation between remote points using a level and a graduated rod.
Curvature and refraction: When considering the divergence between level and horizontal lines, note that all sight lines are refracted downward by the earth's atmosphere. C and R errors are relatively insignificant for differential leveling because the sight distances are seldom greater than 60 m.
$(c + r)_m$ for 30 m is 0.0001m
Terms used in leveling:
Benchmark (BM) is a permanent point of known elevation They are established using precise leveling techniques and are placed on stable structures. Local municipalities publish BM information. The location of the benchmark is always found by a written description rather than a location sketch.
Temporary Benchmark (TBM) is a semipermanent point of known elevation established from an known BM. It is used for convenience on the site to transfer elevations for different stages of a construction project.
Turning Point (TP) is a point temporarily used to transfer an elevation. It should be solid enough that the elevation will not change between rod readings.
Backsight (BS) is a rod reading taken on a point of known elevation (BM). It is added to the elevation of the BM to establish the height of instrument (HI).
Height of Instrument (HI) is the elevation of the line of sight through the level.
Foresight is a rod reading taken on a turning point, benchmark or temporary benchmark which is subtracted from the height of instrument (HI) to determine elevation.
Intermediate foresight (IS) is a rod reading taken on any point where an elevation is required. It is used in many engineering leveling projects such as profiles or cross-sections or in open pit mining surveys where specific ground elevations would be needed to calculate volumes of material.
Waving the rod or rocking the rod allows the instrument person to take the lowest **rod reading**. The lowest rod reading will occur when the rod is plumb or when the rod face and the line of sight are perpendicular.
Leveling loop starts at a BM of known elevation and returns back to the same BM.

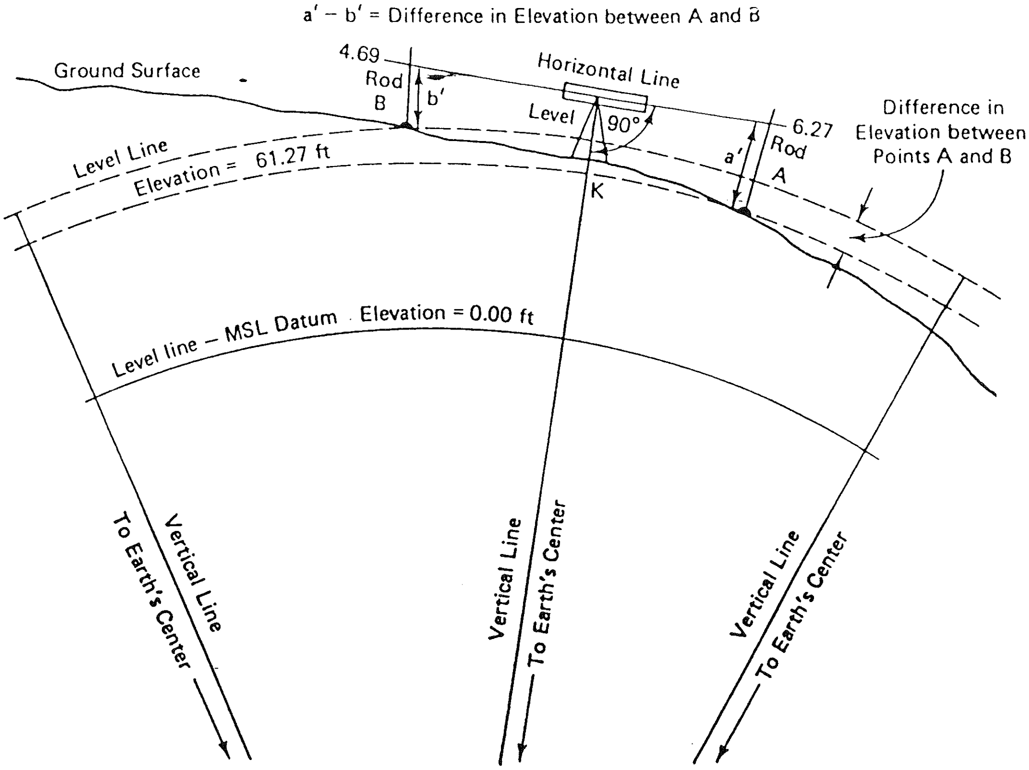
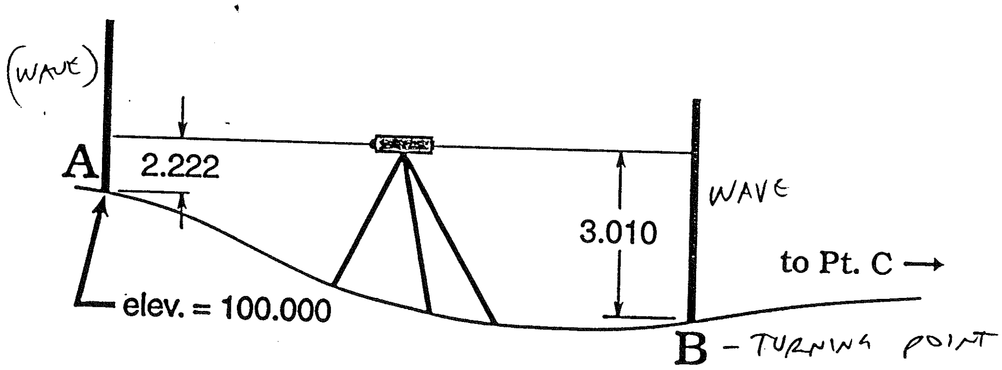


FIGURE 3.2 Leveling terms.

Note:
Level does
not have
to be in
a line
between
stations.
- Can be
anywhere
since sight
is on a
level SURFACE



elevation of A = 100.000

Look back at A and read the rod: backsight = 2.222

elevation of line of sight = HI = 102.222

Look ahead to B and read the rod: foresight = 3.010

elevation of B = 99.212

Sta	B.S.	HI	FS	Elev.	Description
Pt. A	2.222			100.000	Bronze Disk #2389
T.P. #1					Pt. B
Pt. C					Wooden stake
Pt. A					Bronze disk #2389

NOTES

ARITHMETIC CHECK

IS ERROR OK?

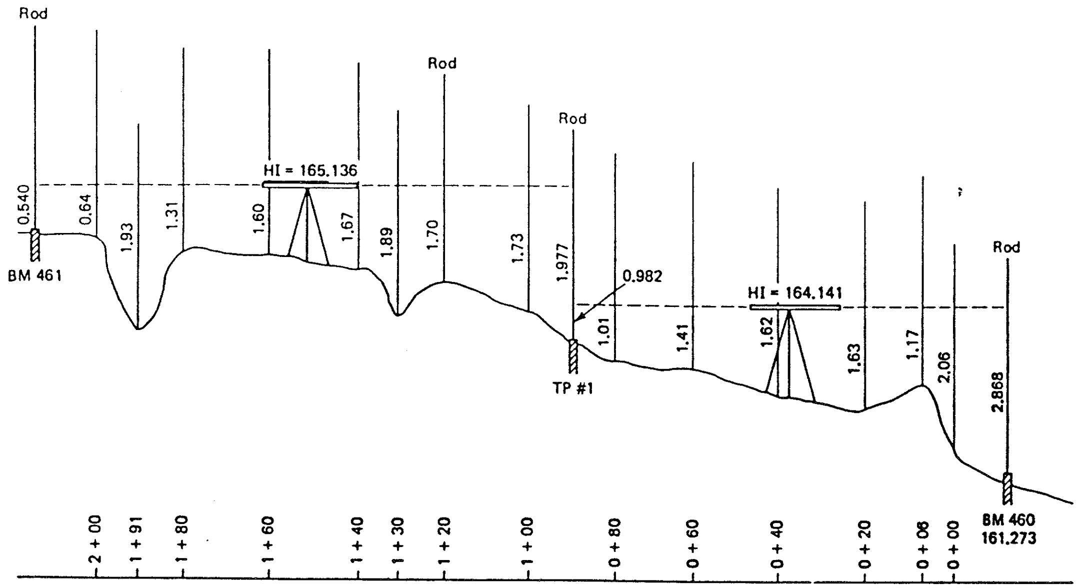


FIGURE 3.20 Example of profile leveling; see Figure 3.21 for survey notes.

PROFILE OF PROPOSED
ROAD 0 + 00 to 2 + 00

SMITH—NOTES
BROWN—⊼
JONES—ROD

Job 21 °C — SUNNY LEVEL #L–14
Date AUG 3 1995 Page 72

STA.	B.S.	H.I.	I.S	F.S.	ELEV.	DESCRIPTION
BM 460	2.868	164.141			161.273	BRONZE PLATE SET IN --- ETC.
0 + 00			2.06		162.08	℄
0 + 06			1.17		162.97	℄ - TOP OF BERM
0 + 20			1.63		162.51	℄
0 + 40			1.62		162.52	℄
0 + 60			1.41		162.73	℄
0 + 80			1.01		163.13	℄
T.P. #1	1.977	165.136		0.982	163.159	NAIL IN ROOT OF MAPLE --- ETC.
1 + 00			1.73		163.41	℄
1 + 20			1.70		163.44	℄
1 + 30			1.89		163.25	℄ BOTTOM OF GULLY
1 + 40			1.67		163.47	℄
1 + 60			1.60		163.54	℄
1 + 80			1.31		163.83	℄
1 + 91			1.93		163.21	℄ BOTTOM OF GULLY
2 + 00			0.64		164.50	℄
BM 461				0.540	164.596	BRONZE PLATE SET IN --- ETC.
						164.591 — PUBLISHED ELEV.
	Σ=4.845			Σ=1.522		E = 164.596
ARITHMETIC CHECK: 161.273 + 4.845 −1.522						164.591
					= 164.596	0.005
						ALLOWABLE ERROR (3RD ORDER)
						= 12 mm $\sqrt{K}$, = .012 $\sqrt{.2}$ = .0054 m
						ABOVE ERROR (.005) SATISFIES 3RD ORDER.

FIGURE 3.21 Profile field notes.

ERRORS IN LEVELING

SYSTEMATIC ERRORS

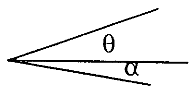
If the line of sight through the level is not horizontal then collimation error can occur. A peg test is performed to calculate collimation error. Keeping the BS and FS distances equal will eliminate this error.

BLUNDERS

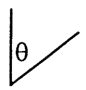
misreading the foot or meter mark on the rod
transposing figures
resting the hands on the tripod while reading the rod, causing the instrument to go off level
switching the BS and FS readings
bubble not centered
mistakes in arithmetic, can be eliminated by arithmetic check of BS's and FS's
rod is not vertical when taking a reading, can be eliminated by waving the rod and taking the lowest rod reading
Parallax: when the cross-hairs are not completely focused, it will appear that the cross-hair is moving slightly up and down as the observer's head moves up and down, this can cause incorrect readings. To eliminate parallax, take the telescope out of focus, make the cross-hairs crisp and dark and then re-focus the telescope. The cross-hairs should be crisp on the image and should not appear to be floating.

Angle Measurements

- units are generally degrees, minutes, seconds
  360 degrees = full circle
  60 minutes = 1 degree
  60 seconds = 1 minute
- measure horizontal or vertical angles
- horizontal typically measured in clockwise direction
- vertical measured either from level



or zenith



- zenith is directly up from instrument (nadir directly down) - form vertical line through the instrument

<u>Equipment</u>
- transit
- theodolite
- total station
- targets or plumb bobs

<u>Theodolite</u>
- modern instrument
- internal optics
- accuracy of ≈10" (T1)
  ≈1" (T2)
  ≈0.2" (T3)

direction
- cannot set zero
- more precise

repeating
- can set zero
- upper and lower motion
- lower allows instrument to be rotated without changing angle setting
- upper changes setting

TRANSITS AND THEODOLITES

These terms can be used to describe those survey instruments that measure horizontal and vertical angles precisely. They are also used to establish straight lines, to establish horizontal and vertical distances through the use of stadia and to establish elevations when used as a level.
The vernier scale on transits allow angles to be read to 1', 30" or 20" of arc. Theodolites with optical micrometers can be read to 20", 10", 6" or 1" of arc.

Measuring and doubling 3 angles of a triangle.

INST AT	SIGHT	D/R	HCR	Mean Angle
A	B	D	0° 00' 00"	
	C	D	60° 10' 00"	
	C	R	120° 21' 00"	60°10' 30"
B	C	D	0° 00' 00"	
	A	D	58° 29' 00"	
	A	R	116° 58' 00"	58° 29' 00"
C	A	D	0° 00' 00"	
	B	D	61° 19' 00"	
	B	R	122° 38' 00"	61°19' 00"
				179° 58' 30"
	closing error =	- 1' 30"		

Adjust each angle equally.

Angle	Corr'n	Corrected angle
A	+30"	60° 11' 00"
B	+30"	58° 29' 30"
C	+30"	61° 19' 30"
		180° 00' 00"

ERRORS WHEN USING A TRANSIT OR THEODOLITE

Instrument errors
-plate bubbles out of adjustment
-axis of line of sight not perpendicular to the horizontal axis (lack of coincidence between the line of sight and the optical axis) reading direct and reverse will eliminate error
-horizontal axis not perpendicular to the vertical axis, no error as long as objects sighted are at the same angle of inclination, however if not then error will be cancelled using direct and reverse (because one error is too large other error is too small)
-unsteady tripod, bolts won't clamp securely

Natural errors
wind - vibrates the theodolite, causes plumb bob to swing on the transit
temperature - hot sun can affect the plate bubbles
refraction - heat waves can cause distortion (a shimmering effect)
soggy ground - settling of instrument

Human errors
-instrument not set up exactly over the point
-bubbles not leveled - check the level before reading and after
-improper use of upper and lower motion clamps and tangent screws
-parallax - cross-hairs not focused correctly on the object sighted
-taking too much time setting the cross-hairs on the target
-careless plumbing at target sight or sight not set directly over the target point

Blunders
-setting up on the wrong point or sighting the wrong point
-recording an incorrect value
-reading the wrong circle (VCR & HCR)

Procedure for Angle Reading Using Repeating Theodolite

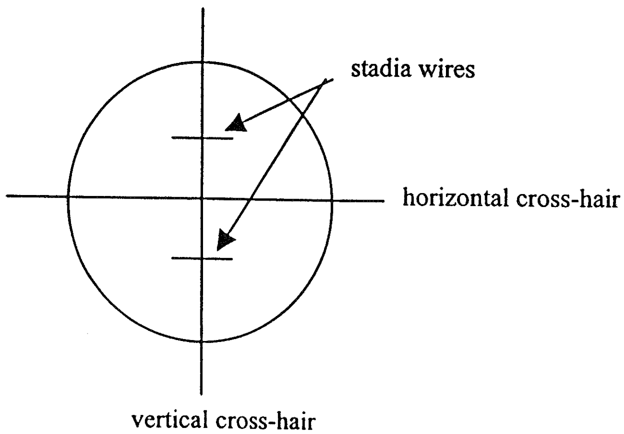
a) Set sclaes to zero. First set micrometer to zero. Then loosen upper clamp and turn horizontal circle until it is near zer. Tighten clamp and use tangent screw (slow motion) to precisely set zero degrees. Note that you usually put the vertical line of zero between two hailines. (Could also loosen both upper and lower clamp to rotate circle.) Instrument should read 0°00'00".

b) Sight initial point. To do so, loosen lower clamp and turn to roughly sight. Tighten lower clamp and then use lower tangent screw to sight point precisely. Instrument should still read zer.

c) Turn angle. Loosen upper clamp and turn to final point (usually clockwise). When close, tighten upper clamp and use upper tangent screw to set vertical cross-hair on point. Read the angle (use the micrometer knob to place even degree mark between wires before reading the micrometer) and record in field book.

d) Repeat (double) the angle. After initial angle is booked, transit (plung, flip) the telescope, loosen lower clamp and sight initial target using lower tangent screw as required. Now sight initial point with initial angle still on the instrument. Loosen the upper clamp and once again turn to final point. Tighten upper clamp and use the upper transit to sight precisely. Reading on instrument is now approximately double the actual angle. Record value in field book.

cross-hairs - focus with knob on eyepiece
then use telescope focus
all instruments have this adjustment
put hand or field book in fron to focus



Note: Make sure you use the proper cross-hair for your application!

Theodolite Setup (by one surveyor)

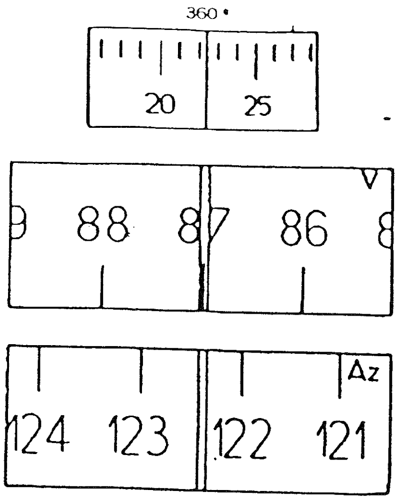
Require: Instrument with tribrach (three screw leveling head), optical plummet, circular ("bulls-eye") bubble, plate (tubular) bubble and tripod with adjustable length legs.

Procedure:

1. Locate the theodolite over the point in such a way that the top of the tripod plate is as level as possible, and so that the point can be seen through the optical plummet. It may help to place one leg on the ground and pick up the other two legs while sighting through the optical plumb. Ensure that the tripod legs are firmly pressed into the ground at the end of this step. Note that if you are setting up on an inclined area one of the tripod legs should face up the incline and the other two face down to ensure maximum stability. Do not set the instrument up too low (or high) for comfortable usage.
2. Use the tribrach leveling screws to locate the bulls-eye of the optical plumb directly over the point. Use two screws first (rotated in opposite directions) and then the third. This allows you to move the sight in perpendicular directions and converge on the point more quickly.
3. Adjust the legs of the tripod (i.e. change the length) to level the instrument using the circular ("bulls-eye") bubble. Normally you use only two legs for this procedure and adjust only one leg at a time.
4. If necessary, repeat steps 2 and 3 until the optical plummet is sighting the point and the circular bubble is centred.
5. Do final leveling of the instrument using the tribrach leveling screws and the plate bubble, first using the two tribrach leveling screws parallel to the plate bubble tube followed by a single screw adjustment of the third screw with the plate bubble tube rotated 90° from the first position. This may slightly move the optical plummet off the point.
6. Move the instrument on the tribrach if required (Do not rotate!) to bring the optical plumb over the point and then, if necessary, re-level the plate bubble.

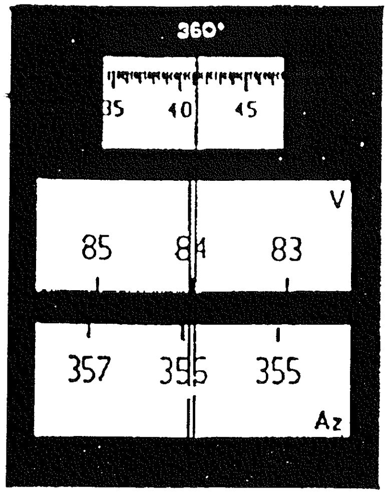
Notes:

1. Before starting your setup ensure that the tribrach leveling screws are approximately in the centre of their travel in order to give the most range for setup.
2. When adjusting the plate bubble, remember that the bubble moves in the direction of your left thumb. This holds true for all instruments (including Engineer's Transits, levels etc.).
3. The adjustment of the circular ("bulls-eye") bubble may not match the plate bubble. The plate bubble is the one which must be correct. If the circular bubble is not entirely in the circle when the instrument is level, ask your instructor for assistance.
4. When adjusting the slow motion screws of the instrument ensure that the final adjustment is made while turning the screw in a clockwise direction. This will prevent errors caused by the slow motion spring not expanding fully (i.e. a clockwise turn guarantees that the motion is against the spring, causing it to compress).
5. On instruments equipped with an alidade bubble, ensure that the bubble is centred prior to each reading of a vertical angle. This is very important!
6. To set the horizontal circle to 0°, first zero the micrometer, then set the circle to zero using the upper clamp and slow motion (tangent) screw. To set either 90° or 270° on the vertical circle, first ensure the alidade bubble is centred (if your instrument does not have an automatic alidade), zero the micrometer, and then set the angle on the vertical circle using the vertical clamp and slow motion screw. To set the horizontal circle of a direction instrument to approximately 0° first set the micrometer to about 30″, then turn the horizontal circle setting knob until 0° and 0°±180° (upside down) are approximately in line.
7. A more detailed description of the setup procedure may be found in the text on page 143 (3rd edition).

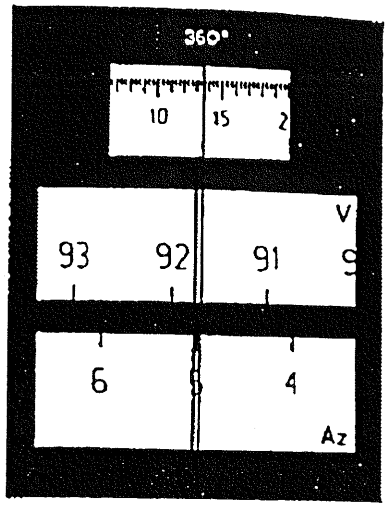


Vertical Circle: 87° 22.4'

WILD TIA

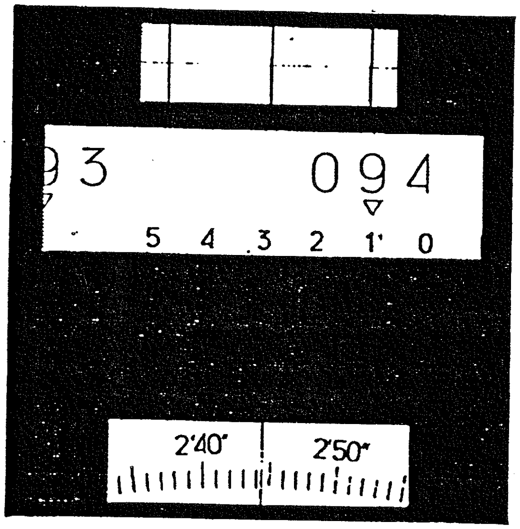


Vertical Circle: 84° 41' 15"



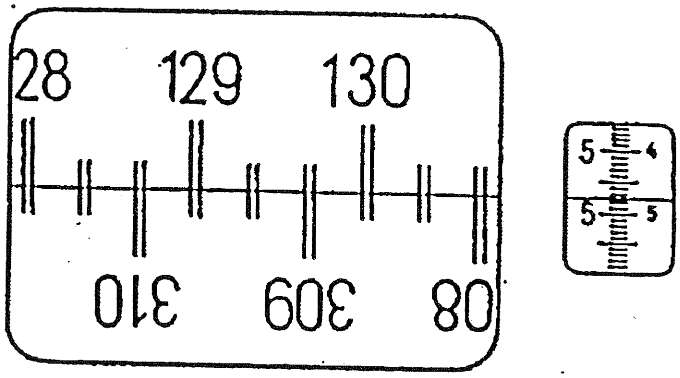
Horizontal Circle: 5° 13' 35"

WILD TI



vertical angle = 94°12'44"

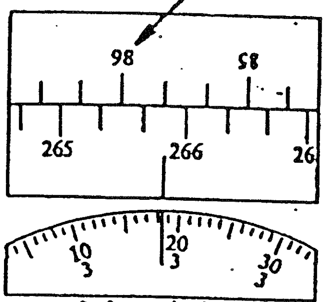
WILD T2 (NEW)



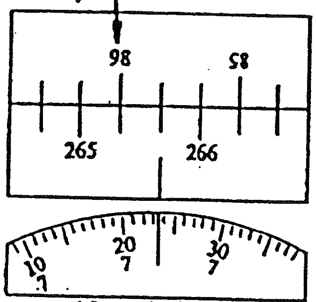
Degrees	. . . .	129°
Number of scale divisions = 2×10 =	. .	20'
Single minutes and seconds	. . . . . .	5' 47.5"
Total reading	. . . .	129° 25' 47.5"

ZEISS THEO 010

Note: 86° (not 98°)

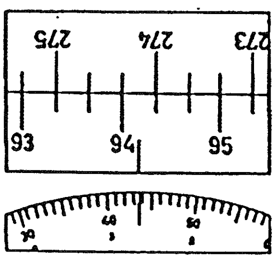


Before coincidence

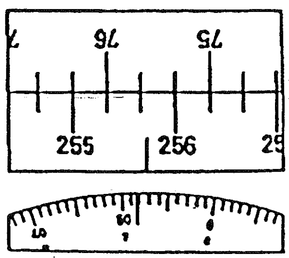


After coincidence

Circle reading..	265° 40'
Drum reading..	7' 23.6"
	265° 47' 23.6"



94° 10'
8' 44"
94° 18' 44"



255° 40'
7' 52"
255° 47' 52"

Note: Not 256° because 76° is left of 256°

WILD T2 (OLD)

FIELD NOTES:

All field notes should be recorded in the field, in the field books, not on scrap paper.
All field notes should be printed in pencil.
All diagrams should be oriented with North to the top of the page.
All sketches should have straight lines drawn with a straight edge.
All errors in field measurements must not be erased but crossed out and re-written.
When copying field notes from a party member, the word copy should be shown.

Name and party number should be on the cover and spine of the field book.
All the pages should be numbered.
Leave 20 pages for the diaries.
Table of contents: left side for diaries, right side for field work.

Notes should contain:

Job Title
Date
Temperature and weather conditions
instrument number, tripod number, rod number, etc.
bag number

Party information:
Party chief (notekeeper)
Instrument person (level)
Instrument person (transit)
Rod person
Chain person

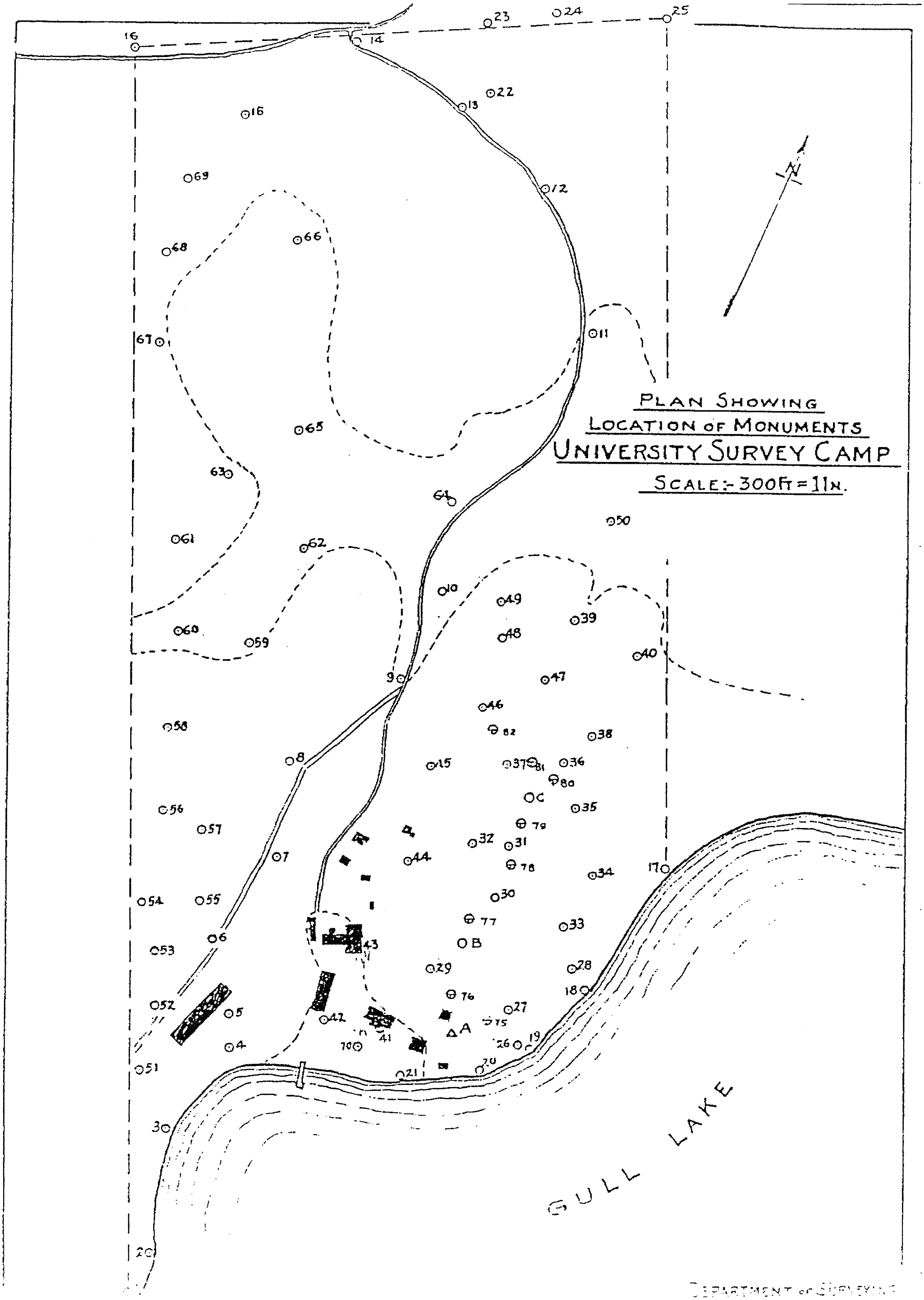
Required Diary Contents (for each exercise):

- Title + Date
- Nature and purpose of work to be undertaken
- Instruments used (make, model & number)
- Weather + temperature
- Party members + tasks each performed (for each portion of the work)
  Suggested format (using icons):
  - Fred Flintstone (*instrument*)
  - Barney Rubble (*notes*)
  - Gayle Granite (*rod*)
  - Stephanie Slate (*chain*)
  - Mary Marble (*plumb bob*)
- State whether work was completed satisfactorily and how much was done
- If necessary, describe any unusual items which may have affected the work

Note: Point form is preferable for diary entries

Page 1: Title Page
Pages 3-20: Diary
Pages 21- : Field Notes

Diary Contents			Field Work Contents		
Page	Location	Date	Page	Location	Date
3	Peg Test		21	Peg Test	



DEPARTMENT OF SURVEYING