Learning Objectives

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Learning objectives

1. Know how leveling is used in civil and environmental applications
2. Know the definitions of: vertical line, level surface, level line, vertical datum, elevations, geoid, mean sea level, tidal datum, benchmark, leveling, vertical control
3. Know how to correct a level survey for earth's curvature and refraction
4. Demonstrate how to set-up and operate a survey level.
5. Show how to keep field book notes for a differential level circuit
6. Be able to adjust a differential level circuit for survey error
7. Know how to calculate elevations using trigonometric leveling
8. Understanding how a compensating level works
9. Know how to complete a two-peg tests

Homework Assignment

Do the following problems from the textbook:

4.2 (5 points)
4.5 (10 points)
4.6 (5 points)
4.8 (10 points)
4.13 (10 points)
4.14 (5 points)
4.16 (10 points)
5.1 (10 points)
5.12 (20 points)
5.35 (10 points)
5.37 (5 points)

Introduction

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Leveling is the general term applied to any of the various processes by which elevations of points or differences in elevation are determined. It is a vital operation in producing necessary data for mapping, engineering design, and construction.

Leveling results are used to

(1) design highways, railroads, canals, sewers, water supply systems, and other facilities having grade lines that best conform to existing topography;
(2) lay out construction projects according to planned elevations;
(3) calculate volumes of earthwork and other materials;
(4) investigate drainage characteristics of an area;
(5) develop maps showing general ground configurations; and
(6) study subsidence and crustal motion of the Earth.

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Definitions

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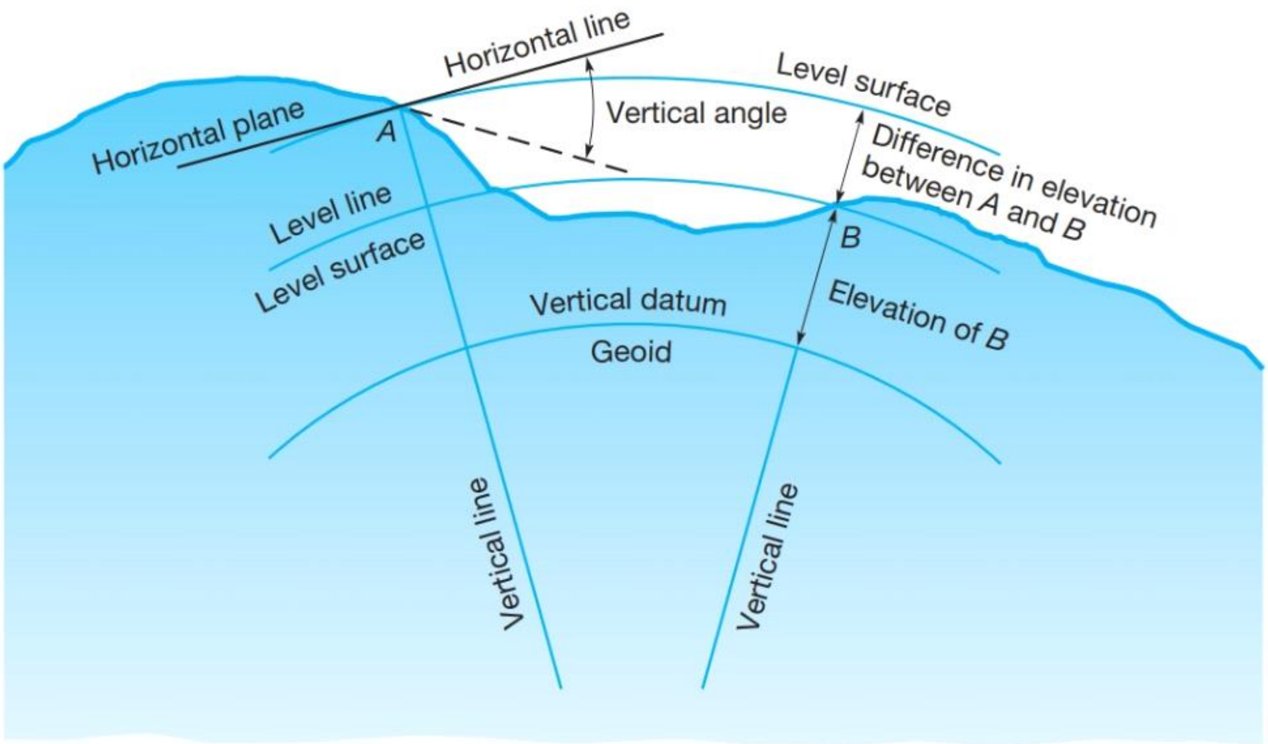

Vertical line. A line that follows the local direction of gravity as indicated by a plumb line.

Level surface. A curved surface that at every point is perpendicular to the local plumb line (the direction in which gravity acts). Level surfaces are approximately spheroidal in shape. A body of still water is the closest example of a level surface. Within local areas, level surfaces at different heights are considered to be concentric.1 Level surfaces are also known as equipotential surfaces since, for a particular surface, the potential of gravity is equal at every point on the surface.

Level line. A line in a level surface—therefore, a curved line. Horizontal plane. A plane perpendicular to the local direction of gravity. In plane surveying, it is a plane perpendicular to the local vertical line. Horizontal line. A line in a horizontal plane. In plane surveying, it is a line perpendicular to the local vertical

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Definitions (cont.)

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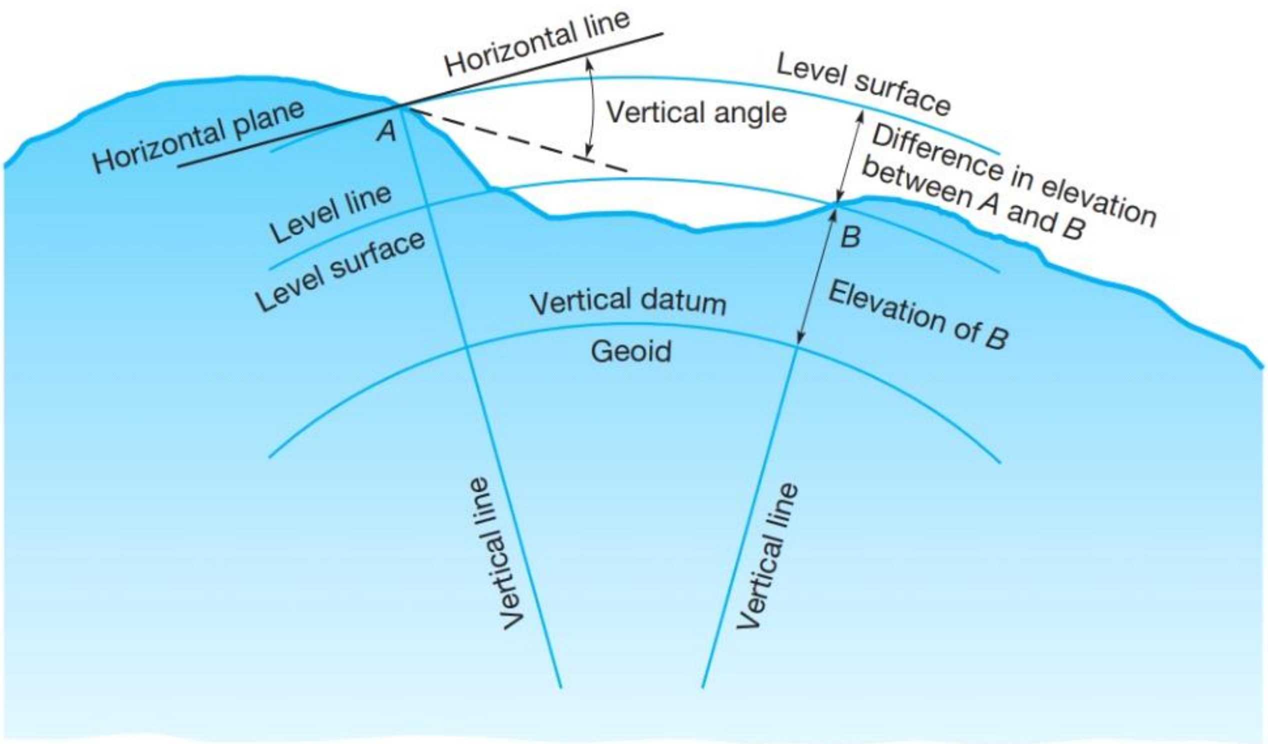

Vertical datum. Any level surface to which elevations are referenced. This is the surface that is arbitrarily assigned an elevation of zero (see Section 19.6). This level surface is also known as a reference datum since points using this datum have heights relative to this surface.

Elevation. The distance measured along a vertical line from a vertical datum to a point or object. If the elevation of point A is 802.46 ft, A is 802.46 ft above the reference datum. The elevation of a point is also called its height above the datum and orthometric height.

Geoid. A particular level surface that serves as a datum for all elevations and astronomical observations.

Mean sea level (MSL). This term is no longer applicable to benchmark elevations in NAVD88. MSL was defined as the average height for the surface of the seas for all stages of tide over a 19-year period as determined by the National Geodetic Vertical Datum of 1929, further described in Section 4.3.

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Definitions (cont.)

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Tidal datum. The vertical datum used in coastal areas for establishing property boundaries of lands bordering waters subject to tides. A tidal datum also provides the basis for locating fishing and oil drilling rights in tidal waters, and the limits of swamp and overflowed lands. Various definitions have been used in different areas for a tidal datum, but the one most commonly employed is the mean high water (MHW) line. Others applied include mean higher high water (MHHW), mean low water (MLW), and mean lower low water (MLLW). Interpretations of a tidal datum, and the methods by which they are determined, have been, and continue to be, the subject of numerous court cases.

Benchmark (BM). A relatively permanent object, natural or artificial, having a marked point whose elevation above or below a reference datum is known or assumed. Common examples are metal disks set in concrete (see Figure 20.8), reference marks chiseled on large rocks, non- movable parts of fire hydrants, curbs, and so on.

Leveling. The process of finding elevations of points or their differences in elevation.

Vertical control. A series of benchmarks or other points of known elevation established throughout an area, also termed basic control or level control. The basic vertical control for the United States was derived from first- and second-order leveling. Less precise third-order leveling has been used to fill gaps between second-order benchmarks, as well as for many other specific projects (see Section 19.10).

Elevations of benchmarks, which are part of the National Spatial Reference System, can be obtained online from the National Geodetic Survey at http://www.ngs.noaa.gov.

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Curvature

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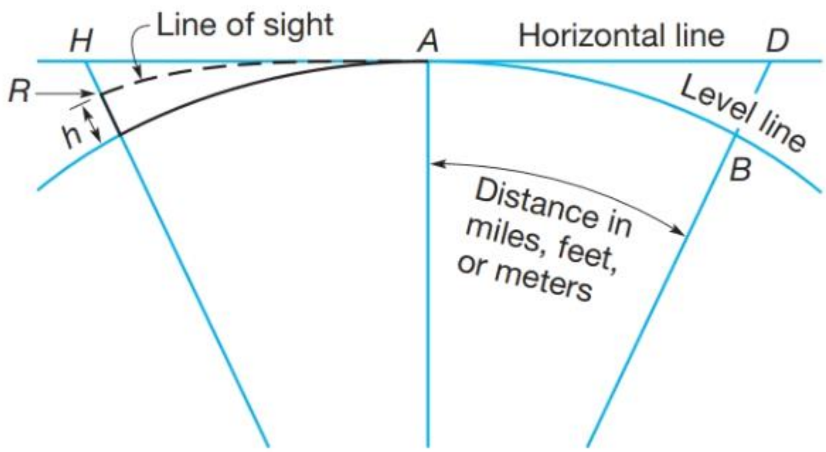

Figure 4.2
Curvature and refraction.

<u>Curvature</u>

Since points A and B are on a level line, they have the same elevation. If a graduated rod was held vertically at B and a reading was taken on it by means of a telescope with its line of sight AD horizontal, the Earth's curvature would cause the reading to be read too high by length BD.

<u>Correction for Curvature</u>

In Figure 4.2 the deviation DB from a horizontal line through point A is expressed approximately by the formulas or

$C_f = 0.667M^2 = 0.0239F^2$ Deviation DB from horizontal line

$C_m = 0.0785K^2$

where the departure of a level surface from a horizontal line is C_f in feet or C_m in meters, M is the distance AB in miles, F the distance in thousands of feet, and K the distance in kilometers.

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Refraction

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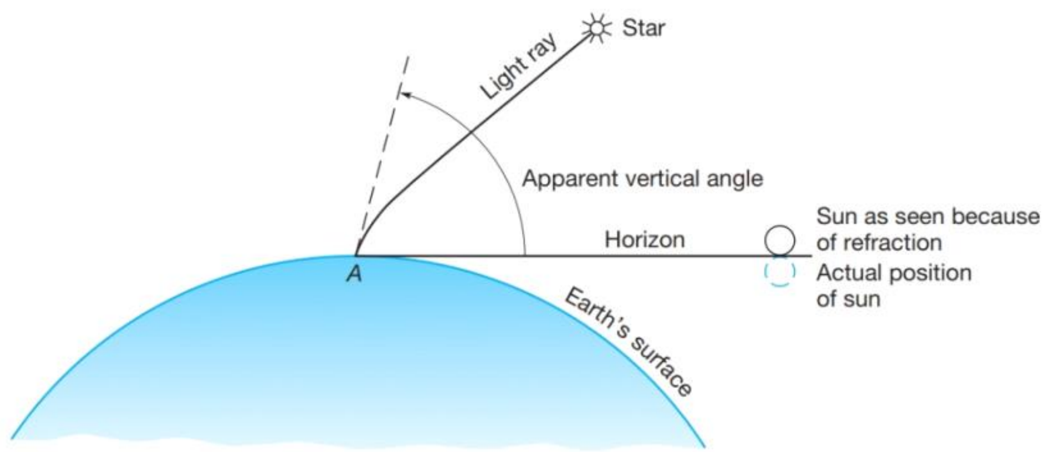

Figure 4.3
Refraction.

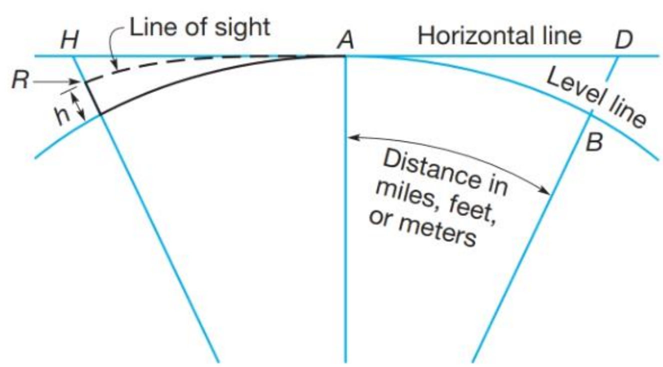

Figure 4.2
Curvature and refraction.

Light rays passing through the Earth's atmosphere are bent or refracted toward the Earth's surface, as shown in Figure 4.3. Thus a theoretically horizontal line of sight, like AH in Figure 4.2, is bent to the curved form AR. Hence the reading on a rod held at R is diminished by length RH.

The effects of refraction in making objects appear higher than they really are (and therefore rod readings too small)

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Refraction (cont.)

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Displacement resulting from refraction is variable. It depends on atmospheric conditions, length of line, and the angle a sight line makes with the vertical. For a horizontal sight, refraction Rf in feet or Rm in meters is expressed approximately by the formulas

$R_f = 0.093\ M^2 = 0.0033\ F^2$

From <https://online.vitalsource.com/#/books/9781292060675/cfi/95!/4/4@0:6.34>

or

$R_m = 0.011K^2$

The refraction effect is about one seventh the effect of curvature of the Earth, but in the opposite direction.

The combined effect of curvature and refraction is approximately or

$h_f = 0.574\ M^2 = 0.0206\ F^2$ (4.3a)

$h_m = 0.0675\ K^2$

where h_f is in feet and h_m is in meters.

From <https://online.vitalsource.com/#/books/9781292060675/cfi/96!/4/4@0.00:0.00>

For sights of 100, 200, and 300 ft, h_f = 0.00021, 0.00082, and 0.0019 ft, respectively, or 0.00068 m for a 100 m length. The combined effects of curvature and refraction produce rod readings that are slightly too large, *proper field procedures in differential leveling can practically eliminate the error due to these causes.* However, this is not true for trigonometric leveling (see Section 4.5.4) where this uncompensated systematic error can result in erroneous elevation determinations. This is one of several reasons why trigonometric leveling has never been used in geodetic surveys

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Differential Leveling

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In this most commonly employed method, a telescope with suitable magnification is used to read graduated rods held on fixed points. A horizontal line of sight within the telescope is established by means of a level vial or automatic compensator.

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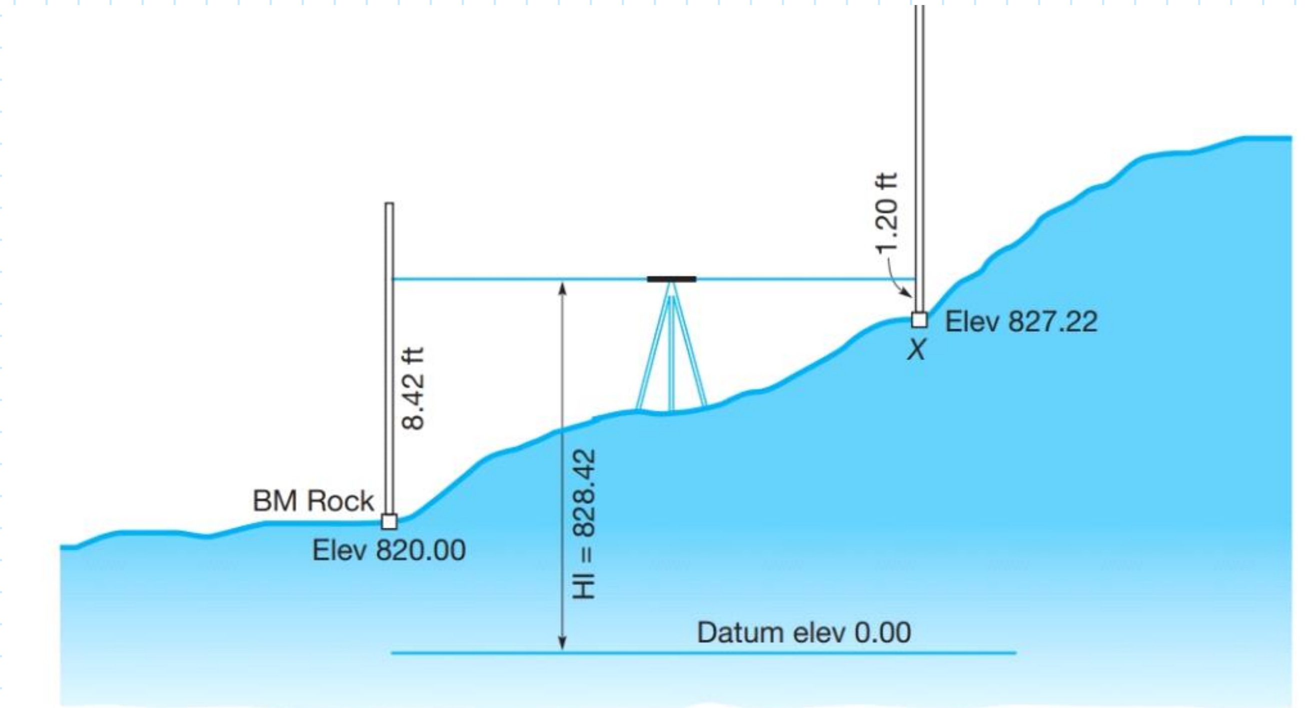

Differential Leveling - Equipment

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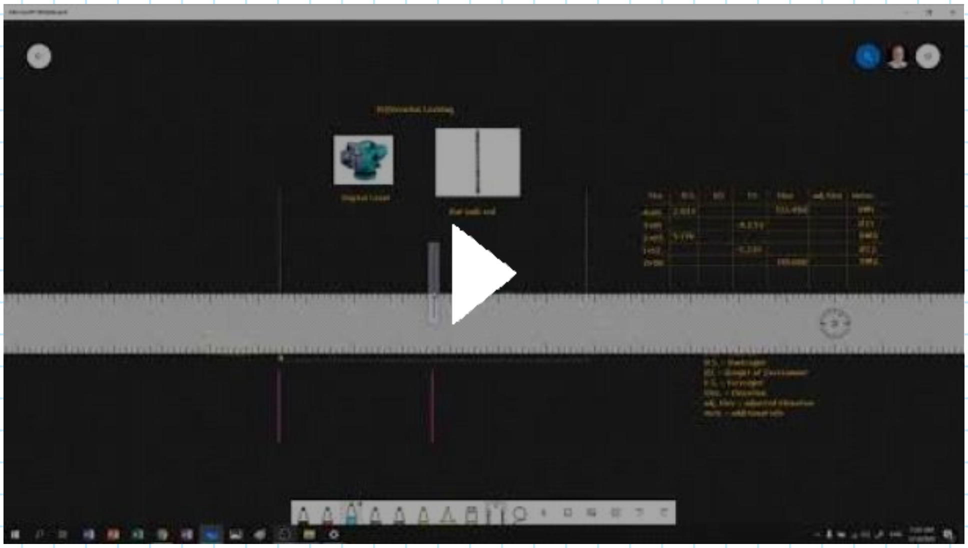
Equipment Setup (12:45 min)

[CVEEN 2410 Ch 4 Level Set up]

Differential Level - Example

[CVEEN 2410 Ch 4 Differential Leveling]



Differential Level - Example 1

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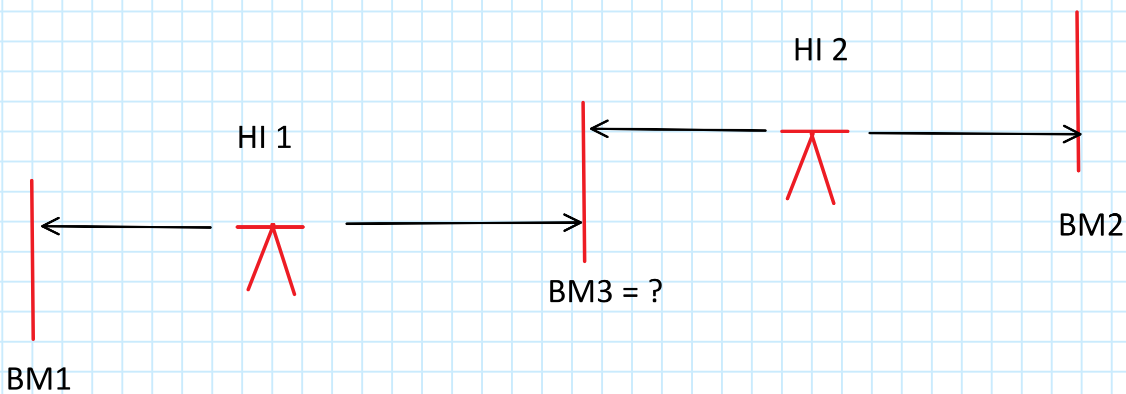
Example 1

Sta.	B.S.	HI	FS	Elev	adj Elev	Notes
4+05	2.817			121.480		BM1
3+01			-4.237			HI1
2+03	3.176					BM3
1+02			-5.231			HI2
0+00				118.000		BM2

HI 1 = 121.480+2.817=124.297
BM3 = 124.297-4.237=120.060
HI 2 = 120.06+3.176=123.236
BM2 = 123.236-5.231=118.005

0.005/4=0.0013 (error per leg)



Adjusted survey

HI 1 = 124.297-0.0013*1=124.2957= 124.300
BM3 = 120.06-0.0013*2=120.0574 = 120.057
HI 2 = 123.236-0.0013*3=123.2321 = 123.232
BM2 = 118.005-0.0013*4=117.9998 =118.000

Differential Level - Example 2 - Turnpoints

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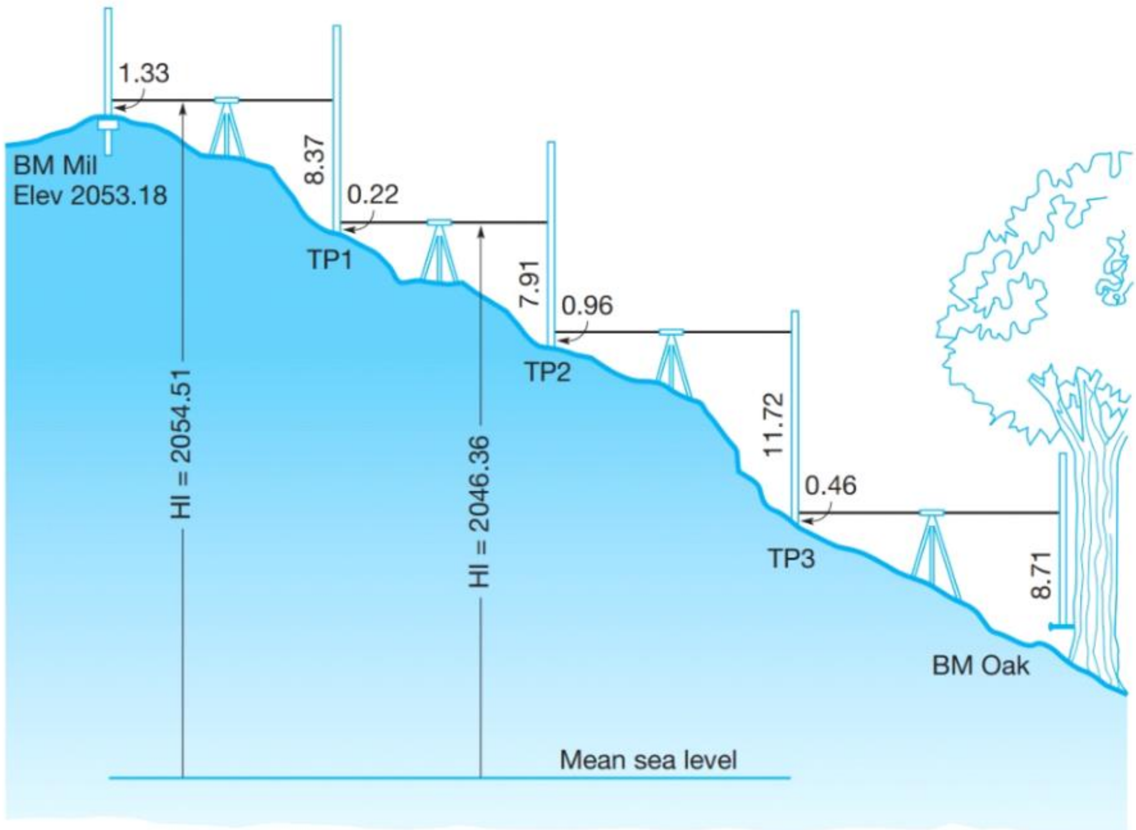
Example 2



Differential Level - Example 2 - Field book

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Example 2 (cont.)

DIFFERENTIAL LEVELS

Sta.	+ B.S.	H.I.	− F.S.	Elev.	Adj. Elev.
BM Mil.	1.33			2053.18	2053.18
		2054.51		(−0.004)	
TP1	0.22		8.37	2046.14	2046.14
		2046.36		(−0.008)	
TP2	0.96		7.91 ~~8.91~~	2038.45	2038.44
		2039.41		(−0.012)	
TP3	0.46		11.72	2027.69	2027.68
		2028.15		(−0.016)	
BM Oak	11.95		8.71	2019.44	2019.42
		2031.39		(−0.022)	
TP4	12.55		2.61	2028.78	2028.76
		2041.33		(−0.026)	
TP5	12.77		0.68	2040.65	2040.62
		2053.42		(−0.030)	
BM Mil.			0.21	2053.21	2053.18
Σ =	+40.24	Σ =	−40.21		
		Page Check:			
		2053.18			
		+ 40.24			
		2093.42			
		− 40.21			
		2053.21	Check		

Balancing Foresight and Backsight

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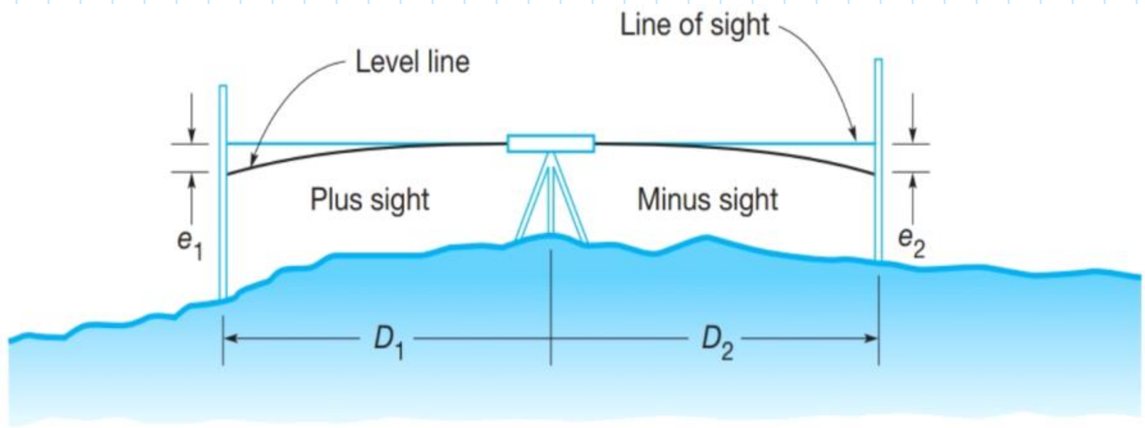

Balancing plus and minus sight distances will eliminate errors due to instrument maladjustment (most important) and the **combined effects of the Earth's curvature and refraction** Here e1 and e2 are the combined curvature and refraction errors for the plus and minus sights, respectively. If D1 and D2 are made equal, e1 and e2 are also equal. In calculations, e1 is added and e2 subtracted; thus they cancel each other.

Balancing Foresight and Backsight

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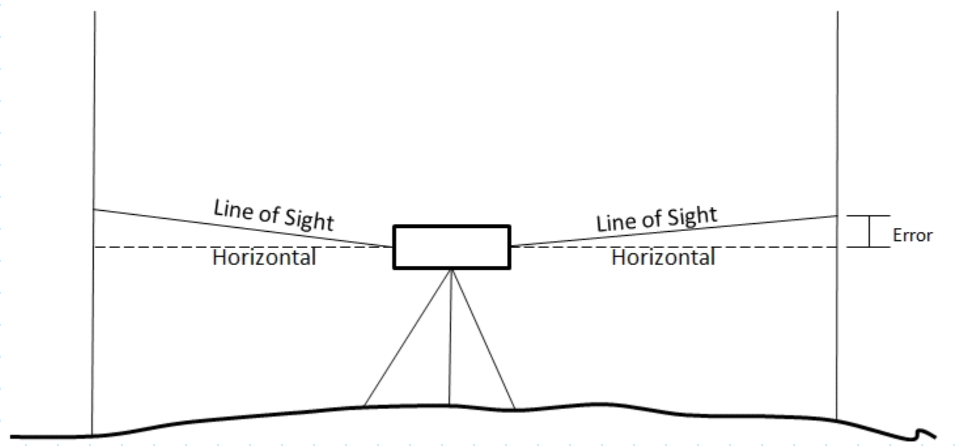

collimation error

[ˌkäl·ə′mā·shən ˌer·ər]
(astronomy)
The amount by which the angle between the optical axis of a transit telescope and its east-west mechanical axis deviates from 90°.
(engineering)
Angular error in magnitude and direction between two nominally parallel lines of sight.
Specifically, the angle by which the line of sight of a radar differs from what it should be.

From <https://encyclopedia2.thefreedictionary.com/collimation+error>

The above figure illustrates the importance of balancing sight lengths if a **collimation error** exists in the instrument's line of sight. This condition exists, if after leveling the instrument, its line of sight is not horizontal. For example, suppose in Figure 5.7 because the line of sight is systematically directed below horizontal, an error e1 results in the plus sight. But if D1 and D2 are made equal, an error e2 (equal to e1) will result on the minus sight and the two will cancel, thus eliminating the effect of the instrumental error. On slopes it may be somewhat difficult to balance lengths of plus and minus sights, but following a zigzag path can do it usually. It should be remembered that Earth curvature, refraction, and collimation errors are systematic and will accumulate in long leveling lines if care is not taken to balance the plus and minus sight distances.

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Distance Measurements Using Stadia

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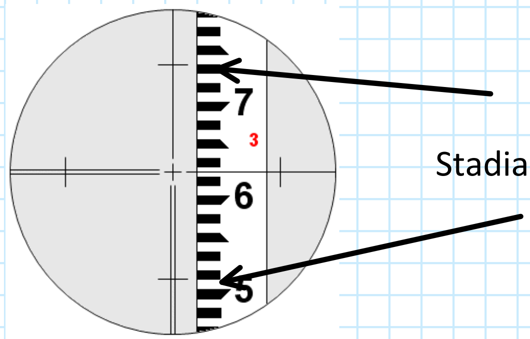
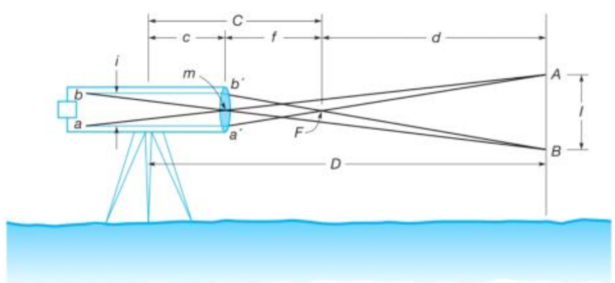


Stadia as seen in level



f = *focal length* of lens (a constant for any particular compound objective lens)
i = spacing between stadia wires (ab in Figure 5.6)
f/i = *stadia interval factor* usually 100 and denoted by K
I = rod intercept (AB in Figure 5.6), also called *stadia interval*
c = distance from instrument center (vertical axis) to objective lens center (varies slightly when focusing the objective lens for different sight lengths but is generally considered to be a constant)
C = stadia constant = $c + f$
d = distance from the focal point F in front of telescope to face of rod
D = distance from instrument center to rod face = $C + d$

From similar triangles of Figure 5.6

$$\frac{d}{f} = \frac{I}{i} \quad \text{or} \quad d = \frac{f}{i}I = KI$$

Thus

$$D = KI + C \tag{5.1}$$

These telescopes are now obsolete in surveying instruments. The objective lens of an internal focusing telescope (the type now used in surveying instruments) remains fixed in position, while a movable negative-focusing lens between the objective lens and the plane of the cross hairs changes directions of the light rays. As a result, the stadia constant, (C), is so small that it can be assumed equal to zero and drops out of Equation (5.1). Thus the equation for distance on a horizontal stadia sight reduces to

D = KI

To determine the stadia interval factor K, rod intercept I for a horizontal sight of known distance D is read. Then in an alternate form of Equation (5.2), the stadia interval factor is K = D>I. As an example, at a measured distance of 300.0 ft, a rod interval of 3.01 was read. Then K = 300.0>3.01 = 99.7. Accuracy in determining K is increased by averaging values from several lines whose observed lengths vary from about 100 to 500 ft by 100-ft increments.

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Precision in Leveling

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Precision in leveling is increased by repeating observations, making frequent ties to established benchmarks, using high-quality equipment, keeping it in good adjustment, and performing the measurement process carefully. However, no matter how carefully the work is executed, errors will exist and will be evident in the form of misclosures, as discussed in Section 5.4. To determine whether or not work is acceptable, misclosures are compared with permissible values on the basis of either number of setups or distance covered. Various organizations set precision standards based on their project requirements. For example, on a simple construction survey, an allowable misclosure of $C = 0.02 \text{ ft}\sqrt{n}$ might be used, where n is the number of setups. Note that this criterion was applied for the level circuit in the field notes of Figure 5.5.

The Federal Geodetic Control Subcommittee (FGCS) recommends the following formula to compute allowable misclosures:[1]

$$C = m\sqrt{K} \tag{5.3}$$

where C is the allowable loop or section[2] misclosure, in millimeters; m is a constant; and K the total length leveled, in kilometers. For "loops" (circuits that begin and end on the same benchmark), K is the total perimeter distance, and the FGCS specifies constants of 4, 5, 6, 8, and 12 mm for the five classes of leveling, designated, respectively, as (1) first-order class I, (2) first-order class II, (3) second-order class I, (4) second-order class II, and (5) third order. For "sections" the constants are the same, except that 3 mm applies for first-order class I and 4 mm applies to first-order class II. The particular order of accuracy recommended for a given type of project is discussed in Section 19.8.

Example:

A differential leveling loop is run from an established BM A to a point 2 mi away and back, with a misclosure of 0.056 ft. What order leveling does this satisfy?

Solution

$$C = \frac{0.056 \text{ ft}}{0.00328 \text{ ft/mm}} = 17 \text{ mm}$$

$$K = (2 \text{ mi} + 2 \text{ mi}) \times 1.61 \text{ km/mi} = 6.4 \text{ km}$$

By a rearranged form of Equation (5.3), $m = \frac{C}{\sqrt{K}} = \frac{17}{\sqrt{6.4}} = 6.7$

This leveling meets the allowable 8-mm tolerance level for second-order class II work, but does not quite meet the 6-mm level for second-order class I, and if that

Precision in Leveling (cont.)

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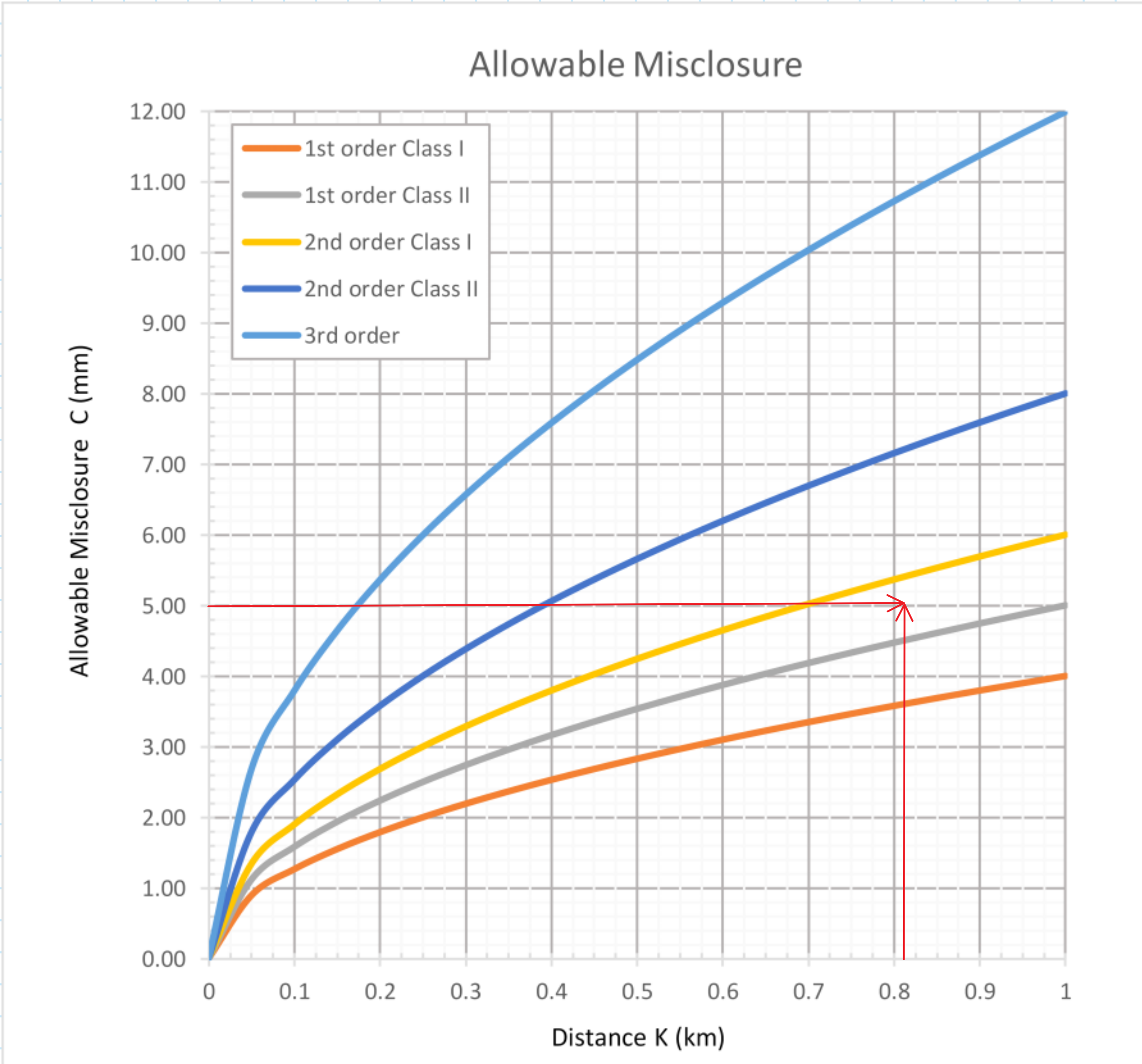

Example 3

Given the chart above, what is the "order" of survey given the amount of misclosure from Example 1.

Closure error = 5 mm
Length of survey, K = 405*2/1000=0.81 Km

Answer (2nd Order, Class I)

Adjustment of Level Surveys

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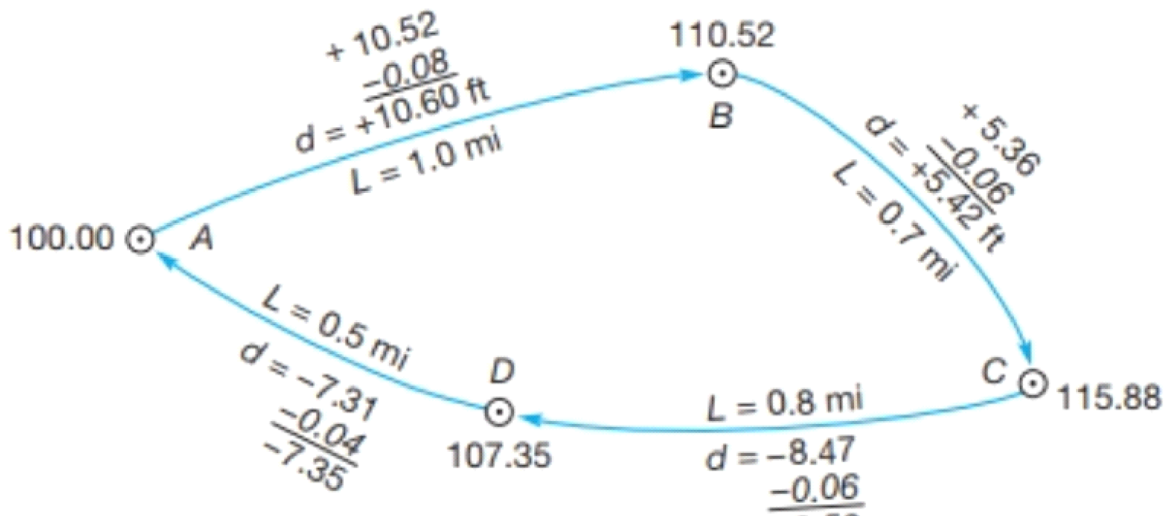

Survey check (uncorrected rods)
d=10.60+5.42-8.47-7.31=0.24
(uncorrected)

Correction of rods
-0.24/3=-0.08 per mile
1st rod 0.08*1.0=-0.08
2nd rod -0.08*0.7=-0.056=-0.06
3rd rod -0.08*0.8=-0.064=-0.06
4th rod -0.08*0.5=-0.04

Survey check (corrected rods)
10.52+5.36-8.53-7.35=0
(corrected rods) (must sum to zero)

Corrected Elevations
100+10.52=110.52
110.52+5.36=115.88
115.88-8.53=107.35
107.35-7.35=100

Since permissible misclosures are based on the lengths of lines leveled, or number of setups, it is logical to adjust elevations in proportion to these values. Observed elevation differences d and lengths of sections L are shown for a circuit in Figure 5.8.

The misclosure found by algebraic summation of the elevation differences is +0.24 ft. Adding lengths of the sections yields a total circuit length of 3.0 mi.

Elevation adjustments are then (0.24 ft / 3.0) multiplied by the corresponding lengths, giving corrections of -0.08, -0.06, -0.06, and -0.04 ft (shown in the figure).

The adjusted elevation differences (shown in black) are used to get the final elevations of benchmarks (also shown in black in the figure).

From <https://online.vitalsource.com/#/books/9781292060675/cfi/132!/4/4@0:0.795>

Adjustment of Differential Leveling Survey

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	Backsight		Foresight	
STA	+	HI	−	ELEV
BM 7	9.432			852.045
		852.045+9.432=861.477		
TP 1	6.780	853.114-8.363=844.751	8.363	853.114
		853.114+6.780=859.894		
BM 8	7.263	859.894-9.822=850.072	9.822	850.072
		850.072+7.263=857.335		
TP 2	3.915	857.335-9.400=847.935	9.400	847.935
		847.935+3.915=851.85		
TP 3	7.223	851.85-5.539=846.311	5.539	846.311
		846.311+7.223=853.534		
BM 7		853.534-1.477=852.057	1.477	852.057

Page check	852.045 + 34.613 − 34.601		852.057
Misclosure =	852.057 − 852.045	0.012	
Correction =	−0.012/5 =	−0.0024	

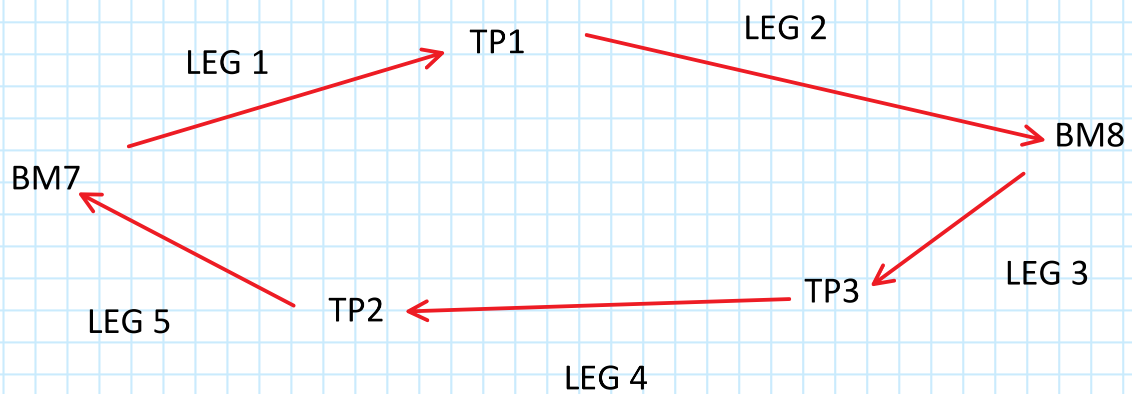
Checked in too high.

Compare 852.057 with starting elevation 852.045

Use 5 because there are 5 in legs or intermediate points to the survey.

Must subtract correction from each reading.

PLAN VIEW OF SURVEY



Trigonometric Leveling

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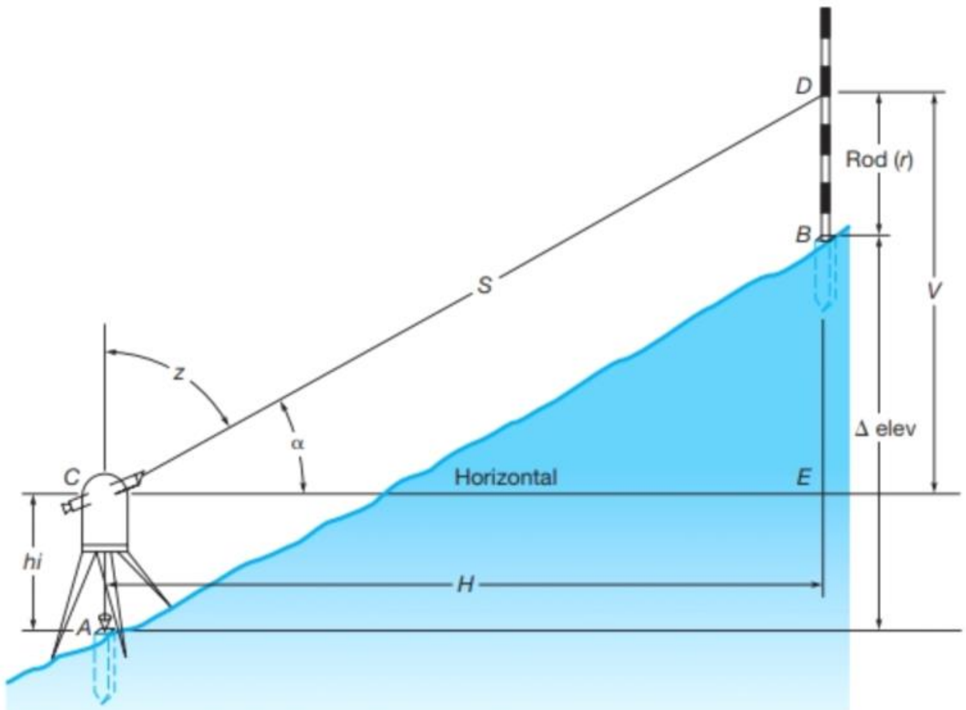
The difference in elevation between two points can be determined by measuring (1) the inclined or horizontal distance between them and (2) the zenith angle or the altitude angle to one point from the other. (Zenith and altitude angles, described in more detail in Section 8.13, are measured in vertical planes. **Zenith angles are observed downward from vertical**, and altitude angles are observed up or down from horizontal.)



$$V = S \cos z \quad (4.6)$$

or

$$V = S \sin \alpha \quad (4.7)$$

Alternatively, if horizontal distance H between C and D is measured, then V is

$$V = H \cot z \quad (4.8)$$

or

$$V = H \tan \alpha \quad (4.9)$$

The difference in elevation (Δelev) between points A and B in Figure 4.7 is given by

$$\Delta\text{elev} = hi + V - r \quad (4.10)$$

Trigonometric Level - Example Calculation

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Given

- hi = 2.300 m (height of instrument)
- α = 5.0312 deg. (angle from horizontal)
- S = 102.230 m (slope distance)
- r = 1.342 m (rod height)

Find:

- Δ elevation

Solution:

$\cos \alpha = H / S$

H = cos α*S

H = cos(5.0312)*102.230=101.8361170484325 m

V = sin α*S

V = sin(5.0312)*102.230=8.96538702443731 m

Δ elevation = hi + V - r

Δ elevation = 2.300+8.96538702443731-1.342=9.92338702443731 m

Δ elevation = 9.923 m

Trigonometric Leveling - Example Using AutoCAD

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- START DRAWING
  - TEMPLATES
    - No Template - Metric
  - CUSTOMIZATION (lower right corner of screen)
    - √ Coordinates
  - OSNAP
    - Nearest
  - PTYPE
    - ⊕
  - POINT
    - 0,0
  - LINE
    - 0,0
    - @2.300<90
    - @102.230<5.0312
    - @1.342<270
  - POINT
    - END
  - Go to home menu
  - UTILITIES
    - ID POINT

Trigonometric Leveling - Example Using AutoCAD (cont.)

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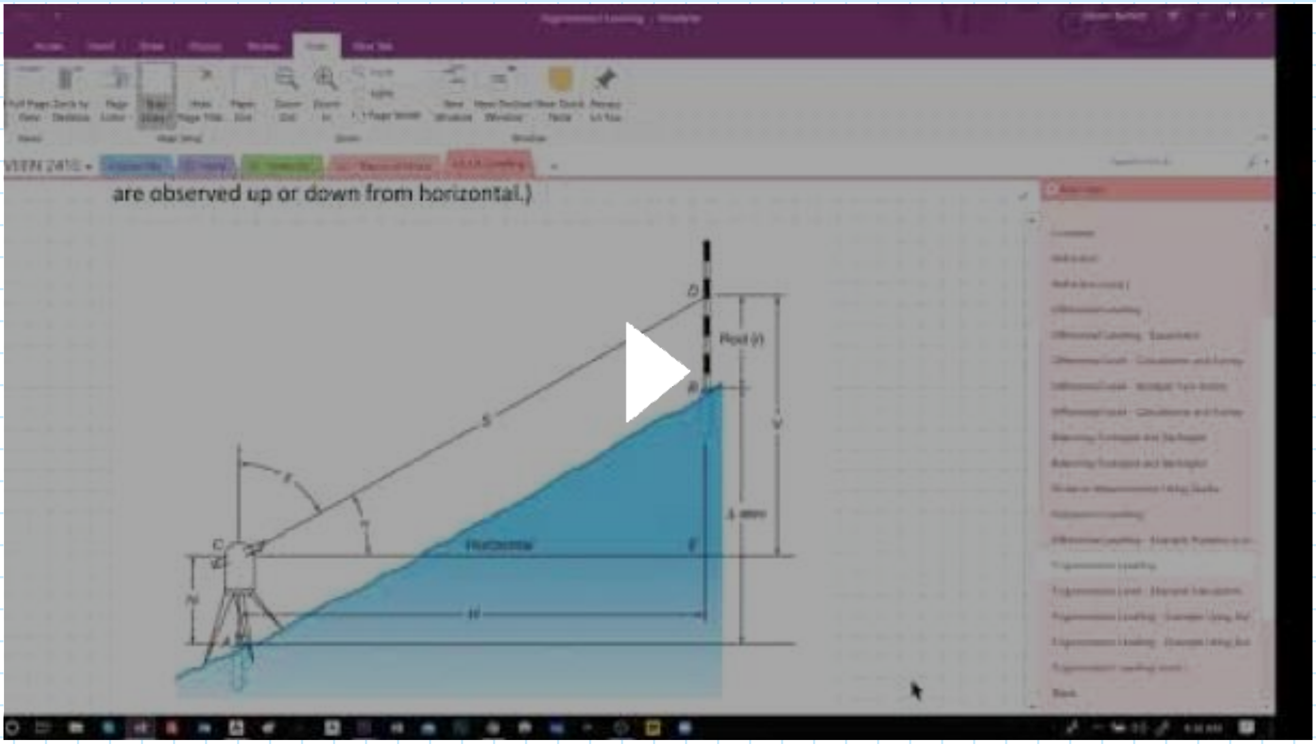
[CVEEN 2410 Trig Leveling Example Autocad]



Trigonometric Leveling - Correction for Refraction and Curvature

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For short lines (up to about 1000 ft in length) elevation differences obtained in trigonometric leveling are appropriately depicted by Figure 4.7 and properly computed using Equations (4.6) through (4.10).

For longer lines Earth curvature and refraction become factors that must be considered. Figure 4.8 illustrates the situation.

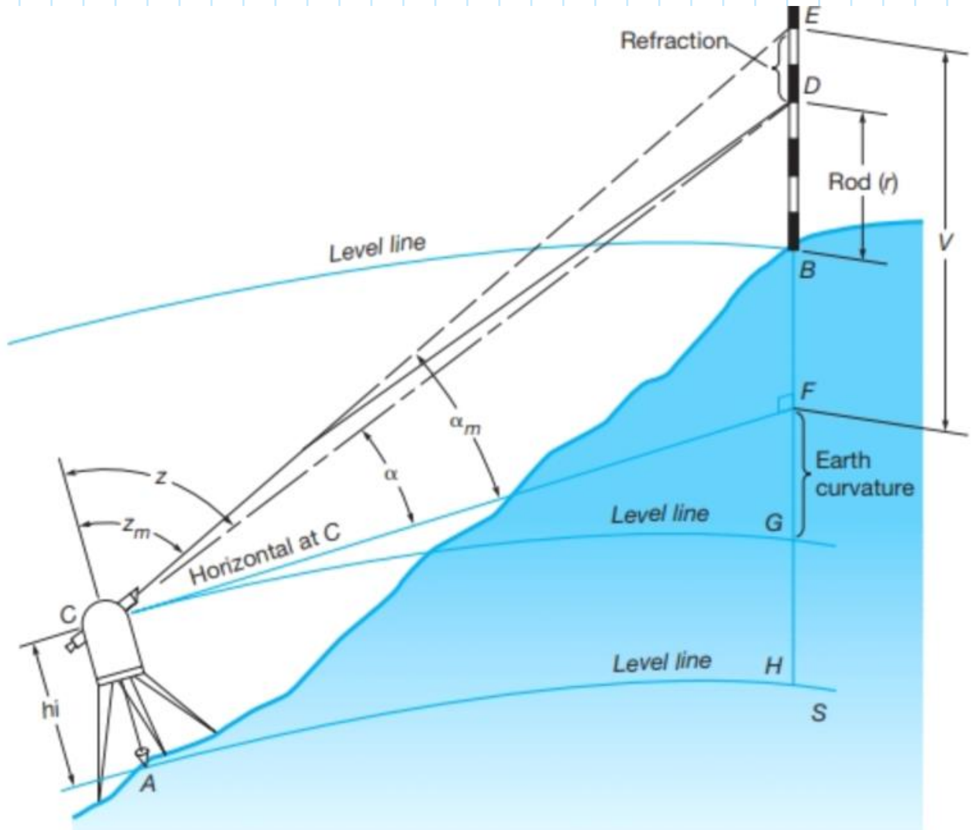

Figure 4.8 Trigonometric leveling—long lines.

$h_{CR} = 0.0206(CE*\sin(z))/1000)^2$

Example 4.1

The slope distance and zenith angle between points *A* and *B* were observed with a total station instrument as 9585.26 ft and 81°42′20″, respectively. The *hi* and rod reading *r* were equal. If the elevation of *A* is 1238.42 ft, compute the elevation of *B*.

Solution

By Equation (4.3a), the curvature and refraction correction is

$$h_{CR} = 0.0206\left(\frac{9585.26 \sin 81°42'20''}{1000}\right)^2 = 1.85 \text{ ft}$$

Trigonometric Leveling - Correction for Refraction and Curvature

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(Theoretically, the horizontal distance should be used in computing curvature and refraction. In practice, multiplying the slope distance by the sine of the zenith angle approximates it.)

By Equations (4.6) and (4.11), the elevation difference is (note that *hi* and *r* cancel)

$$V = 9585.26 \cos 81°42'20'' = 1382.77 \text{ ft}$$
$$\Delta\text{elev} = 1382.77 + 1.85 = 1384.62 \text{ ft}$$

Finally, the elevation of B is

$$\text{elev}_B = 1238.42 + 1384.62 = 2623.04 \text{ ft}$$

Note that if curvature and refraction had been ignored, an error of 1.85 ft would have resulted in the elevation for B in this calculation. Although Equation (4.11) was derived for an uphill sight, it is also applicable to downhill sights. In that case, the algebraic sign of V obtained in Equations (4.6) through (4.9) will be negative because the vertical angles, α or z, will cause the trigonometric functions to return a negative value.

For uphill sights curvature and refraction is added to a positive V to increase

Automatic Level

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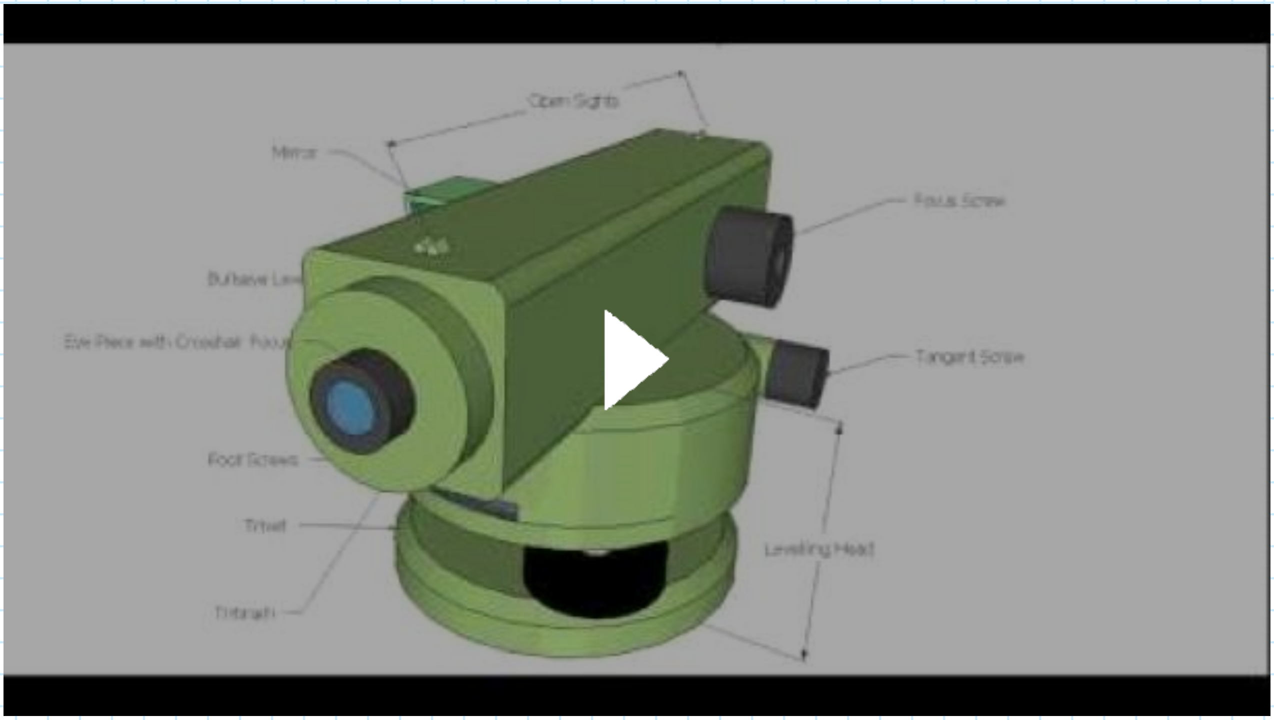
[Parts of an Automatic Level]



Self-Reading Electronic Digital Level

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The newest type of automatic level, the electronic digital level, is pictured in Figure 4.16(a). It is classified in the automatic category because it uses a pendulum compensator to level itself, after an operator accomplishes rough leveling with a circular bubble. With its telescope and cross hairs, the instrument could be used to obtain readings manually, just like any of the automatic levels. However, this instrument is designed to operate by employing electronic digital image processing. After leveling the instrument, its telescope is turned toward a special bar-coded rod [Figure 4.16(b)] and focused. At the press of a button, the image of bar codes in the telescope's field of view is captured and processed. This processing consists of an onboard computer comparing the captured image to the rod's entire pattern, which is stored in memory. When a match is found, the rod reading is displayed digitally. It can be recorded manually or automatically stored in a survey controller.

From <https://online.vitalsource.com/#/books/9781292060675/cfi/110!/4/4@0:0.00>

Specifications

mode)
Accuracy:*1 (with SOKKIA staff BGS40/50 or BIS20/30)
Height Standard deviation for 1 km of double-run
SDL30:
Electronic Measurement
0.6mm (0.03in.) (with BIS20/30)
1.0mm (0.04in.) (with BGS40/50/50G)
1.2mm (0.047in.) (with BAS55)
Visual Measurement
1.0mm (0.04in.) (with BGS40/50/50G)
1.5mm (0.06in.) (with BAS55)

Distance
±10mm (less than 10m measurement)
±(0.1% x D) (10 to 50m measurement)
±(0.2% x D) (more than 50m measurement)
(D: measured distance, unit: m)

Compensating Levels

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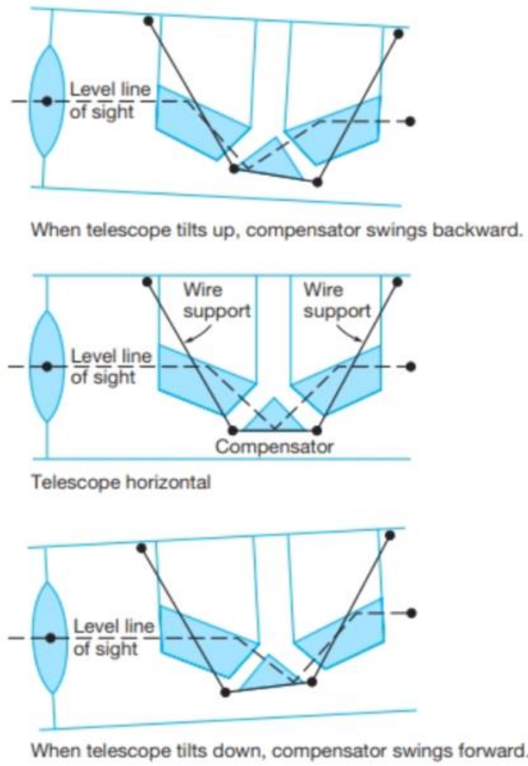
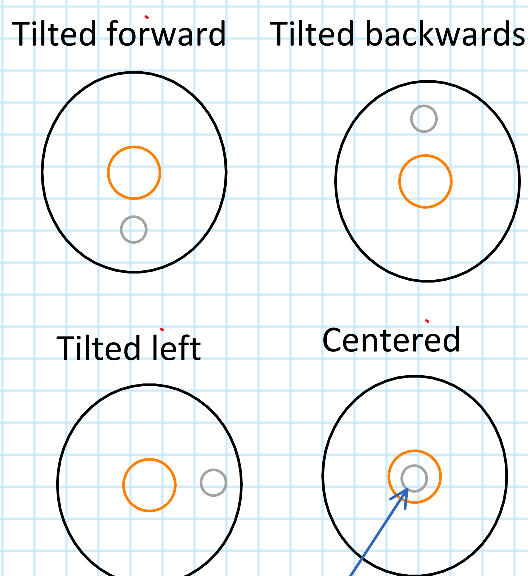


devices shorten the time for the pendulum to come to rest, so the operator does not have to wait.

Circular Bubble Level



If bubble remains in orange circle then compensator will adjust and reading is acceptable.

2 Peg Test for Compensating Levels

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Two-Peg Test: Method for checking and recalibrating a level or transit This method is either for an optical or digital level, or a transit being used as a level. If this error is corrected with a transit, it also improves the accuracy of its vertical angle readings. The two-peg test is very simple, but provides a way to test the accuracy of a level, and if you know which screw to turn (for analog instruments) or menu to follow (for the digital level), you can adjust it to remove the error. See specific instrument instructions for making adjustments.

[Surveying 3 - Two peg test OTEN Building & Construction]

Hand Level

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The hand level (Figure 4.17) is a handheld instrument used on low-precision work, or to obtain quick checks on more precise work. It consists of a brass tube approximately 6 in. long, having a plain glass objective and peep-sight eyepiece. A small level vial mounted above a slot in the tube is viewed through the eyepiece by means of a prism or 45° angle mirror. A horizontal line extends across the tube's center. As shown in Figure 4.18, the prism or mirror occupies only one half of the tube, and the other part is open to provide a clear sight through the objective lens. Thus the rod being sighted, and the reflected image of the bubble, is visible beside each other with the horizontal cross line superimposed. The instrument is held in one hand and leveled by raising or lowering the objective end until the cross line bisects the bubble. Resting the level against a rod or staff provides stability and increases accuracy. This instrument is especially valuable in quickly checking proposed locations for instrument setups in differential leveling.

From <https://online.vitalsource.com/#/books/9781292060675/cfi/112!/4/4@0.00:0.00>

Figure 4.17 Hand level. (Courtesy Topcon Positioning Systems.)

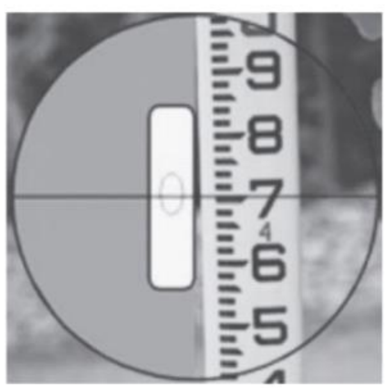

Figure 4.18 View of level rod through a hand level.

Rods

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A variety of level rods are available, some of which are shown in Figure 4.19. They are made of wood, fiberglass, or metal and have graduations in feet and decimals, or meters and decimal

From <https://online.vitalsource.com/#/books/9781292060675/cfi/112!/4/4@0.00:0.00>

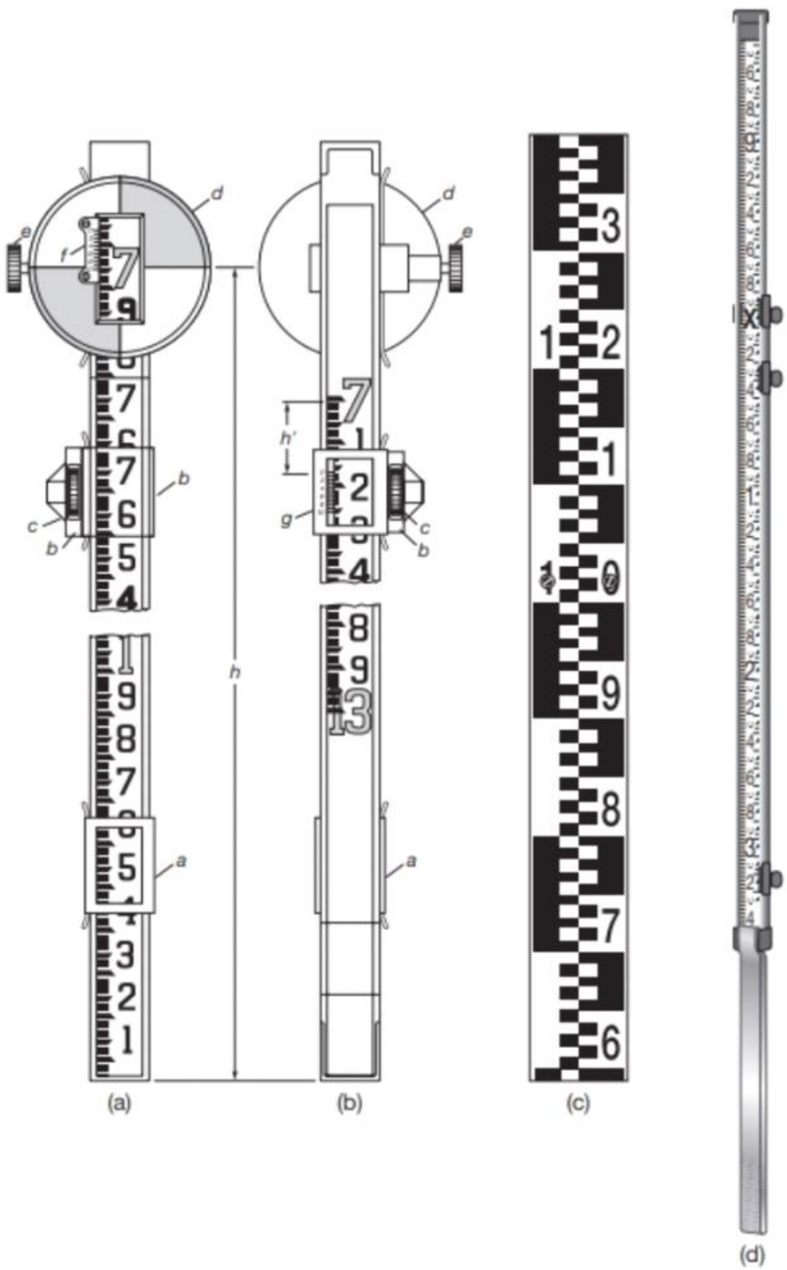

Figure 4.19 (a) Philadelphia rod (front). (b) Philadelphia rod (rear). (c) Double-faced leveling rod with metric graduations. (d) Lenker direct-reading rod. (Courtesy of (c) Leica, Inc., (d) Lenker Manufacturing Company.)

Blank

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