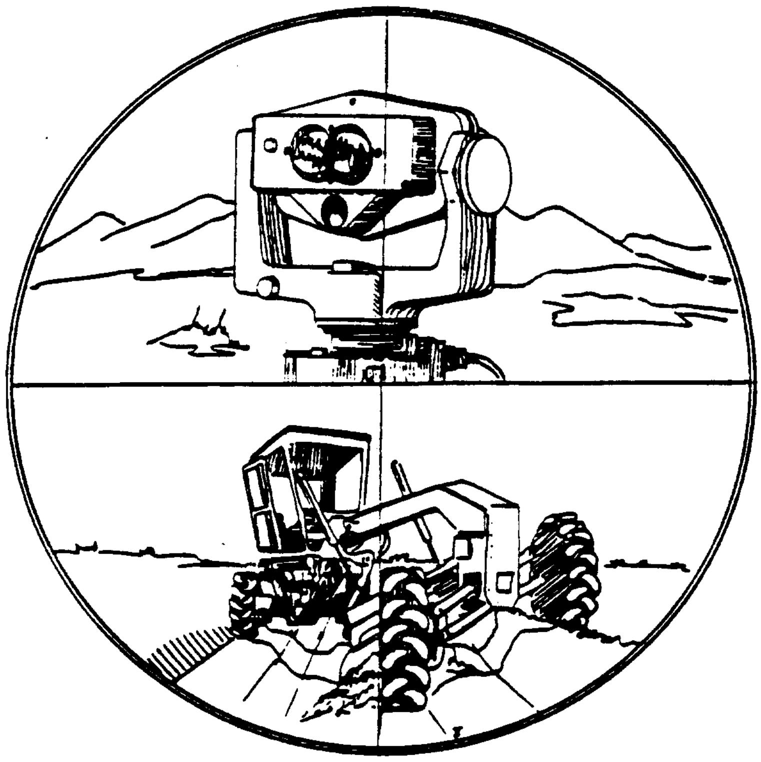
1984

LOCATION AND CONSTRUCTION

SURVEYING GUIDE



U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL DIRECT FEDERAL DIVISION
DENVER, COLORADO

LOCATION/CONSTRUCTION SURVEYING GUIDE

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JUNE 1984

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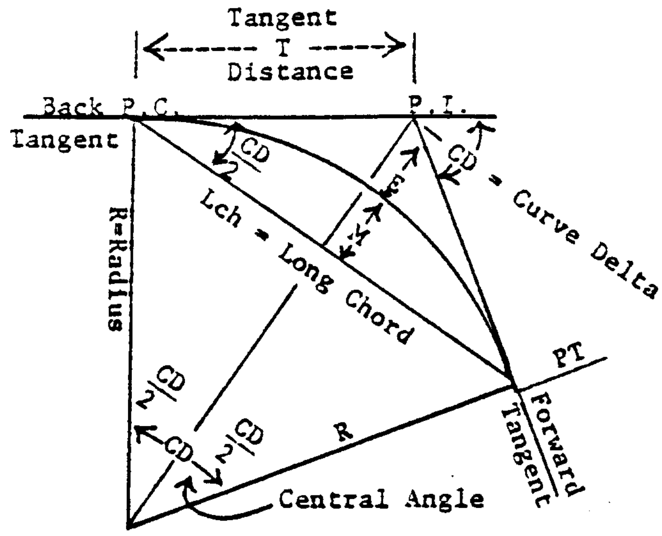
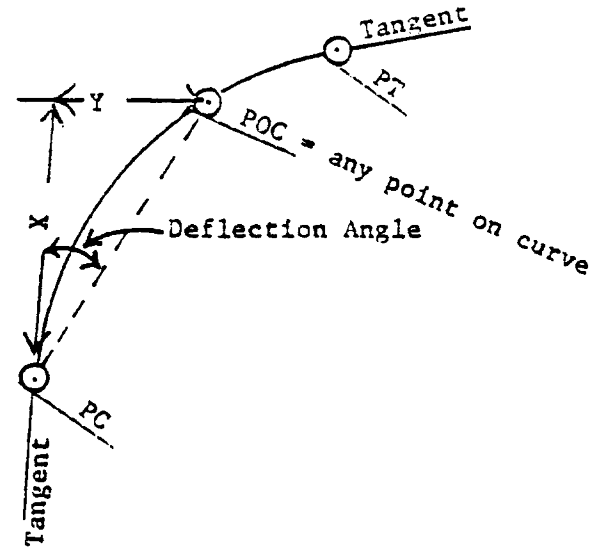
SIMPLE CURVE





E = External
M = Mid-ordinate

The curvature of a simple curve is constant; i.e., the Degree of Curve is a constant. The Degree of Curve is the central angle of a curve 100 feet long.

DC = Degree of Curve
CD = Curve Delta
LC = The Total Length of Curve (in feet)
lc = The length (in feet) of any portion of a curve
Lch = The Long Chord from PC to PT
Ch = Any Chord Length
R = Radius

Deflection angle $= \frac{DC}{200} \times lc$

$R = \frac{18,000}{\pi \times DC}$ Radius = $\frac{18,000}{\text{Pi} \times \text{Degree of Curve}}$

$LC = \frac{CD}{DC} \times 100$ . . . . Length of Curve

Lch = 2 x R x SIN of ½CD . . . . Long Chord

Ch = 2 x R x SIN of the Deflection angle. . . . . Chord Length

T = R x TAN ½CD. . . . . Tangent Distance

$R. = \frac{T}{\text{TAN of } \frac{1}{2}CD}$

E = T x TAN of ¼CD . . . . External Distance

$$E = \frac{R}{\text{COS of } \tfrac{1}{2}\text{CD}} - R$$

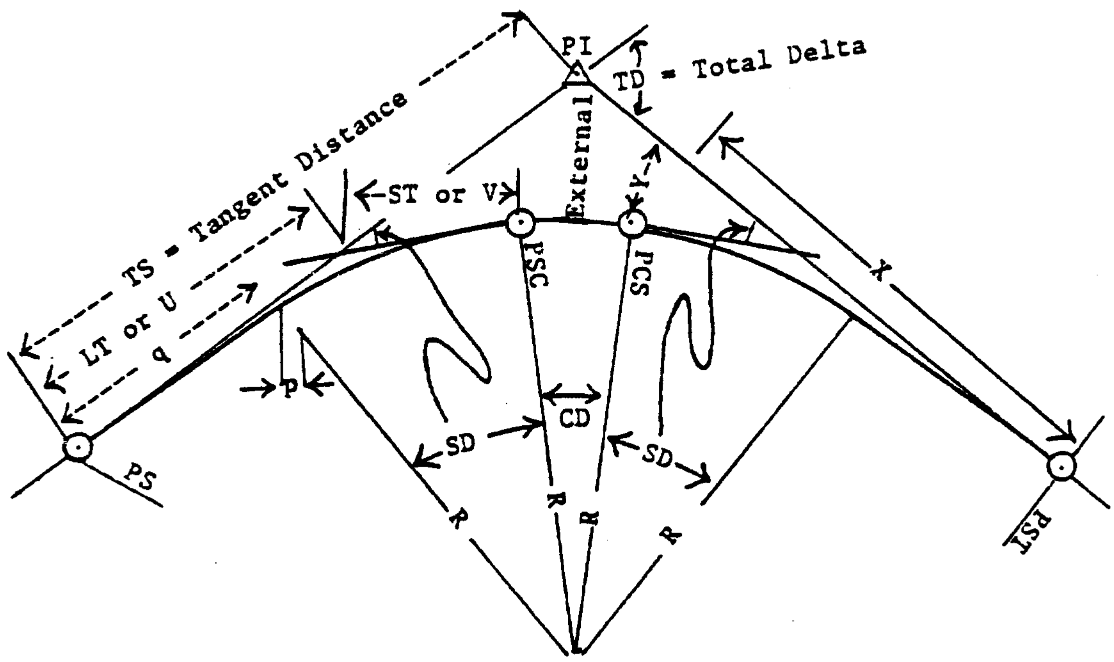
$$R = E \div \left(\frac{1}{\text{COS of } \tfrac{1}{2}\text{CD}} - 1\right)$$

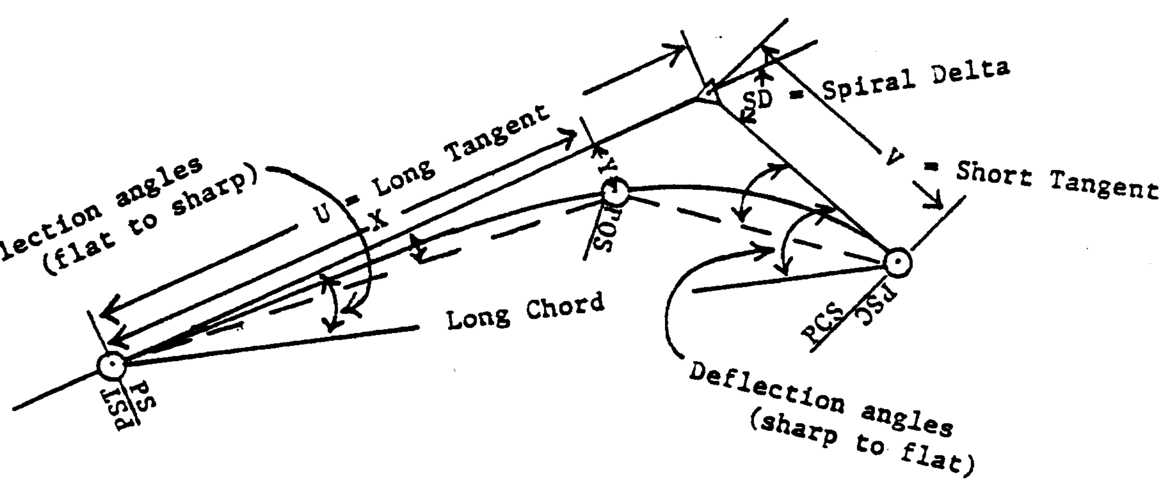
M = R x (1-COS of ½CD) . . . . Mid-ordinate

X = R x SIN of 2 x the deflection angle

Y = R x (1 - COS of 2 x the deflection angle)

SPIRALED CURVES





The curvature of a spiral changes at a constant rate; i.e., the degree of curve varies directly with the distance along a spiral.

DC = The degree of curve at the PSC and PCS
dc = The degree of curve at any point on a spiral POS
LS = The total length of spiral (in feet)
ls = The length (in feet) of any portion of a spiral

Deflection angles $\frac{\text{flat}}{\text{(PS)}}$ to $\frac{\text{sharp}}{\text{(PSC)}} = \frac{DC \times ls^2}{600 \times LS}$

Deflection angle $\frac{\text{sharp}}{\text{(PCS)}}$ to $\frac{\text{flat}}{\text{(PST)}} = \frac{DC \times ls}{200} - \frac{DC \times ls^2}{600 \times LS}$

Subtract the correction, Cs, from the deflection angles for a more accurate value.

Values of the Correction C_s for
Large Spiral Deflections
Formula: C_s (in seconds) = $0.0031\theta_s^3 + 0.0023\theta_s^5(10)^{-5}$,
where θ_s = spiral angle (in degrees)

θ_s	C_s	θ_s	C_s	θ_s	C_s	θ_s	C_s
5°	0.4"	14°	8.5"	23°	0'37.9"	32°	1'42.4"
6	0.7	15	10.5	24	43.0"	33	1'52.3"
7	1.1	16	12.7	25	48.7"	34	2'02.9"
8	1.6	17	15.3	26	54.8"	35	2'14.1"
9	2.3	18	18.1	27	1'01.3"	36	2'26.0"
10	3.1	19	21.3	28	1'08.4"	37	2'38.0"
11	4.1	20	24.9	29	1'16.1"	38	2'51.9"
12	5.4	21	28.8	30	1'24.3"	39	3'06.0"
13	6.8	22	33.1	31	1'33.0"	40	3'20.8"

$S = \frac{ls}{200} \times dc$; where: S = spiral angle
$dc = \frac{DC}{LS} \times ls$
ls (in feet) is from the PS/PST

$SD = \frac{LS}{200} \times DC$

The Chord distance from the PS/PST to any point on a spiral (POS) can be determined by multiplying the length of spiral (ls) in feet from the PS/PST multiplied times the C/L value from Table IV that corresponds with either a "i" value (the deflection angle of the POS computed from the PS/PST) or a "S" value where:

$S = \frac{ls^2 \times DC}{200 \times LS}$ where: ls (in feet) is from the PS/PST
DC is the Degree of Curve at the PSC/PCS
LS (in feet) is the total Length

SPIRALED CURVES

$$SD = \frac{LS}{200} \times DC$$

The Chord distance from the PS/PST to any point on a spiral (POS) can be determined by multiplying the length of spiral (ls) in feet from the PS/PST multiplied times the C/L value from Table IV that corresponds with either a "i" value (the deflection angle of the POS computed from the PS/PST) or a value of S where:

$$S = \frac{ls^2 \times DC}{200 \times LS}$$

where: ls (in feet) is from the PS/PST
DC is the Degree of Curve at the PSC/PCS
LS (in feet) is the total Length of Spiral

For the total Long Chord distance from the PS/PST to the PSC/PCS SD = S and LS = ls

The total Long Chord distance can also be determined from the North and East coordinates of the PS/PST and the PSC/PCS.

$$Lch = \sqrt{(East_{psc} - East_{ps})^2 + (North_{psc} - North_{ps})^2}$$

or by using the Polar/Rectangular Conversion calculator function with:

$(East_{psc} - East_{ps})$ in the Y-register
$(North_{psc} - North_{ps})$ in the X-register

The Long tangent "U" and the Short tangent "V" can be determined using Table IV; i.e., by multiplying the total Length of Spiral--LS--(in feet) multiplied times the U/L -- V/L value from Table IV that corresponds with a S value of SD.

Tangent Distance--TS--for Spiraled Curves with equal spirals:

$$TS = (R + p) \times TAN \text{ of } \tfrac{1}{2}TD + q$$

where R = Radius of the DC at the PSC/PCS = $\frac{18,000}{\pi \times DC}$
TD = Total Delta
p and q are determined from Table IV; S = SD

Tanget Distances TS_1 and TS_2 for Spiraled Curves with unequal spirals:

$$TS_1 = q_1 + (R + p_2) \times TAN \text{ of } \tfrac{1}{2}TD - \frac{(P_1 - P_2)}{TAN \text{ of } TD}$$

$$TS_1 = q_1 + (R + p_2) \times \text{TAN of } \tfrac{1}{2}TD + \frac{(P_1 - P_2)}{\text{SIN of } TD}$$

LS_1 = Longer Spiral

SPIRALED COMPOUND CURVES

DC_2 = the sharper curve

$$\text{The Compound Spiral Delta SD} = \frac{LS_c \times DC_1}{200} + \frac{LS_c \times DC_2}{200}$$

Where LS_c (in feet) is the total length of the compound spiral.

Compound Spiral Deflection angles (flat to sharp) -- PSC to PCS

$$\text{Deflection angle} = \frac{DC_1 \times ls}{200} + \frac{(DC_2 - DC_1) \times ls^2}{600 \times LS_c}$$

Where ls (in feet) is from the PSC if DC_2 is sharper than DC_1.

Compound Spiral Deflection angles (sharp to flat) -- PCS to PSC

$$\text{Deflection angle} = \frac{DC_2 \times ls}{200} - \frac{(DC_2 - DC_1) \times ls^2}{600 \times LS_c}$$

Where ls (in feet) is from the PCS if DC_2 is sharper than DC_1.

In the equations for TS_1 and TS_2, the angles I_1 and I_2 are equal to:

$$I_1 = SD_1 + CD_1 + \frac{LS_c \times DC_1}{200}$$

$$I_2 = SD_2 + CD_2 + \frac{LS_c \times DC_2}{200}$$

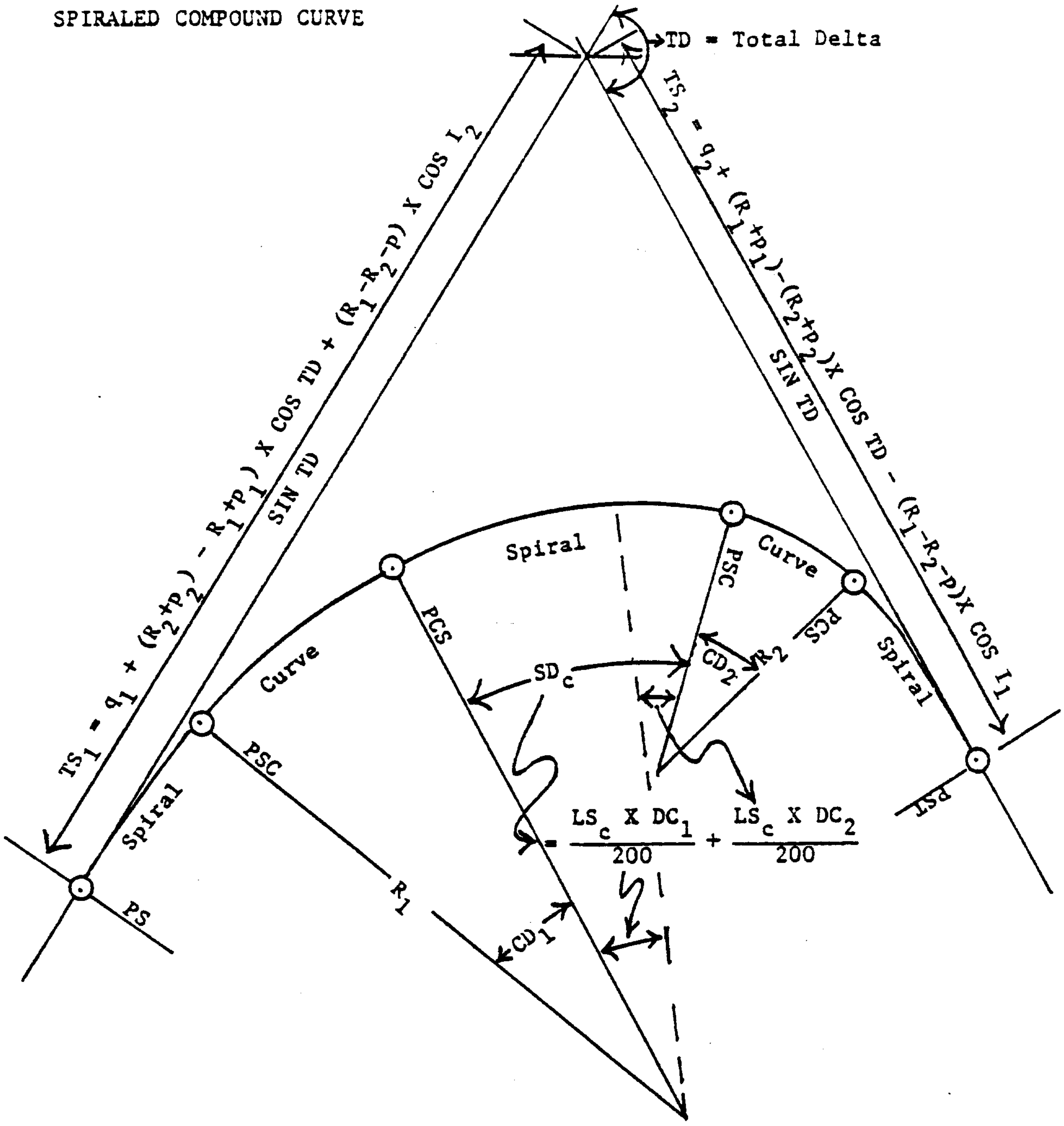
The Spiral angle "S" that corresponds with the p/L and q/L values in Table IV:

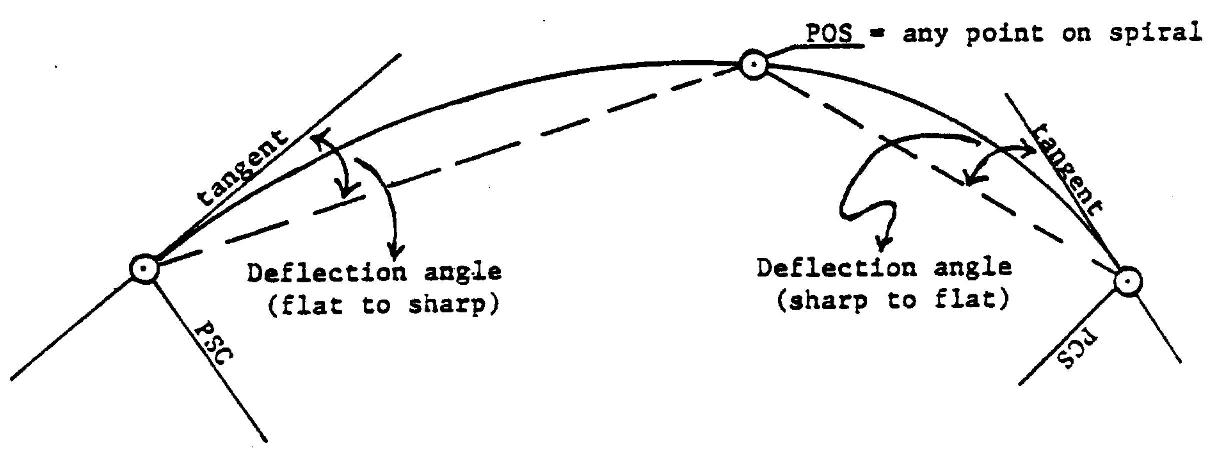
$$S = \frac{DC_2 - DC_1}{200} \times LS_c$$

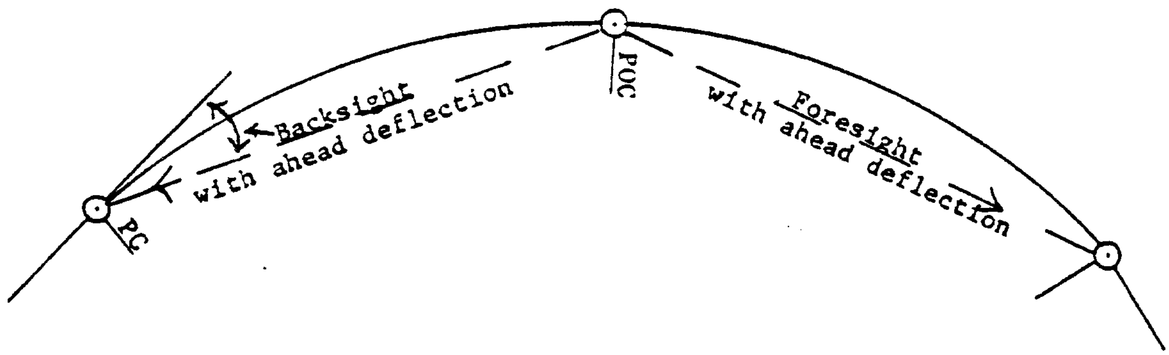
The Long Chord of a Compound Spiral can be determined from the North and East Coordinates of the PSC and PCS.

$$Lch = \sqrt{(East_{pcs} - East_{psc})^2 + (North_{pcs} - North_{pcs})^2}$$

SPIRALED COMPOUND CURVE







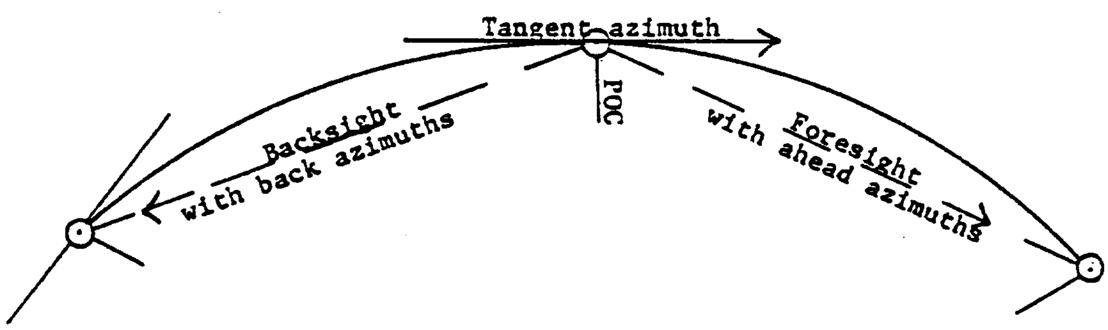
The transit book used for staking centerline has the deflection angles computed for every 50 foot station. If a POC is required, the only deflection angle computation necessary is that used to set a POC if it is other than a 50 foot station.

Deflection angle = $\frac{DC}{200}$ x cl

Where: DC = Degree of Curve
cl = Curve length (in feet) from PC to POC

With the instrument at the POC:

1. Backsight with the scope inverted and the ahead deflection angle of the backsight station locked in (0^{o} if backsight station is the PC). Continue to run in the curve using the ahead deflection angles as computed from the PC.



2. Figure the deflection angle of the POC.

   Deflection angle = $\frac{DC}{200}$ x cl

   Where: DC = Degree of Curve
   cl = Curve length (in feet) from PC to POC

3. Figure the set azimuth of the POC by adding if right (subtracting if left) the deflection angle to the tangent azimuth at the PC.

4. Figure the tangent azimuth at the POC by adding if right (subtracting if left) the deflection angle of the POC to the set azimuth of the POC.

5. Figure the deflection angles for running in the remaining curve:

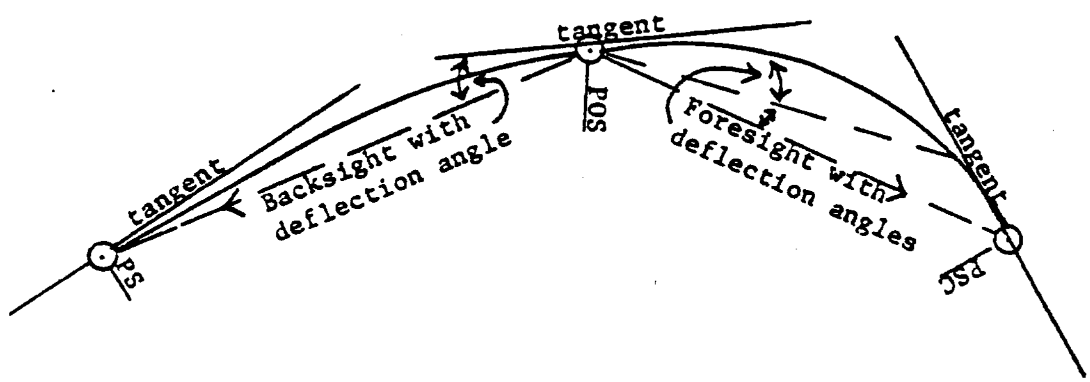
$$\text{Deflection angle} = \frac{DC}{200} \times cl$$

Where: DC = Degree of Curve
cl = Curve length (in feet) from POC

6. Figure the set azimuths for running in the remaining curve by adding if right (subtracting if left) the deflection angles to the tangent azimuth at the POC.

When running in a curve from a POC using azimuths, backsight with the scope right side up and the set azimuth of the POC +/- 180° set on the horizontal angle scale that measures angle to the right. Turn to the set azimuths of the remaining curve stations.

Running in a simple curve using azimuth is best suited to the use of a single vernier instrument such as a T-2 or a HP 3810.



A procedure for figuring a POS:

1. Compute the degree of curve (dc) at the POS.

$$dc = \frac{DC}{LS} \times ls$$

Where: DC = Degree of Curve at the PSC/PCS
LS = Total Length (in feet) of Spiral PS/PST to PSC/PCS
ls = length (in feet) of spiral PS to POS
dc = degree of curve at the POS

2. Figure the deflection angles for running in the spiral from the POS.

$$\text{deflection angles } \frac{\text{flat to}\quad\text{sharp}}{\text{POS}\quad\text{to PSC/PCS}} = \frac{dc}{200} \times ls + \frac{DC \times ls^2}{600 \times LS}$$

$$\text{deflection angles} \quad \frac{\text{sharp to flat}}{\text{POS to PS/PST}} = \frac{dc}{200} \times ls - \frac{DC \times ls^2}{600 \times LS}$$

Where: dc = degree of curve at the POS
ls = length (in feet) of spiral from the POS
DC = Degree of Curve at the PSC/PCS
LS = Total Length (in feet) of Spiral PS/PST to PSC/PCS

To run in a spiral from a POS using deflection angles backsight with the scope inverted and the deflection angle of the backsight station figured from the POS locked in; i.e., if the ahead deflection angles are right deflections and the backsight is the PS, then the deflection angle of the PS figured from the POS is left and is locked in on the left vernier scale.

To run in a spiral using azimuths:

3. Figure the set azimuth of the POS by adding if right (subtracting if left) the deflection angle of the POS figured from the PS, to the tangent azimuth at the PS.

4. Figure the tangent azimuth at the POS by adding if right (subtracting if left) the deflection angle of the PS figured from the POS, to the set azimuth of the POS.

5. Figure the set azimuths for running in the spiral from the POS by adding if right (subtracting if left) the deflection angles figured from the POS, to the tangent azimuth at the POS.

Backsight with the ahead set azimuth of the POS +/- 180° and the scope right side up. Turn to the set azimuth of the remaining spiral stations.

COORDINATE COMPUTATIONS

The computation of traverse coordinates is best broken down into four steps:

1. Compute the azimuth of each leg of the traverse from angles right.

2. Azimuth adjustment--by adjusting the angles right as necessary.

3. Compute the horizontal distance of each leg as necessary using the appropriate slope reduction method.

4. Compute the North and East coordinates of each point.

COMPUTATION OF AZIMUTHS FROM ANGLES RIGHT

Given a traverse with points T-1, T-2, T-3, . . . T-n with the azimuth from T-1 to T-2 known and the angles right measured at each point; for example:

BS = T-1 ⊼=T-2 FS = T-3

BS = T-2 ⊼=T-3 FS = T-4 . . .

BS = T-(n-1) ⊼=T-(n) FS = T-(n+1)

Azimuth T-1 to T-2 + or - 180° + the angle right at T-2 = Azimuth T-2 to T-3; then,

Azimuth T-2 to T-3 + or - 180° + the angle right at T-3 = Azimuth T-3 to T-4 . . .

Azimuth T-(n-1) to T-n +/- 180° + the angle right at T-n = Azimuth T-n to T-(n+1)

NOTE: If the computed azimuth is greater than 360°, subtract 360°. If it is negative, add 360°.

COMPUTATION OF NORTH AND EAST COORDINATES

The coordinates of T-1 = ($North_1$, $East_1$); coordinates of T-2 = ($North_2$, $East_2$); . . . coordinates of T-n = $North_n$, $East_n$)

The north coordinate of T-2 = the North coordinate of T-1 - (the horizontal distance T-1 to T-2 multiplied times the COS of the Azimuth T-1 to T-2); i.e., $North_2$ = $North_1$ - (horizontal distance x COS AZ of T-1 to T-2)

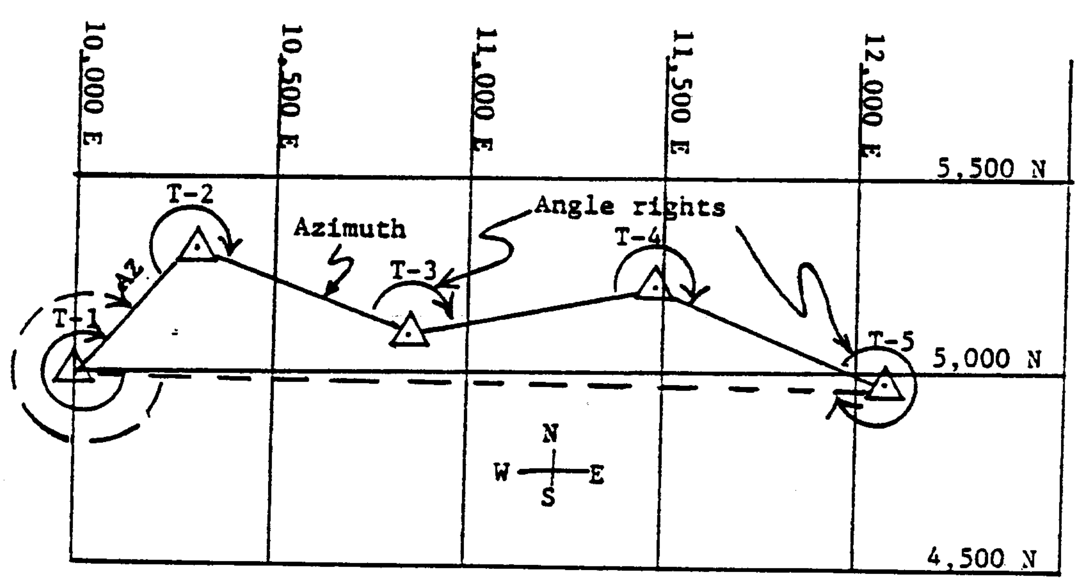
$North_{(n+1)}$ = $North_n$ - [horizontal distance x COS AZ to T-n to T-(n+1)]

The East Coordinate of T-2 = East coordinate of T-1 - [the horizontal distance T-1 to T-2 multiplied times the SIN of the Azimuth T-1 to T-2]; i.e., $East_2$ = $East_1$ - (horizontal distance x SIN AZ of T-1 to T-2).

$East_{(n+1)}$ = $East_n$ - [horizontal distance x SIN AZ of T-n to T-(n+1)]

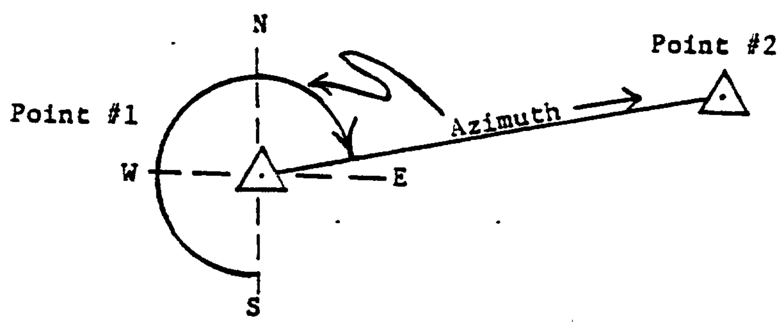
The North and East coordinates can be computed simultaneously with some calculators using the Polar/Rectangular conversion function.

TRAVERSE COORDINATES



POINT	ANGLE RT.	AZIMUTH	HORIZONTAL DISTANCE	COORDINATE NORTH	COORDINATE EAST
T-5					
		90° 10'00"			
T-1	319° 24'17"			5,000.000	10,000.000
		229° 34'17"	502.02		
T-2	245° 14'50"			5,325.56	10,382.11
		294° 49'07"	533.21		
T-3	145° 18'32"			5,101.75	10,866.07
		260° 07'39"	637.768		
T-4	211° 08'58"			5,211.10	11,494.39
		291° 16'37"	598.229		
T-5	338° 53'23"			4,994.02	12,051.84
		90° 10'00"	2,051.882		
T-1				4,999.99	9,999.97

TIE COMPUTATIONS



Computing the distance and azimuth between any two points can be done simultaneously using any model of Hewlett-Packard calculator with the Rectangular/Polar conversion function if the North and East coordinates of the two points are known.

The calculating procedure is as follows:

East coordinate of point No. 2	ENTER
East coordinate of point No. 1	—
North coordinate of point No. 2	ENTER
North coordinate of point No. 1	—
Activate Rectangular to Polar Conversion	→P

(The X-value is the horizontal distance between the two points)

Exchange X and Y	X ⇆ Y

Add 180°	180 +
(This is the azimuth in degrees from point No. 1 to point No. 2)	
Change to °, ′, ″	H.MS

AZIMUTH AND COORDINATE COMPUTATIONS

AZ (1) - Given - 229°34′17″

AZ (2) 229°34′17″ - 180 + 222°44′26″ = 272°18′43″

AZ (3) 272°18′43″ - 180 + 164°04′53″ = 256°23′36″

AZ (4) 256°23′36″ - 180 + 1°49′21″ = 78°12′57″

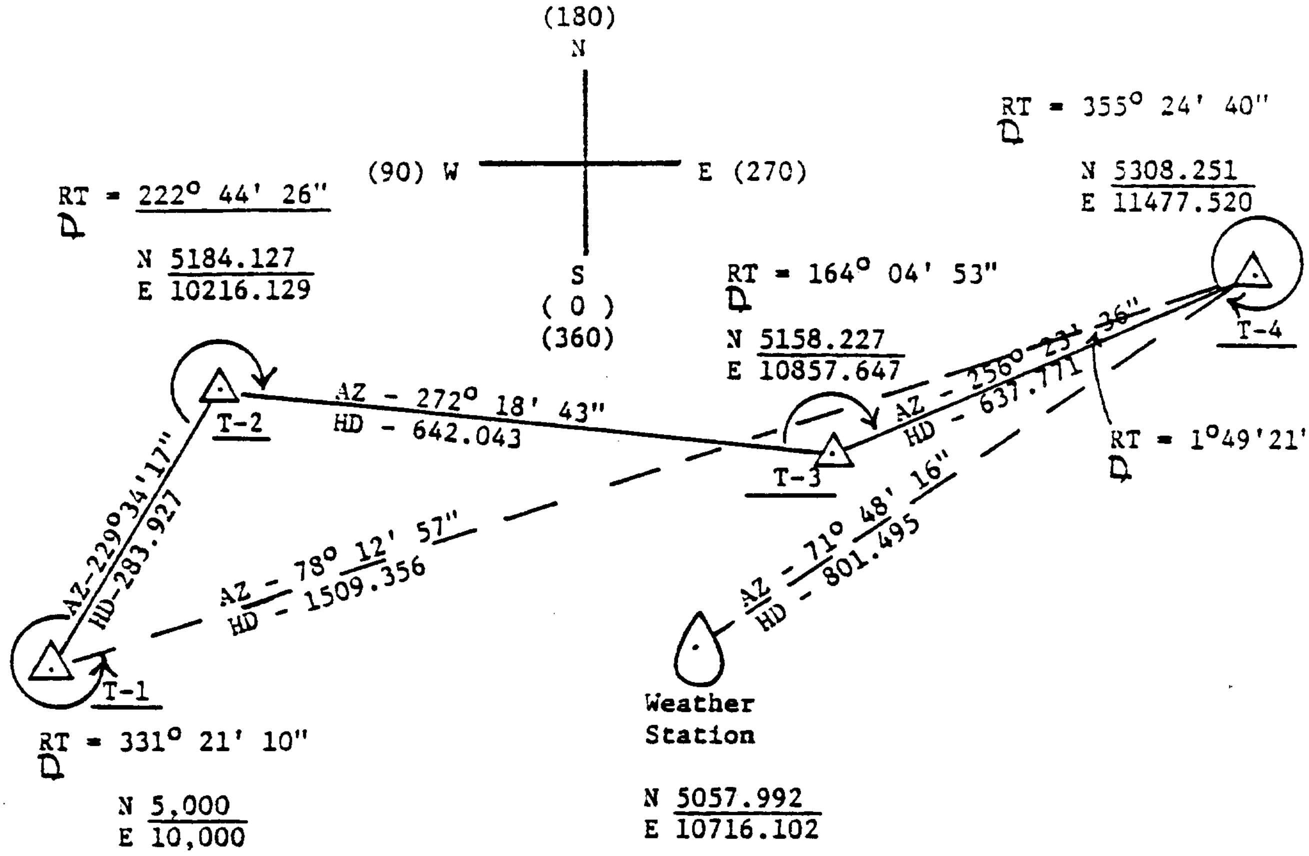
AZ (5) 256°23′36″ - 180 + 355°24′40″ = 71°48′16″

NORTH COORDINATES

1	Given	5,000						
2	COS	229°34′17″	x	283.927	+	5,000	=	5,184.127
3	COS	272°18′43″	x	642.043	-	5,184.127	=	5,158.227
4	COS	256°23′36″	x	637.771	+	5,158.227	=	5,308.251
1	COS	78°12′57″	x	1,509.356	-	5,308.251	=	5,000.002
Weather Station	COS	71°48′16″	x	801.495	-	5,308.251	=	5,057.975

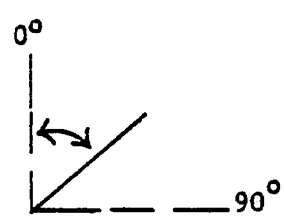
EAST COORDINATES

1	Given	10,000						
2	SIN	229°37′17″	x	283.927	+	10,000.000	=	10,216.129
3	SIN	272°18′43″	x	642.043	+	10,216.129	=	10,857.647
4	SIN	256°23′36″	x	637.771	+	10,857.647	=	11,477.520
1	SIN	78°12′57″	x	1,059.356	-	11,477.520	=	9,999.975
Weather Station	SIN	71°48′16″	x	801.495	-	11,477.520	=	10,716.102

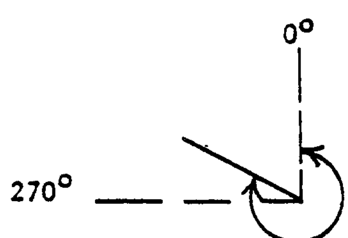


REDUCTION TO HORIZONTAL

The vertical circle readings taken with a Theodolite (T-16 or T-2) are either the Zenith angle or 360°- the Zenith angle; i.e., 0 degrees is at the Zenith, straight up. The vertical circle reading is equal to the Zenith angle when the scope is in the righted position (vertical circle right of the scope) then 90° is horizontal. The vertical circle reading is equal to 360°- the Zenith angle when the scope is inverted (vertical circle left of the scope) then 270° is horizontal.



Vertical angle reading (scope right side up)



Vertical angle reading (scope inverted)

HORIZONTAL AND VERTICAL DISTANCES FROM SLOPE DISTANCE AND VERTICAL ANGLE READINGS

Horizontal distance = Slope distance multiplied times the SIN of the vertical angle reading

NOTE: If the vertical angle reading taken with the scope inverted is used, then the horizontal distance will be negative.

Vertical distance = Slope distance multiplied times the COS of the vertical angle reading

NOTE: The vertical distance will have the proper sign using the vertical angle reading taken with the scope in either position.

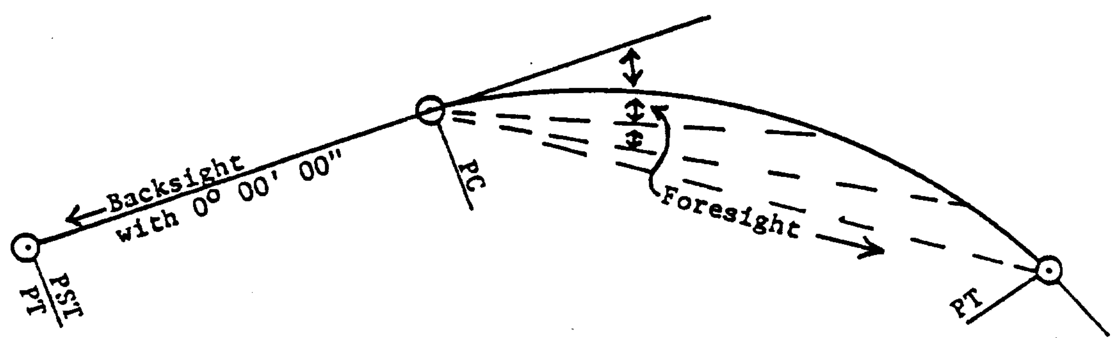
The horizontal and vertical distance can be computed simultaneously with some calculators using the Polar/Rectangular conversion function.

HORIZONTAL DISTANCE FROM ELEVATIONS AND SLOPE DISTANCE

$$\text{Horizontal distance} = \sqrt{(\text{slope distance})^2 - (\text{elev.}_2 - \text{elev.}_1)^2}$$

SLOPE DISTANCE FROM HORIZONTAL DISTANCE AND VERTICAL ANGLE READINGS

$$\text{Slope distance} = \frac{\text{Horizontal distance}}{\text{SIN of the vertical angle reading}}$$



Running in a simple curve from the PC using deflection angles.

Backsighting any transmit point on tangent back of the PC; i.e., (PT/PST of previous curve or any POT).

1. Backsight with the slope inverted and 0°00' locked in on the horizontal angle scales.

2. Unlock the horizontal angle scale, right (plunge) the scope, and run in the curve using the ahead deflection angles as computed from the PC.

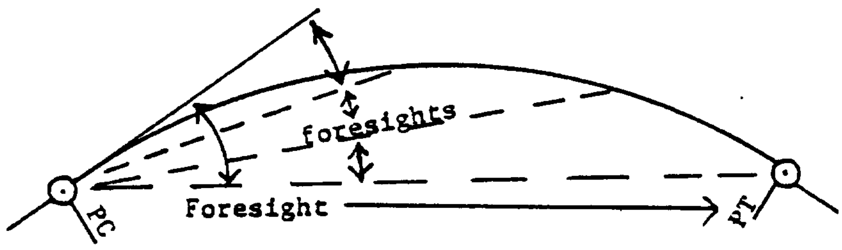
3. To reduce instrument error of an instrument in poor adjustment, for better precision you should double on the PT as follows:

   a. Turn the instrument and re-backsight (note the scope will be in the upright position).

   b. The horizontal angle scale should read 180°00'. If not, lock in 180° and re-backsight.

   c. Unlock the horizontal angle scales, plunge the scope, and turn to the ahead deflection angle of the PT + 180° (note the scope will be in the inverted position).

The same procedure applies when running in a simple curve from the PT using the back deflection angles and backsighting any tangent point ahead of the PT.



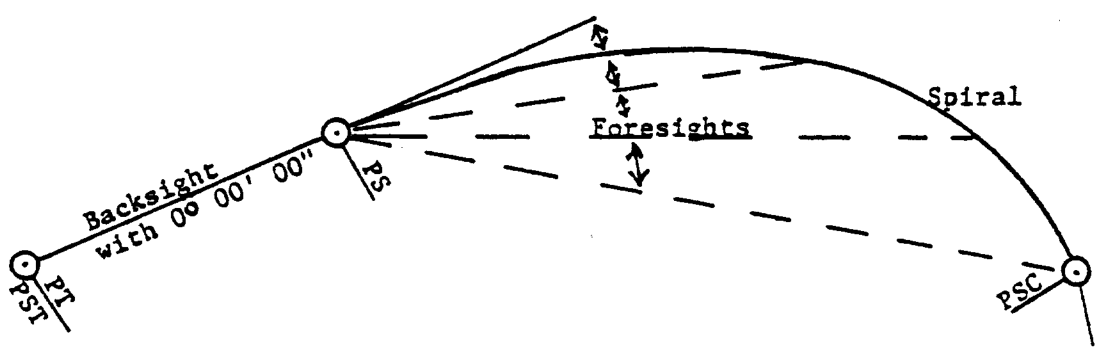
Running in a simple curve from the PC and foresighting the PT.

1. Foresight the PT with the scope upright and the ahead deflection angle of the PT locked in on the horizontal angle scales.

2. Unlock the horizontal angle scales and run in the curve using the ahead deflection angles.

NOTE: When using a foresight there is no real reason to double except to verify your first deflection angle reading.

The same procedure applies when running in a simple curve from the PT and foresighting the PC.



Running in a spiral from the PS using deflection angles when backsighting any transit point on tangent back of the PS; i.e., (PST/PT of previous curve or any POT).

1. Backsight with the scope inverted and 0°00′ locked in on the horizontal angle scales.

2. Unlock the horizontal angle scales and right (plunge) the scope and run in the sprial using the ahead deflection angles.

3. Instrument error can be minimized by doubling on the PT as follows:

   a. Turn the instrument and re-backsight (note the scope will be in the upright position).

   b. The horizontal angle scale should read 180°00′; if not, lock in 180°00′ and re-backsight.

   c. Unlock the horizontal angle scales, plunge the scope, and turn to the ahead deflection angle of the PSC + 180° (note the scope will be in the inverted position).

This same procedure applies when running in a spiral from the PST backsighting any tangent point ahead of the PST.

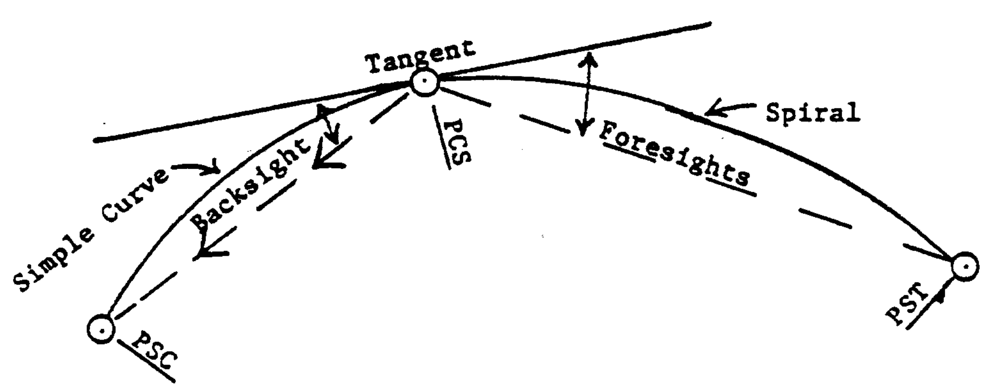
Running in a spiral from the PS using deflection angle foresighting the PSC.

1. Foresight the PSC with the scope upright and the ahead deflection angle locked in on the horizontal angle scale.

2. Unlock the horizontal angle scales and run in the spiral using the ahead deflection angles.

NOTE: When using a foresight there is no real reason to double except to verify your horizontal angle readings.

This same procedure applies when running in a spiral from the PST and foresighting the PCS.



Running in a spiral from the PCS using deflection angles when backsighting the PSC.

1. Backsight the PSC with the scope inverted and the back deflection angle of the PSC (as computed from the PCS) locked in on the appropriate horizontal angle scale.

2. Unlock the horizontal angle scales, right (plunge) the scope and run in the spiral using the ahead deflection angels.

3. To reduce the instrument error of an instrument in poor adjustment and for better precision you should double on the PST as follows:

   a. Turn the instrument and re-backsight on the PSC (note the scope will be in the upright position).

   b. The horizontal angle scale should read the back deflection angle of the PSC + 180°. If not, lock in the back deflection angle of the PSC + 180° and re-backsight the PSC.

   c. Unlock the horizontal angle scales, plunge the scope, and turn to the ahead deflection angle of the PST + 180° (note: the scope will be in the inverted position).

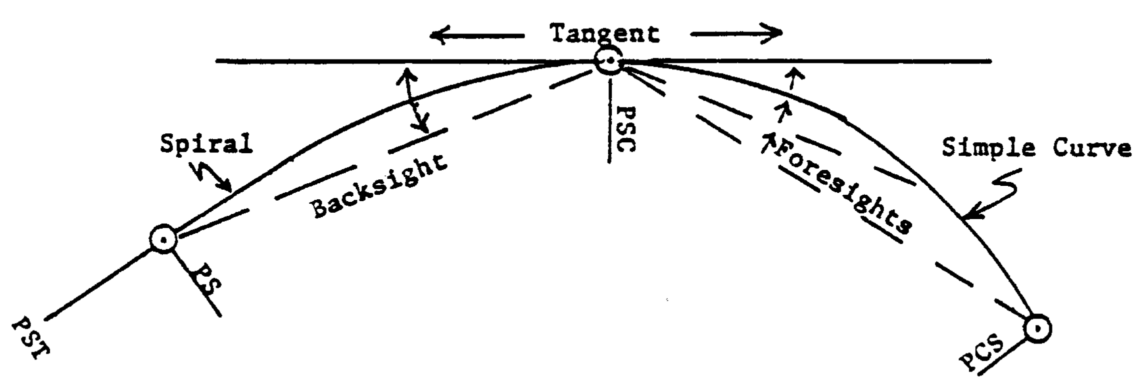
The same procedure applies to running in a spiral from the PSC when backsighting the PCS using the back deflection angles.

Running in a spiral from the PCS when foresighting the PST.

1. Foresight the PST with the scope in the upright position and the ahead deflection angle of the PST (as computed from the PCS) locked in on the appropriate horizontal angle scale.

2. Unlock the horizontal angle scales and run in the spiral using the ahead deflection angles.

3. When using a foresight there is no real reason to double except to verify your first deflection angle reading.



Running in the simple curve from the PSC using deflection angles when backsighting the PS:

1. Backsight the PS with the scope inverted and the back deflection angle of the PS (as computed from the PSC) locked in on the appropriate horizontal angle scale.

2. Unlock the horizontal angle scales, right (plunge) the scope, and run in the curve using the ahead deflection angles.

3. Instrument error can be minimized by doubling on the PCS as follows:

   a. Turn the instrument and re-backsight on the PS (note the scope will be in the righted position).

   b. The horizontal angle scale should read the back deflection angle of the PS + 180°. If not, lock in the back deflection angle of the PS + 180° and re-backsight the PS.

   c. Unlock the horizontal angle scale, plunge the scope, and turn to the ahead deflection angle of the PCS + 180° (note: the scope will be in the inverted position).

This same procedure applies when running in the curve from the PCS (using the back deflection angles) and backsighting the PST.

Running in the simple curve from the PSC foresighting the PCS:

1. Foresight the PCS with the scope upright and the ahead deflection angle of the PCS locked in on the appropriate horizontal angle scale.

2. Unlock the horizontal angle scales and run in the curve using the ahead deflection angles.

3. When using a foresight there is no real reason to double except to verify your first deflection angle reading.

This same procedure applies when running in the simple curve from the PCS and foresighting the PSC.

To run centerline using azimuth:

1. Figure the set azimuths of each station by adding if right (subtracting if left) the deflection angles of each station to (from) the tangent azimuth of the transit point occupied by the instrument.

2. Backsight with the scope in the upright position and the ahead set azimuth plus or minus 180° of the transit point occupied locked in on the horizontal angle scale that measures to the right.

3. Unlock the horizontal scales and turn to the set azimuth of each station.

4. Doubling

   a. Plunge the scope and re-backsight (note: the scope will be inverted).

   b. The horizontal angle reading on the scale that measures to the right should read the ahead set azimuth of the transit point occupied, if not, lock in the ahead set azimuth of transit point occupied and re-backsight.

   c. Unlock the horizontal angle scales and turn to the ahead set azimuth of the transit point being set plus 180°.

SPIRAL FORMULAS

Flat/Sharp Spirals

$$\text{DEFL.Angle} = \frac{\text{D.C. x ls}^2}{\text{600 x Ls}} \quad \text{or} \quad \frac{\text{S.D.}}{\text{3 x (LS}^2\text{)}} \text{ x ls}^2.$$

$$\text{Long Chord} = \frac{36{,}000}{\pi \text{ x D.C. x .517}} \text{ x SIN (.005 x DC x Ls x .517)}$$

Sharp/Flat Spirals

$$\text{DEFL.Angle} = \frac{\text{DC x ls}}{200} - \frac{\text{DC x ls}^2}{\text{600 x LS}}$$

P.O. Spirals

$$\text{dc (@ POS)} = \frac{\text{DC}}{\text{LS}} \text{ x ls}$$

Flat/Sharp

$$\text{DEFL.Angle} = \frac{\text{dc x ls}}{200} + \frac{\text{DC x ls}^2}{\text{600 x ls}}$$

$$\text{Chord} = \text{2 x (R) x SIN of } \frac{\text{(C.D.)}}{2}$$

$$\text{(R)} = \frac{18{,}000}{\pi \text{ x dc}}$$

$$\text{(CD)} = \frac{\text{LS x dc}}{100}$$

Sharp/Flat

$$\text{DEFL Angle} = \frac{dc \times ls}{200} - \frac{DC \times ls^2}{600 \times LS}$$

COMPOUND SPIRALS

Flat/Sharp

$$\text{DEFL Angle} = \frac{DC \times ls}{200} + \frac{(DC_2 - DC_1) \times ls^2}{600 \times Ls_c}$$

Sharp/Flat

$$\text{DEFL Angle} = \frac{DC_2 \times ls}{200} - \frac{(DC_2 - DC_1) \times ls^2}{600 \times LSc}$$

CDFD 16-001

SURVEY CREW - DAILY RECORD

WEATHER: ______________________ DATE: ____________

DAY OF WEEK:

Su	M	T	W	Th	F	S

PROJECT: ______________________

SECTION OR AREA WORKED: ______________________

CREW MEMBERS	DUTIES	HOURS WORKED
		to
		to
		to
		to
		to
		to

PRODUCTION SUMMARY

TYPE OF SURVEY	STATION TO STATION

COMMENTS: ______________________

____

____

SIGNATURE ______________________

NOTE KEEPING

The keeping of good survey notes is of utmost importance because they: (1) present the entire record of a survey activity; (2) may be used by people unfamiliar with the work done on a survey project; (3) may be introduced as evidence in future legal action; (4) are used for determining pay quantities; and (5) reflect the quality of work done during the course of the survey. All too often, survey notes are inadequate as crew members get in the habit of recording just a minimum of information, often in a messy and illegible fashion. It is the party chiefs responsibility to exercise his/her initiative in teaching the other crew members the requirements for good notekeeping and maintaining a high standard of quality.

A few principles that apply to all forms of notekeeping are: (1) be complete; (2) be neat; (3) be concise; (4) avoid copying as much as possible; (5) mark data for easy reference; and (6) DOT NOT ERASE--mark through it so it can still be read.

Be Complete. It is true there is never too much information in survey notes if it is presented in the proper manner. The most common omission is the names of the crew members, their jobs, the date, and the weather. It is good practice to form a habit of entering this information everyday regardless of how obvious it seems at the time that the same personnel are entered day after day. Notes should be completed in the field and not left in part to be filled in later in the office.

Be Neat. This point cannot be stressed too much! Sloppy notes mean sloppy field work to anyone using them. Pencils should be 4-H or harder. The note keeper should be a person that takes pride in doing a good job of printing. Entries in the notebook should be spaced adequately to make them easy to read and to avoid any possible misinterpretation.

Be Concise. Perhaps this can only be done through practice. It is better for the beginner note keeper to get all the information in the notes rather than worry about how easy they are to interpret. As experience is gained, however, efforts should be made to keep the notes as brief as possible and to keep them arranged in a logical sequence, still getting all the information desired.

CENTERLINE REFERENCE POINTS (RP)

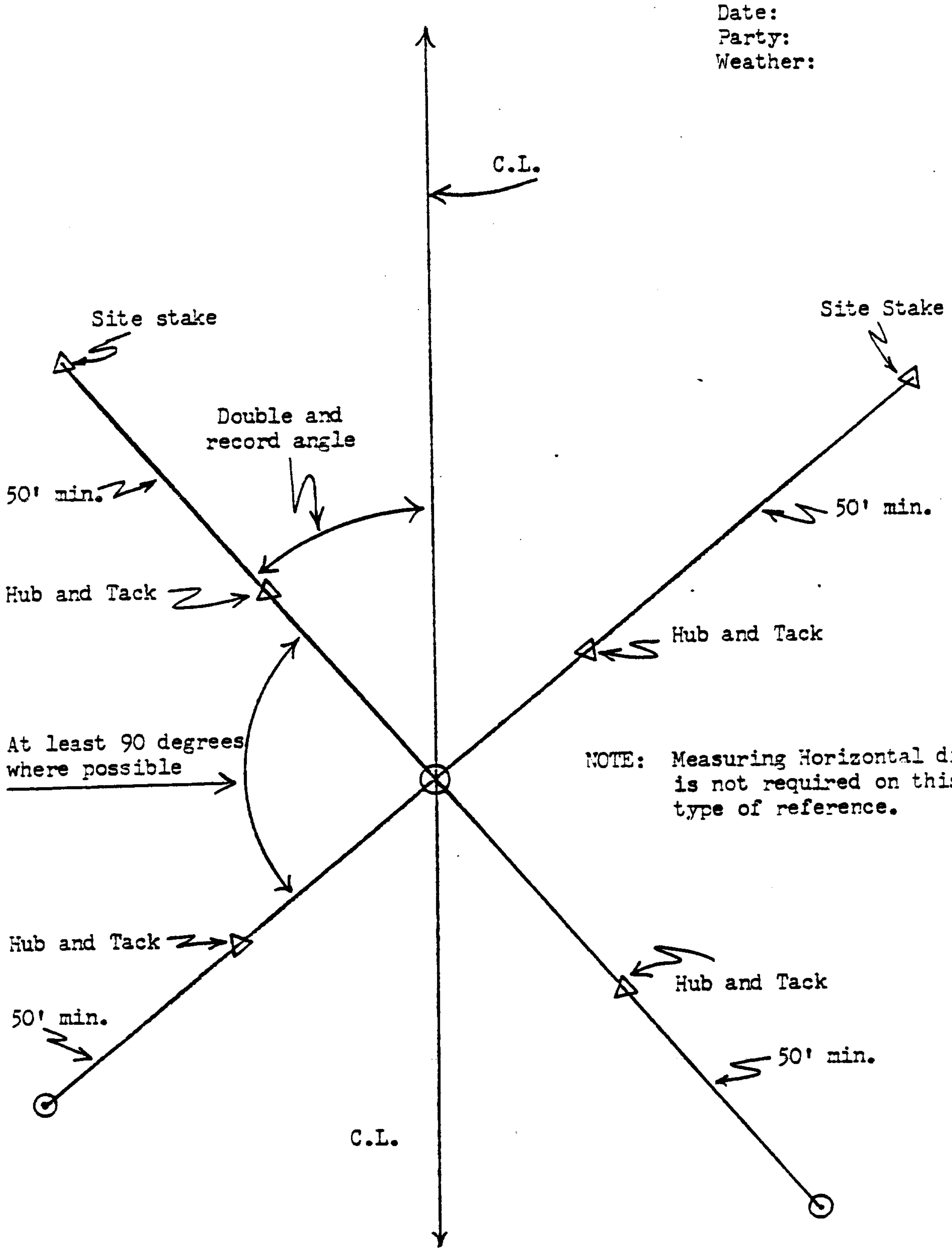
Good centerline reference points are essential for all future control of a project. With the emphasis now on contractor staking, referencing of control points is a very important part of centerline staking. Poorly set reference points can be grounds for a contract claim against the Government.

After the centerline has been run and closed, control points should be referenced. Reference points should be set on both tangents and curves so that the retracing of short sections of centerline can be done without extensive work. Tangents should be referenced at the beginning and end and at 1,000-foot intervals or as the terrain dictates. Curves should be referenced at 500-foot intervals. On steep grades and sharp curves, shorter segments should be referenced.

All reference notes shall be kept in the transit book with date set, party, and weather. A sketch of the reference shall be made, with a complete description of angle and distances, as required. All angles must be doubled and care used in measuring distances. All distances should be recorded and used as horizontal measurements. If slope distances are used, the conversion to horizontal measurements should be recorded. It is usually preferable to reference at 90 degrees to centerline where possible, but when steep terrain is encountered, cross references should be used. However, whatever method is used is at the discretion of the project engineer. Make sure reference stakes are outside the construction limits and clearly marked.

CROSS TIES AND/or STRADDLE TIES

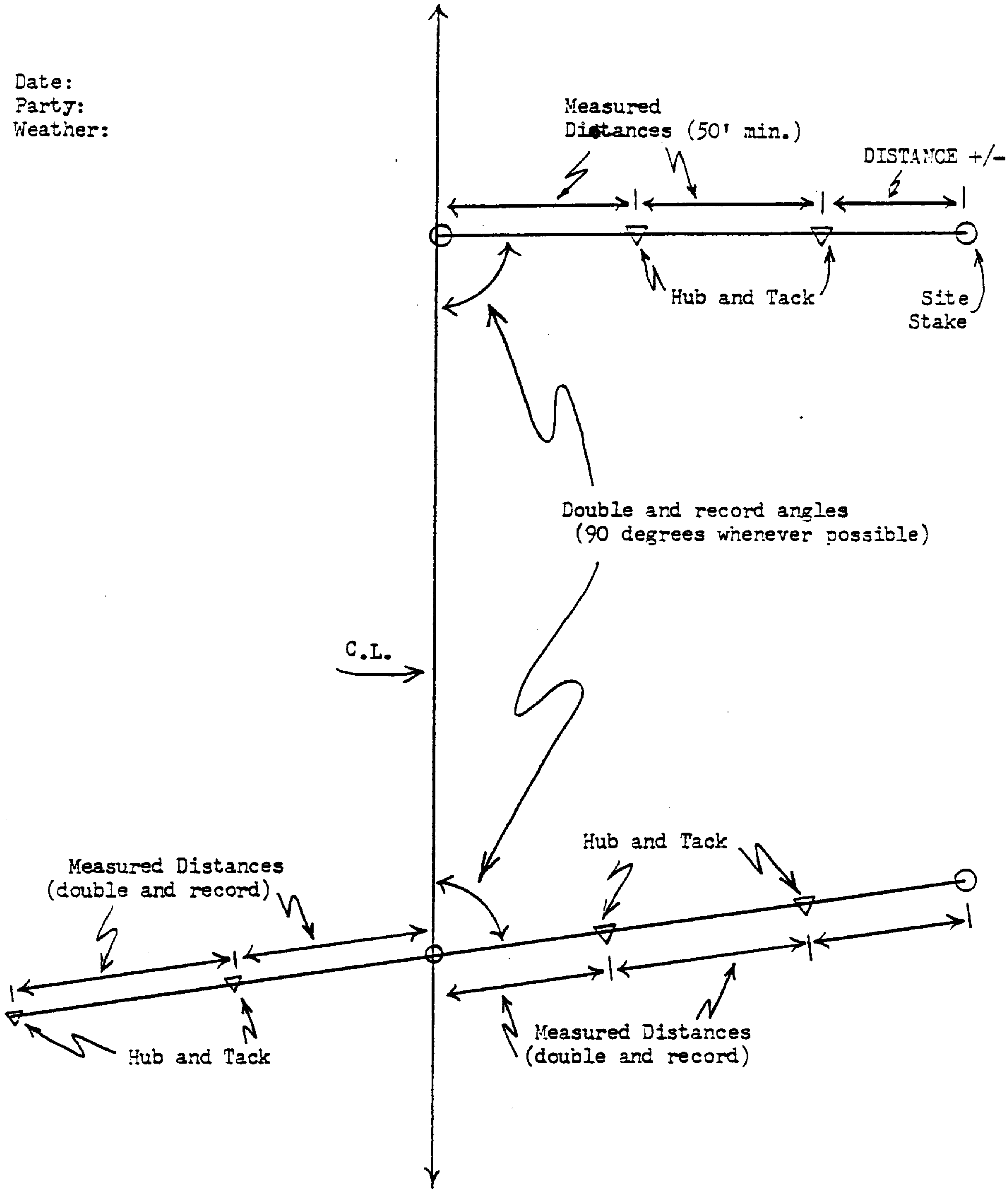
Date:
Party:
Weather:



NOTE: Measuring Horizontal dist. is not required on this type of reference.

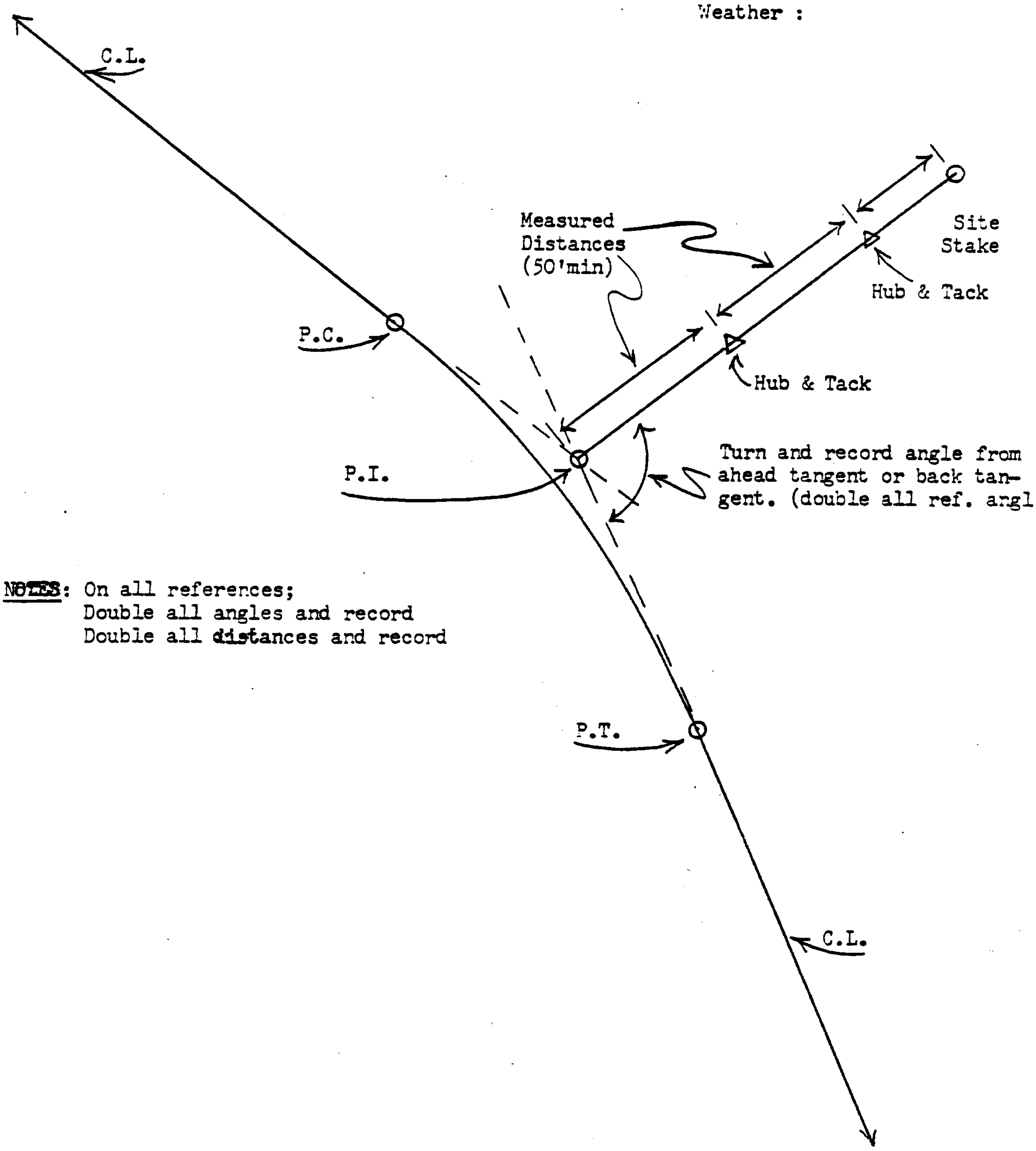
RIGHT ANGLE TIES

Date:
Party:
Weather:



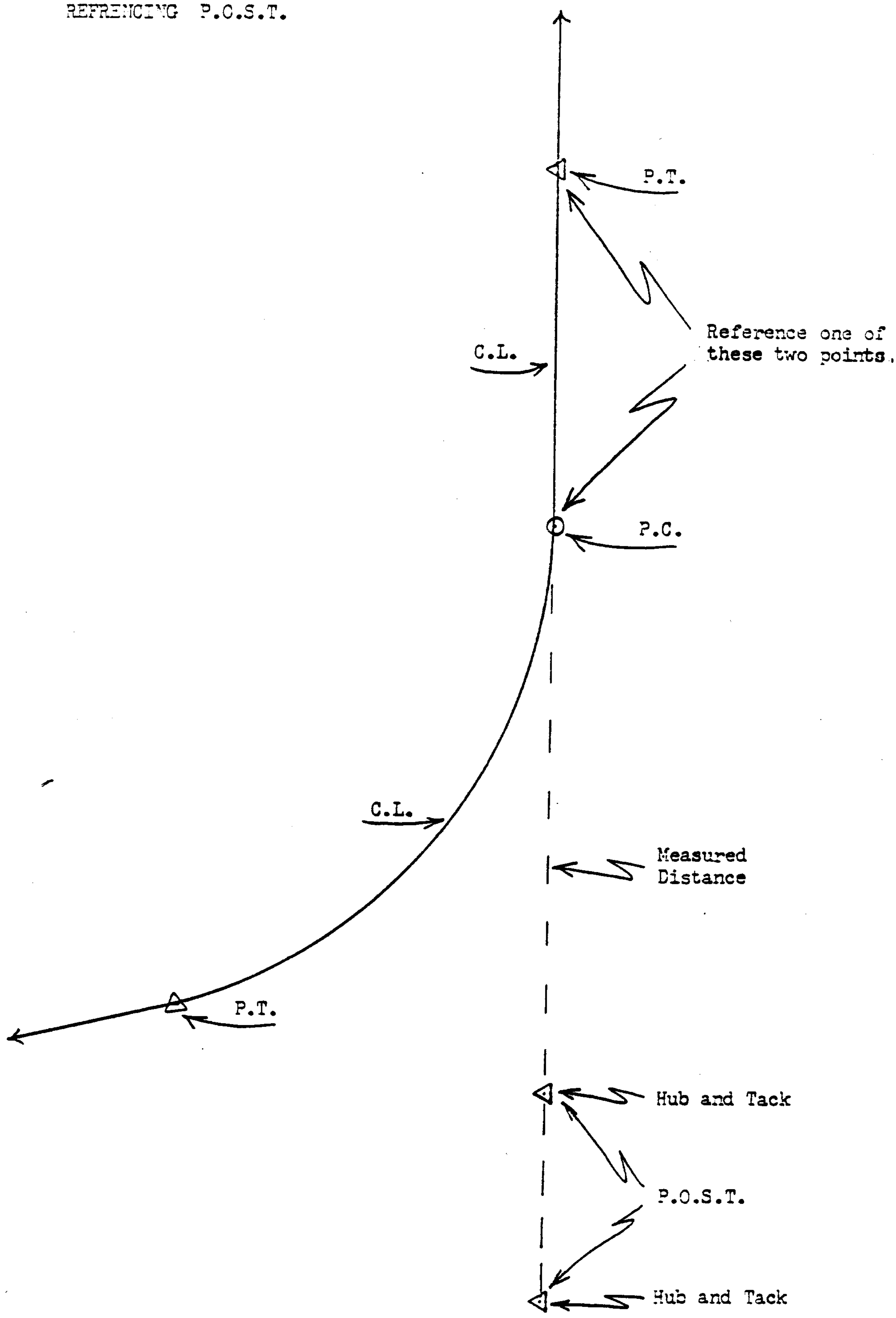
REFERENCING POINTS OF INTERSECTION (P.I.)

Date:
Party:
Weather :



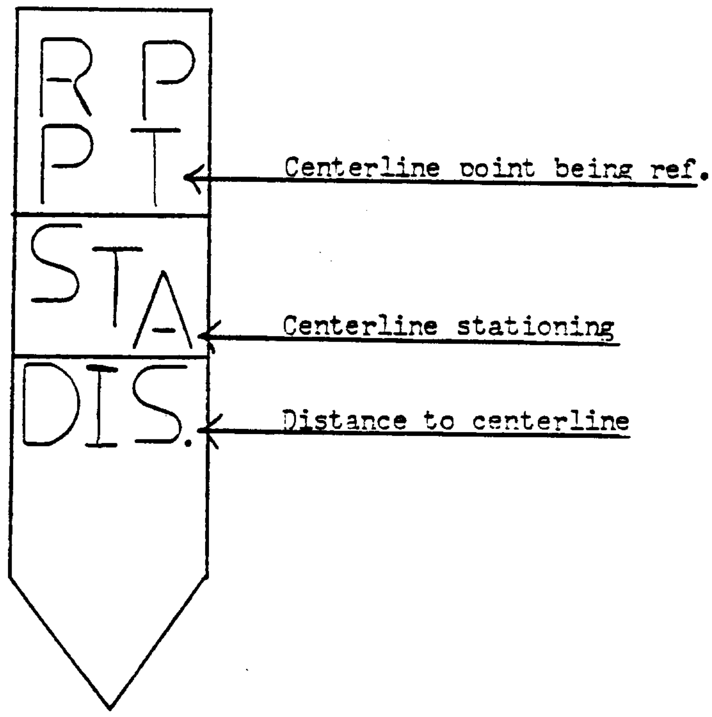
NOTES: On all references;
Double all angles and record
Double all distances and record

REFRENCING P.O.S.T.

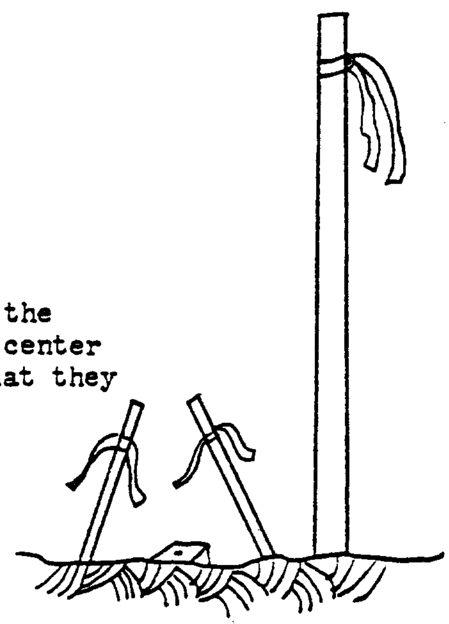


CENTER LINE REFRENCING
STAKE WRITE UP

Make sure informations is clearly written on front and back of stakes



Pound stakes securly in the ground perpendicular to center line and in such a way that they will not have to be disturbed to read them or to accupy the point.



BENCH MARKS

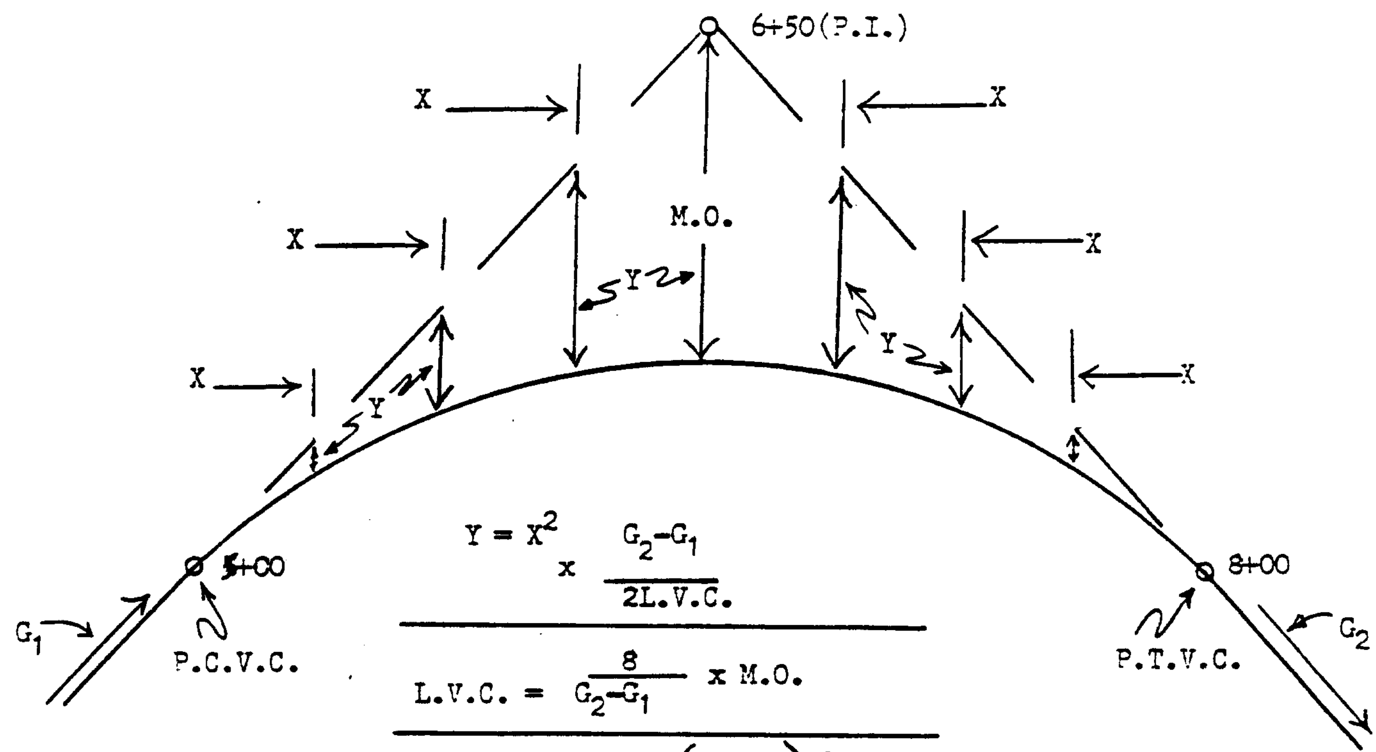
Bench marks shall be set using closed loops at intervals established by the project engineer (usually 500 to 800 feet). Bench levels shall have closure equal to accuracy required for red and blue tops (usually .03 of a foot) and shall be set using a standard "Philadelphia Rod." Use of 25-foot fiber glass topo rods will not be permitted. Level notes are to be kept in the standard format with dates and names of the party noted. A summary of bench mark locations with elevations shall be noted in the first pages of the level book. All "bad" level circuits should be voided (crossed through) and a reference made to "good" circuits.

Bench marks should be placed where they will be easily accessible from the finished (completed) roadway. Do not place them within the clearing limits, above high cuts, or adjacent to deep embankments. Make sure all bench marks are secure and in a location likely to not be disturbed. When placing a bench mark on a tree, be sure that the spike is long enough to be solid and placed so that the rod can be held plumb when on the point. When placing a bench mark in the ground, use a rebar or steel post that is long enough to be solid. Do not use wood hubs or guineas. Identify each bench mark with a unique name, number, or other identification and clearly establish their location in the field. Location of bench marks should be accurately noted in the level book.

Bench marks should be located near all major structures (bridges, bin walls, large culverts, etc.) on the project.

Centerline profile loops for slope staking should be closed to within 0.10 of a foot and use of 25-foot fiberglass topo rods may be permitted at the discretion of the project engineer. All level loops are to be checked. This includes the transfer of profile elevations from the level book to the slope stake book.

VERTICAL CURVE COMPUTATIONS



$$Y = X^2 \times \frac{G_2 - G_1}{2L.V.C.}$$

$$L.V.C. = \frac{8}{G_2 - G_1} \times M.O.$$

$$M.O. = \frac{G_2 - G_1}{2L.V.C.} \times \left(\frac{L.V.C.}{2}\right)^2$$

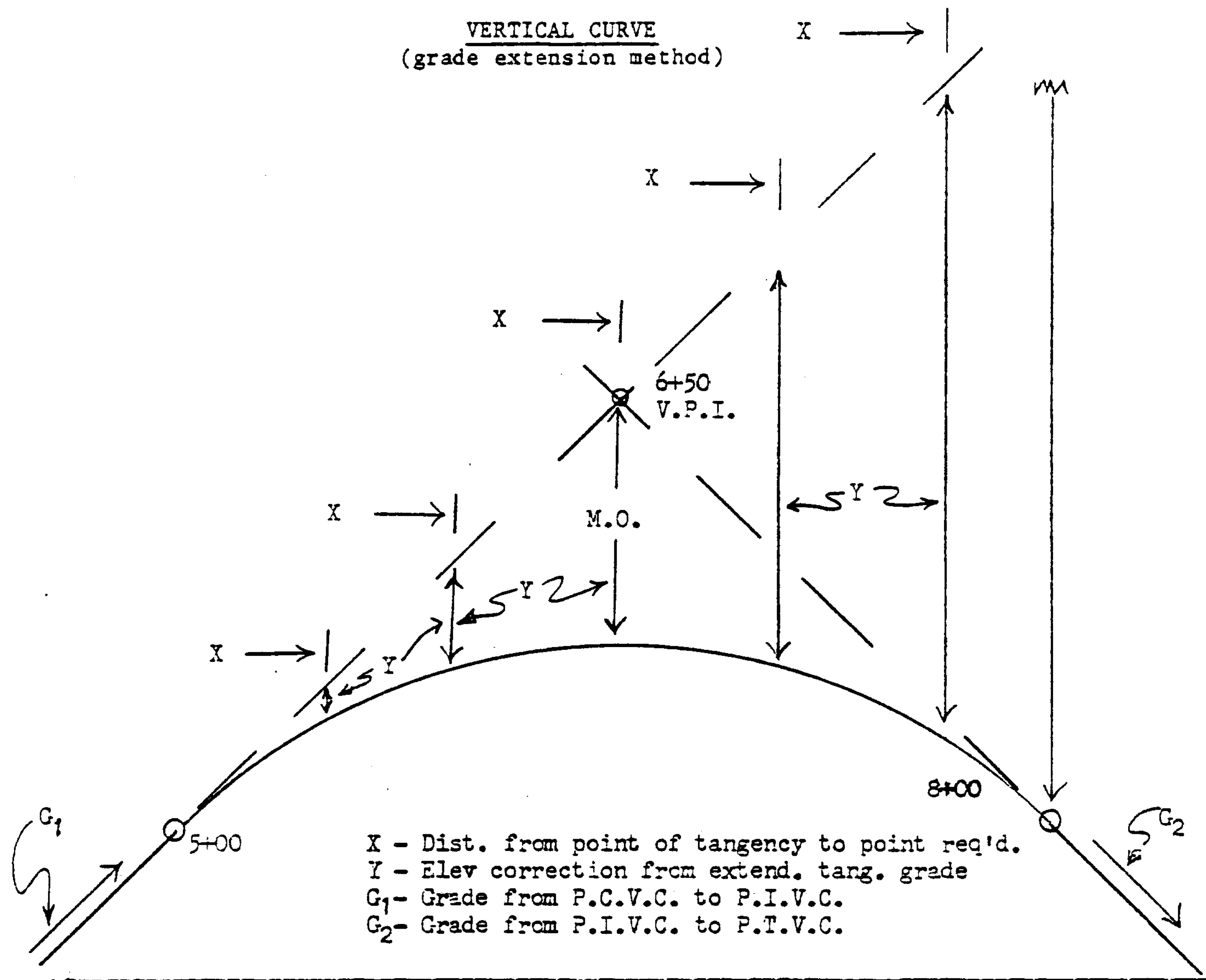
EXAMPLE: $G_1 = +4.000\%$, $G_2 = -2.000\%$ (use decimal equivalent of % for computing)

Station	Tang. El.	Dist. X	Dist. X^2	Y	Curve Grade
5+00	100.00	0.0	0.0	0	100.00
+50	102.00	50.0	2,500.0	-0.25	101.75
6+00	104.00	100.0	10,000.0	-1.00	103.00
+50	106.00	150.0	22,500.0	-2.25	103.75
7+00	105.00	100.0	10,000.0	-1.00	104.00
+50	104.00	50.0	2,500.0	-0.25	103.75
8+00	103.00	0.0	0.0	0	103.00

X - Distance from point of tangency to point required.
Y - Elevation correction from tangent grade.
G1 - Grade from P.C.V.C. to P.I.V.C.
G2 - Grade from P.I.V.C. to P.T.V.C.

NOTE: When subtracting grade two from grade one, use algebraic value.

VERTICAL CURVE
(grade extension method)



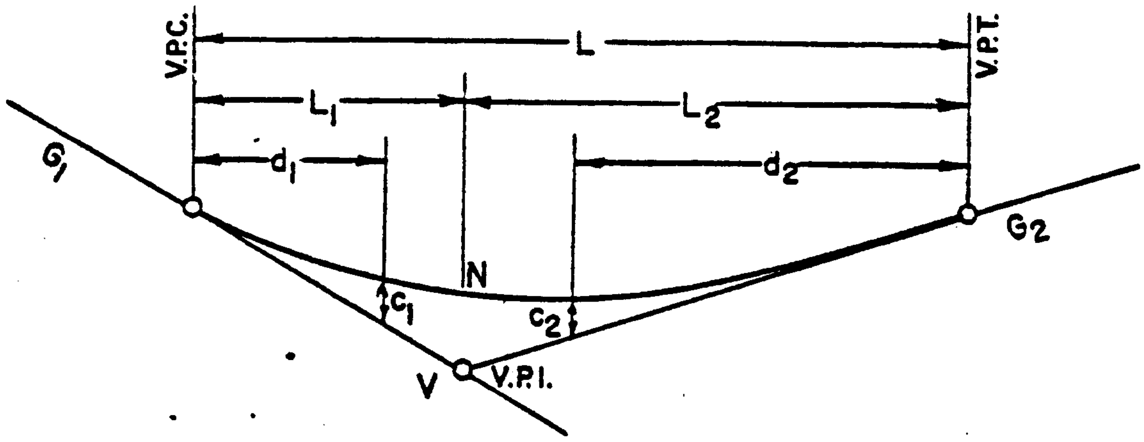
EXAMPLE: G_1 = +4.000%, G_2 = -2.000% (use decimal equivalent of % for computing)

Station	Tang. El.	Dist. X	Dist. X^2	Y	Curve Grade
5+00	100.00	0.0	0.0	0	100.00
+50	102.00	50.0	2,500.0	- .25	101.75
6+00	104.00	100.0	10,000.0	-1.00	103.00
+50	106.00	150.0	22,500.0	-2.25	103.75
7+00	108.00	200.0	40,000.0	-4.00	104.00
+50	110.00	250.0	62,500.0	-6.25	103.75
8+00	112.00	300.0	90,000.0	-9.00	103.00

NOTE: When subtracting grade two from grade one, use the algebraic value.

Algebraic value examples: +4 - (-2) = +6   -4 - (-2) = -2
+4 - (+2) = +2   -4 - (+2) = -6

UNSYMMETRICAL VERTICAL CURVES



$$NV = \frac{L_1 L_2 G}{2L}$$

$$c_1 = NV\left(\frac{d_1}{L_1}\right)^2 \quad \text{or} \quad c_2 = NV\left(\frac{d_2}{L_2}\right)^2$$

Where:
- G = Algebraic difference in grades
- L = Length of vertical curve in stations

Note: Use stations for all horizontal distances.

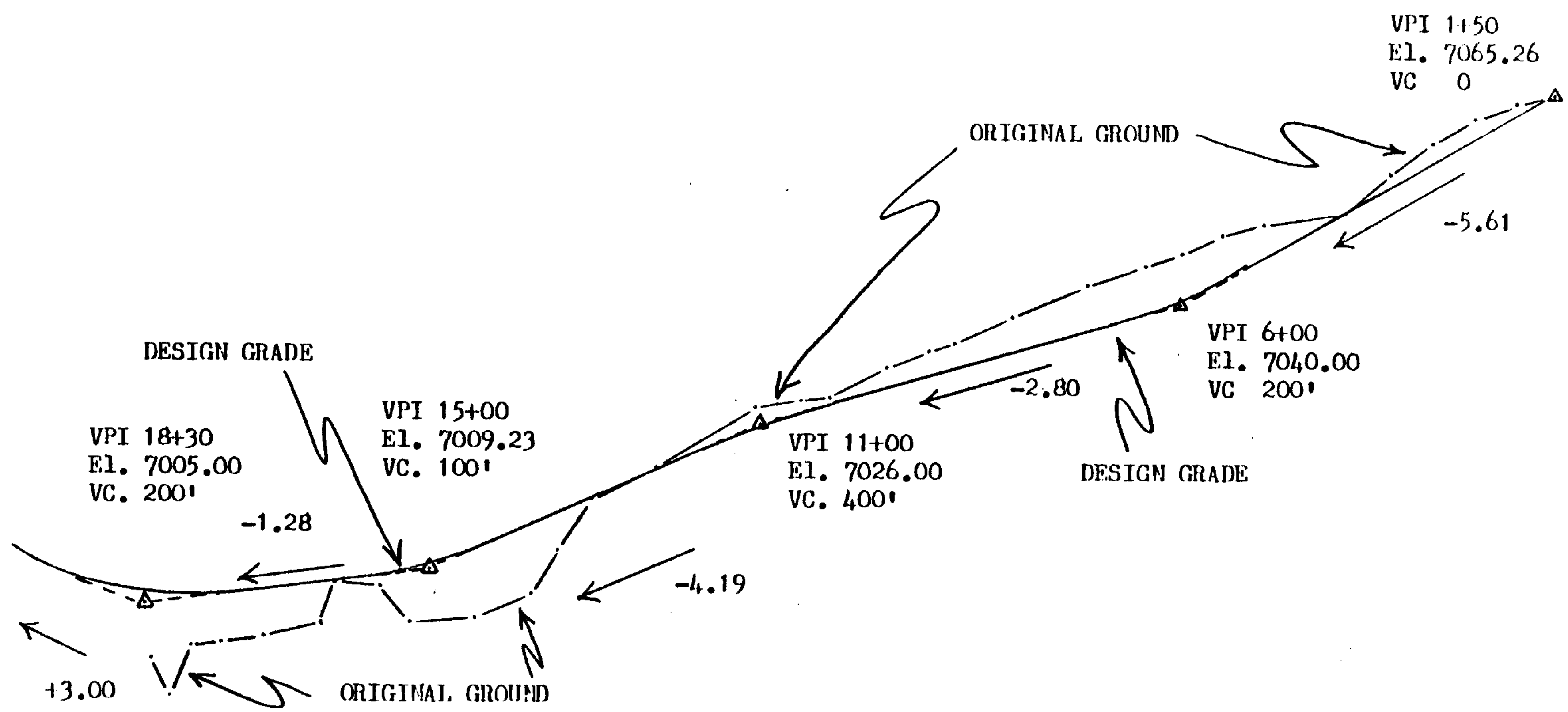
To determine the high point (crest curves) or the low point (sag curves) use:

$$X_1 = \frac{L L_1 G_1}{L_2 G} \quad \text{or} \quad X_2 = \frac{L L_2 G_2}{L_1 G}$$

Where:
- X_1 = Horizontal distance from the V.P.C. to the low (or high) point.
- X_2 = Horizontal distance from the V.P.T. to the low (or high) point.

Note: If X_1 is computed and exceeds the distance L_1, the formula is invalid and the point lies ahead of the V.P.I. X_2 should then be computed and used.
Use stations for all horizontal distances.

PLAN AND PROFILE EXAMPLE

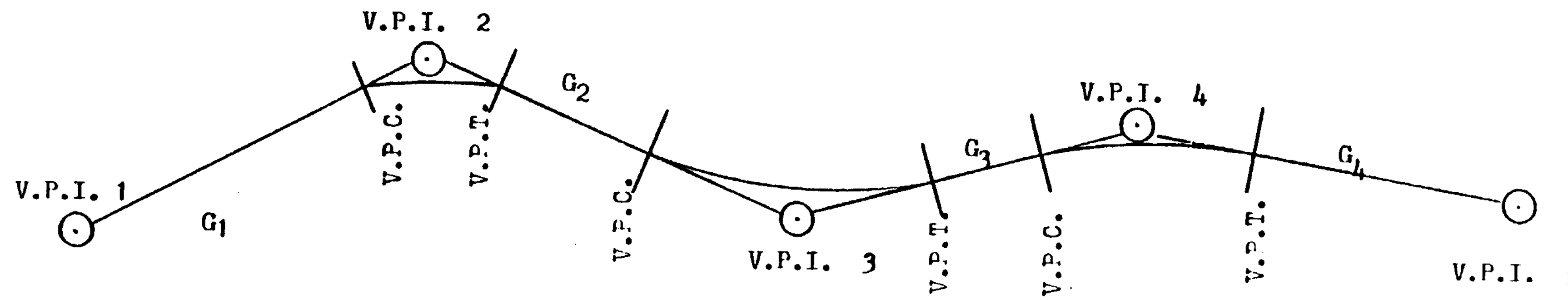


Grade	Point	Station	Tang. Elev.	Correction	Grade Elev.
		1+50	7065.26	-----	7065.26
-5.61		2+00	62.45	-----	62.45
		+50	59.65	-----	59.65
		3+00	56.84	-----	56.84
		+50	54.03	-----	54.03
		4+00	51.23	-----	51.23
		+50	48.42	-----	48.42
	VPC	5+00	45.61	-----	45.61
		+50	42.81	+ .17	42.98
	VPI	6+00	7040.00	+ .71	40.71
-2.80		+50	38.60	+ .17	38.77
	VPT	7+00	37.20	-----	37.20
		+50	35.80	-----	35.80
		8+00	34.40	-----	34.40
		+50	33.00	-----	33.00
	VPC	9+00	31.60	-----	31.60
		+50	30.20	- .04	30.16
		10+00	28.80	- .17	28.63

Grade	Point	Station	Tang. Elev.	Correction	Grade Elev.
		+50	27.40	- .39	27.01
	VPI	11+00	26.00	- .70	25.30
-4.19		+50	23.91	- .39	23.52
		12+00	7021.81	- .17	7021.64
		+50	19.72	- .04	19.68
	VPT	13+00	17.62	-----	17.62
		+50	15.52	-----	15.52
		14+00	13.42	-----	13.42
	VPC	+50	11.33	-----	11.33
	VPI	15+00	9.23	+ .36	9.59
-1.28	VPT	+50	8.59	-----	8.59
		16+00	7.95	-----	7.95
		+50	7.31	-----	7.31
		17+00	6.67	-----	6.67
		+50	6.03	+ .04	6.07
		18+00	5.38	+ .53	5.91
	VPI	+30	7005.00	+1.07	7006.07
+3.00					

$$\% \text{ GRADE} = \frac{\text{Elev. Diff. VPI to VPI}}{\text{Dist. Diff. VPI to VPI}}$$

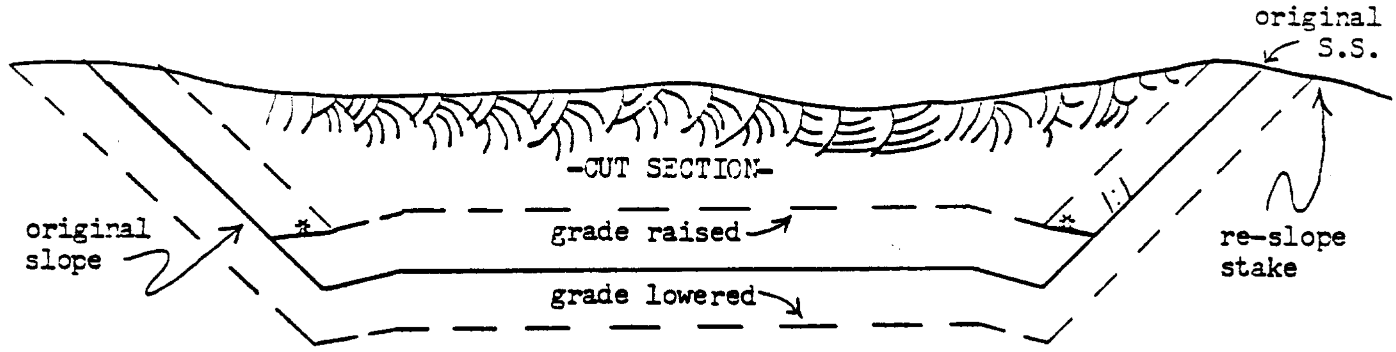
EFFECT OF GRADE ADJUSTMENTS ON VERTICAL ALIGNMENT



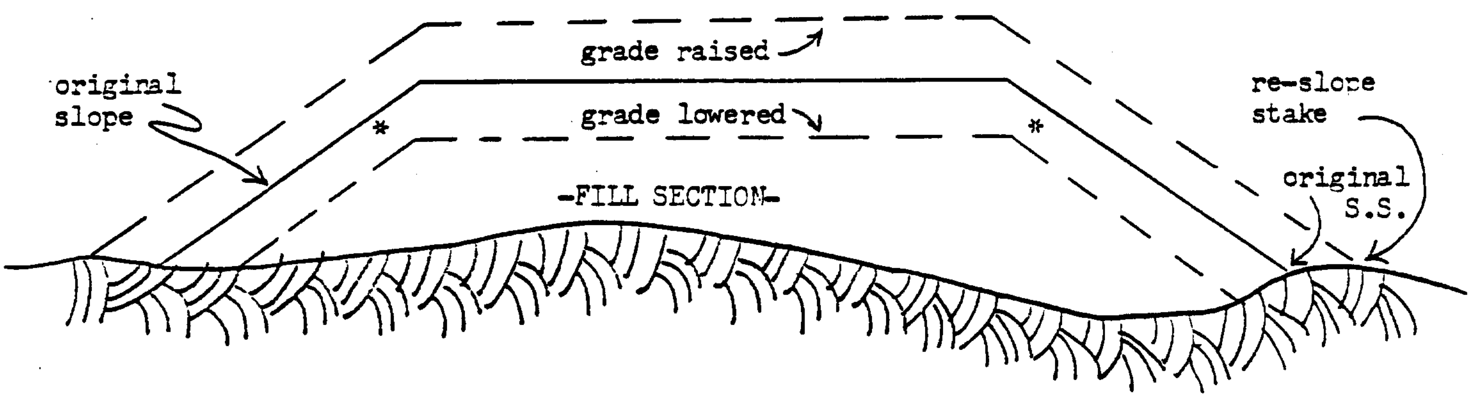
Note: A change of the M.O. (midordinate) will only change the L.V.C.
A change of the V.P.I. elevation will change grade ahead and back.

EXAMPLE: A change of elevation at V.P.I. 3, will change grades from the V.P.C. of V.P.I. 2 to the V.P.T. of V.P.I. 4.

EFFECT OF GRADE CHANGE ON ROADWAY PRISM



Note: If grade is lowered, section will have to be re-slope staked.
If grade is raised, section should also be re-slope staked; *however if excavation has already begun you will probably have to settle for extra wide section



SLOPE STAKING

1. Be aware of design template.

2. Use correct notekeeping procedures for the type of sectioning being used.

3. Establish and maintain uniformity throughout the project.

4. Double check "working" cut/fill calculations.

5. Make sure right angles are correct; especially on roadways with steep grades and sharp curvature.

6. When cross sectioning, note all edge of road shots (ER), edge of stream (ES), fence lines (fence), etc.

7. Get all break points--a point on a slope where its gradient changes. Do not attempt to average shots.

8. Horizontal chaining when slope staking is critical and special care should be exercised. Do not tie the tape to the top of a 25-foot rod.

9. A minimum of two members of the crew should be checking all mathematical calculations.

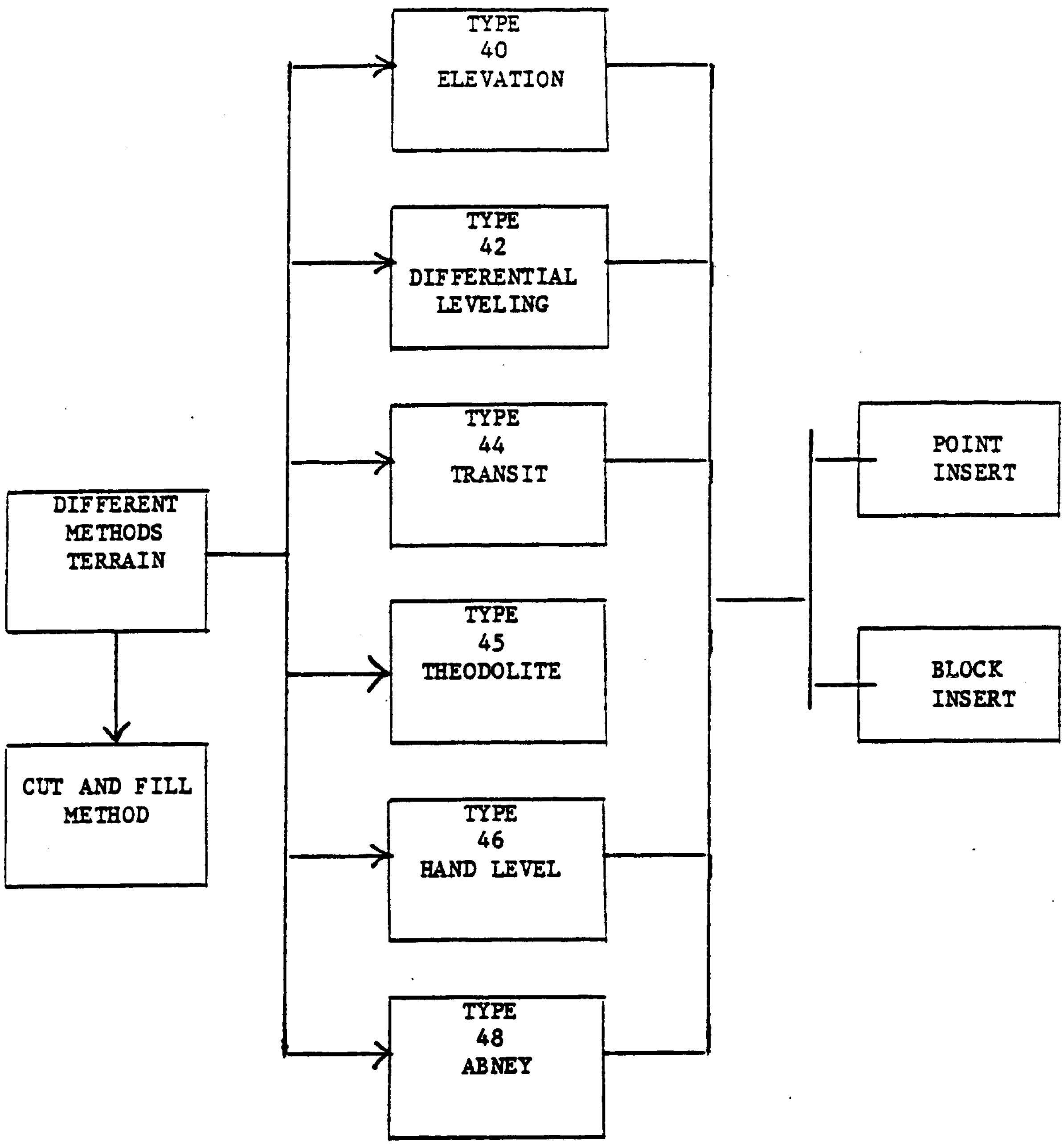
10. Make sure all notes are legible on all copies.

11. Slope and reference stakes should be solidly placed. Information written on the stakes must be legible. Use of stake pencils is generally preferred.

12. After stakes have been written, painting with spray lacquer will help preserve the markings from weathering.

13. Try to place stakes so that they can be easily located on the ground.

14. When slope staking, always cross check between stations. Running a profile level circuit over the reference hubs is one good checking procedure.



Type RT40 — Standard Topography - The rod shots are either plus or minus from centerline or elevations picked from contours. All distances are left and right of centerline.

Type RT42 — Differential Leveling - Method using a level with a fixed H.I. and a Philadelphia rod.

Type RT44 — Transit Cross Section - Method using a surveyors transit on the right angle line; engineer provides vertical angle and slope distance.

Type RT45 — Theodolite Cross Section - Basically the same as the transit method except the vertical angles are all positive and can be between 0 and 360 degrees.

Type RT46 — Hand Level Cross Section - Topography with rods and turns, can also be taken with Rhodes Arc.

Type RT 48 — Abney Level Cross Section - Percent of grade and slope distance with turns.

Type Cut and Fill -- The complete cross section is taken in cut and fill notation. The cut and fill shots are in reference to the template. The distances are horizontal from centerline.

Type Point Insert -- A catch point and one or two reference points in cut and fill notation. The points are in reference to the template hinge point. The ditances are horizontal from centerline. The points are inserted into previously taken terrain.

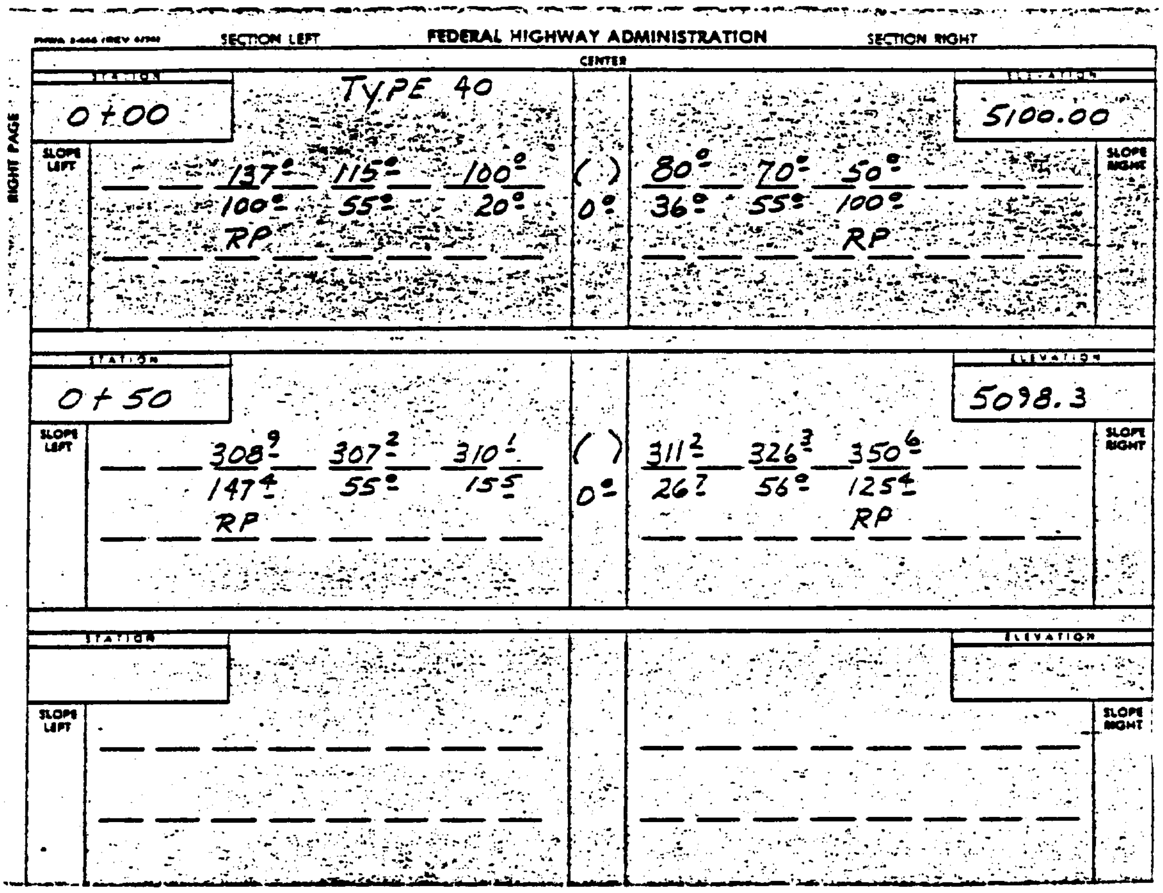
Type Block Insert -- A group of points and/or a catch point in cut and fill notation. The points are in reference to the template. The distances are horizontal from centerline. These points are to replace previously taken terrain.

NOTE: OFFSET CENTERLINE CAN ONLY BE SUBMITTED USING TYPE 42 OR TYPE 45 METHOD.

Type RT40--Direct Elevations or Rod Readings

This type of cross section is usually associated with direct elevations using a Lenker Rod and level as well as topography picked off of aerial photos or contour maps. However, plus and minus rods are also allowed. There can be no turns. If turns are required, use the RT46 format. Input may be submitted on special topography sheets, field books, or slope stake pages.

All rods and elevations are referenced to the reference elevation. All distances are referenced to centerline. If the centerline rod or elevation is blank, the RDS will interpolate a reading. All readings and distances can be to the nearest 1/100th of a foot if so desired.



FEDERAL HIGHWAY ADMINISTRATION

SECTION LEFT			CENTER	SECTION RIGHT		
STATION 0+00	TYPE 40					ELEVATION 5100.00
137.0	115.0	100.0	( )	80.0	70.0	50.0
100.0	55.0	20.0	0.0	36.0	55.0	100.0
RP						RP
STATION 0+50						ELEVATION 5098.3
308.9	307.2	310.1	( )	311.2	326.3	350.6
147.4	55.0	15.5	0.0	26.7	56.0	125.4
RP						RP

Type RT42—Differential Leveling, Rod Readings

This type of cross section is usually obtained with a level and rod, and the data consists of a rod reading and horizontal distance for each ground break. These measurements may be taken to the nearest 1/100th foot if so desired. Fundamentally, the computer establishes the height of instrument (H.I.) by locating the 0.0 reading. The rod shot at that point is added to the given reference elevation to establish the beginning H.I. Rod shots on the break points are subtracted from the H.I. until a turn is encountered. The turn is indicated by two rod readings at the same distance. The first reading establishes the elevation of the TP, and the second reading establishes the new H.I. The symbol used to indicate a turn is entered in the outermost of the two readings. The instrument does not have to occupy a point on the cross section line. Centerline is indicated by either the 0.0 distance or a C-L under a particular shot. If centerline is not at the 0.0 reading, the computer will reduce the cross section, locate centerline, and shift the reading to reflect the actual centerline as the 0.0 distance. If there is no reading on the centerline point, the reading will be interpolated.

<u>NOTE:</u> The use of the handlevel is allowed.

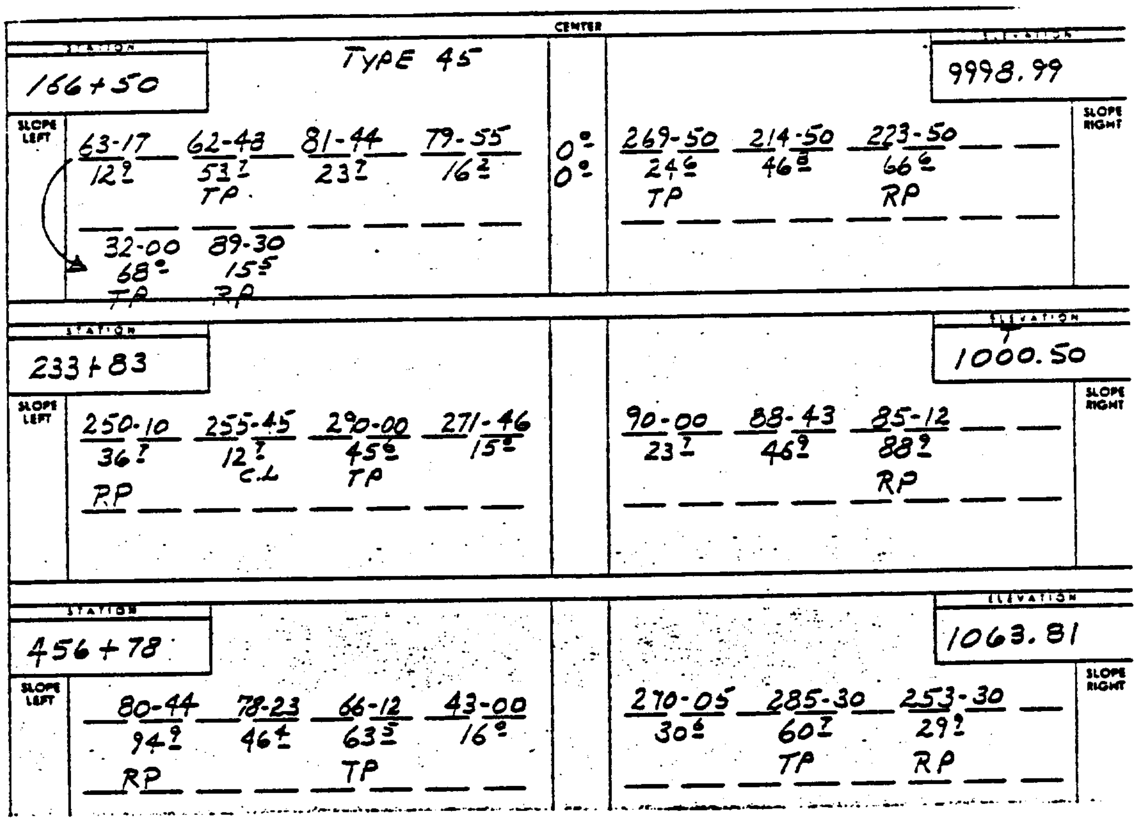
FHWA 5446 (REV 6/74) — SECTION LEFT — FEDERAL HIGHWAY ADMINISTRATION — SECTION RIGHT

STATION	SECTION LEFT	CENTER	SECTION RIGHT	ELEVATION
1+25	TYPE 42 $\frac{00^{2}}{50^{0}}$ $\frac{03^{1}}{20^{0}}$ $\frac{04^{2}}{10^{0}}$ RP (under $\frac{00^{2}}{50^{0}}$)	$\frac{5^{0}}{0^{0}}$	$\frac{05^{1}}{15^{0}}$ $\frac{07^{3}}{26^{0}}$ $\frac{09^{5}}{45^{0}}$ RP (under $\frac{09^{5}}{45^{0}}$)	1322.3
1+50	$\frac{09^{4}}{16^{8}}$ $\frac{11^{4}}{8^{2}}$ $\frac{12^{6}}{32^{3}}$ $\frac{01^{1}}{32^{5}}$ $\frac{02^{6}}{20^{6}}$ RP (under $\frac{09^{4}}{16^{8}}$), TP (under $\frac{12^{6}}{32^{3}}$)	$\frac{7^{5}}{0^{0}}$	$\frac{08^{3}}{15^{7}}$ $\frac{13^{0}}{36^{4}}$ $\frac{00^{2}}{36^{4}}$ $\frac{06^{7}}{33^{1}}$ $\frac{03^{9}}{41^{2}}$ TP (under $\frac{00^{2}}{36^{4}}$), RP (under $\frac{03^{9}}{41^{2}}$)	1330.5
1+75	$\frac{10^{3}}{51^{4}}$ $\frac{05^{4}}{29^{7}}$ $\frac{00^{6}}{47^{0}}$ $\frac{13^{1}}{47^{0}}$ $\frac{10^{5}}{32^{3}}$ $\frac{06^{0}}{16^{0}}$ RP (under $\frac{10^{3}}{51^{4}}$), TP (under $\frac{00^{6}}{47^{0}}$)	$\frac{3^{4}}{0^{0}}$	$\frac{02^{1}}{12^{0}}$ $\frac{00^{6}}{22^{0}}$ $\frac{12^{8}}{22^{0}}$ $\frac{10^{3}}{16^{0}}$ $\frac{08^{7}}{22^{0}}$ $\frac{05^{6}}{45^{0}}$ TP (under $\frac{12^{8}}{22^{0}}$), C.L. (under $\frac{10^{3}}{16^{0}}$), RP (under $\frac{05^{6}}{45^{0}}$)	1335.7

Type RT44 and RT45—Transit and Theodolite

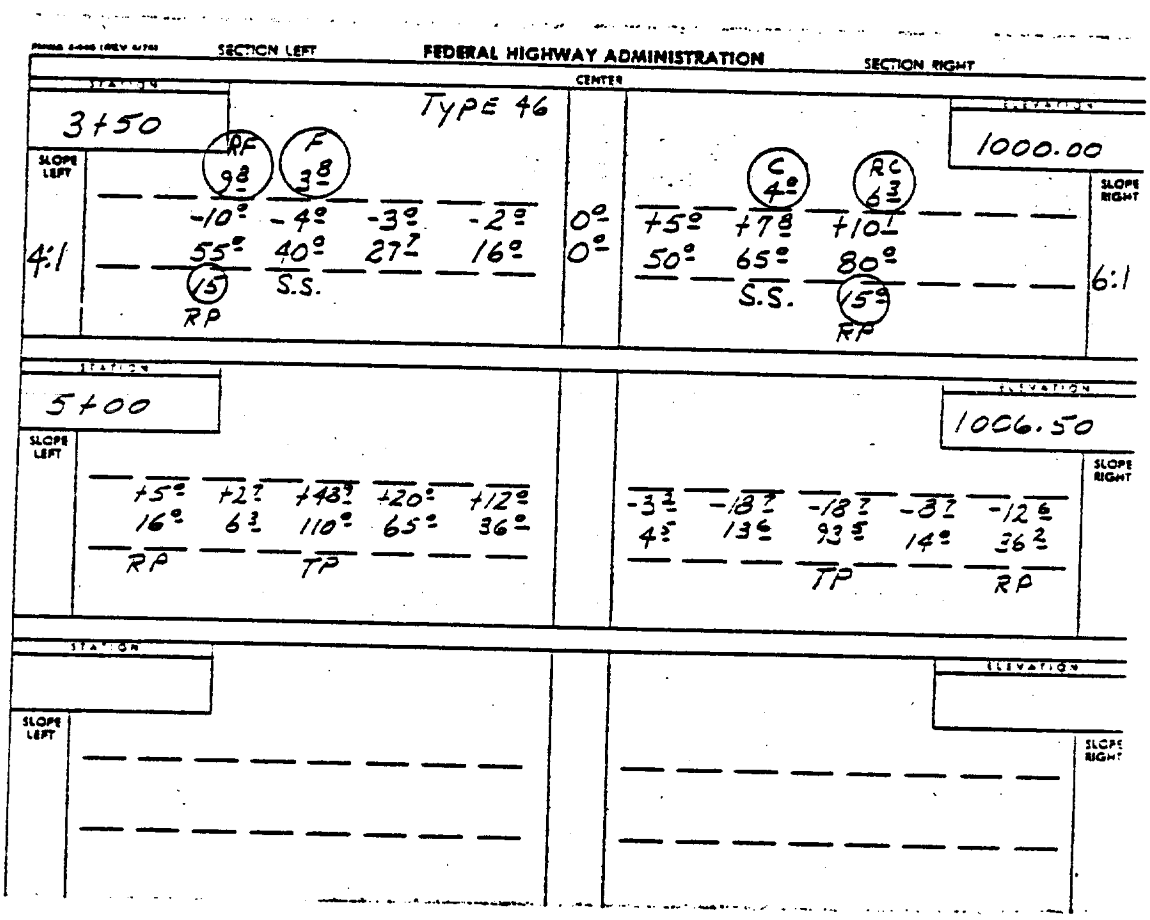
This type of cross section is obtained by use of a transit (RT44) or theodolite (RT45) and a H.I. stick. Data consists of a vertical angle reading and a slope distance for each ground break. Angular measurement is given to the nearest minute, and distance measurement may be to the nearest 1/100th of a foot if so desired. It is important that the instrument is always sighted on the H.I. point on the rod; otherwise, the angle measured will be in error. All distances recorded are slope distances with the only exception being the case where a reading is to be interpolated, such as a centerline that cannot be occupied. In this case, the distance recorded is horizontal from either the centerline or the last turn point. The instrument in the case of this type of cross section must occupy a point on the cross section line, but not necessarily the centerline point. If the staked centerline is not at this 0.0 reading, the computer will reduce the cross section, locate the centerline, and shift the readings to reflect the actual centerline as the 0.0 distance. If the reading on the centerline point is left blank, the computer will interpolate the elevation. If the reference elevation recorded in the notes refers to the elevation of the centerline point and not the 0.0 instrument, place a (T) enclosed in parentheses below the reference elevation.

NOTE: Sin. of vertical angle x slope distance = horizontal distance
Cos. of vertical angle x slope distance = vertical distance



Type RT46--Handlevel, Rhodes Arc

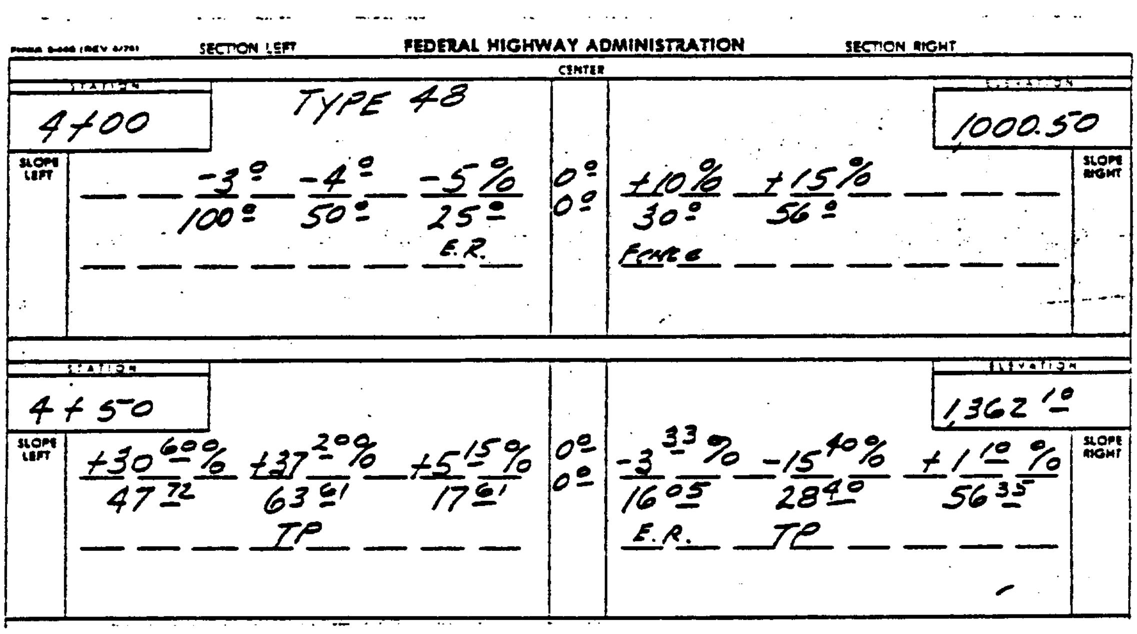
This type of cross section is usually obtained with a handlevel, H.I. stick, cloth tape, and a rod. Data consists of a plus or minus reading and horizontal distance from the instrument location to each ground break. Measurement can be entered to the nearest 100th of a foot if so desired. The plus or minus readings are added to or subtracted from the given reference elevation at each shot establishing the elevation of that point. Turn points are allowed. Offsetting or interpolation of readings cannot be performed using this type of data entry.



Type RT48--Percent of Slope

This type of cross section is usually obtained with an Abney and rod. Data consists of percent of slope and slope distance for each ground break. The measurements may be taken to the nearest 1/100th of a percent vertical and 1/100th of a foot slope distance.

The computer converts the plus and minus percentages and slope distances to plus or minus rod shots and horizontal distances from centerline.


SECTION LEFT — FEDERAL HIGHWAY ADMINISTRATION — SECTION RIGHT

CENTER

STATION 4+00 TYPE 48 ELEVATION 1000.50

SLOPE LEFT: -3.0 / 100.0, -4.0 / 50.0, -5% / 25.0, E.R. | CENTER: 0.0 / 0.0 | SLOPE RIGHT: +10% / 30.0, +15% / 56.0, Fence

STATION 4+50 ELEVATION 1362.10

SLOPE LEFT: +30.60% / 47.72, +37.20% / 63.61, +5.15% / 17.61, TP | CENTER: 0.0 / 0.0 | SLOPE RIGHT: -3.33% / 16.05, E.R., -15.40% / 28.40, TP, +1.10% / 56.35

REDUCING FROM %

(TAN^{-1}) % x (SIN) x (Distance) = Vertical Distance

(TAN^{-1}) % x (COS) x (Distance) = Horizontal Distance

EXAMPLE

(TAN^{-1}) 7.0% x (SIN) x (17.7) = 1.236

(TAN^{-1}) 7.0% x (COS) x (17.7) = 17.657

Cut and Fill Terrain

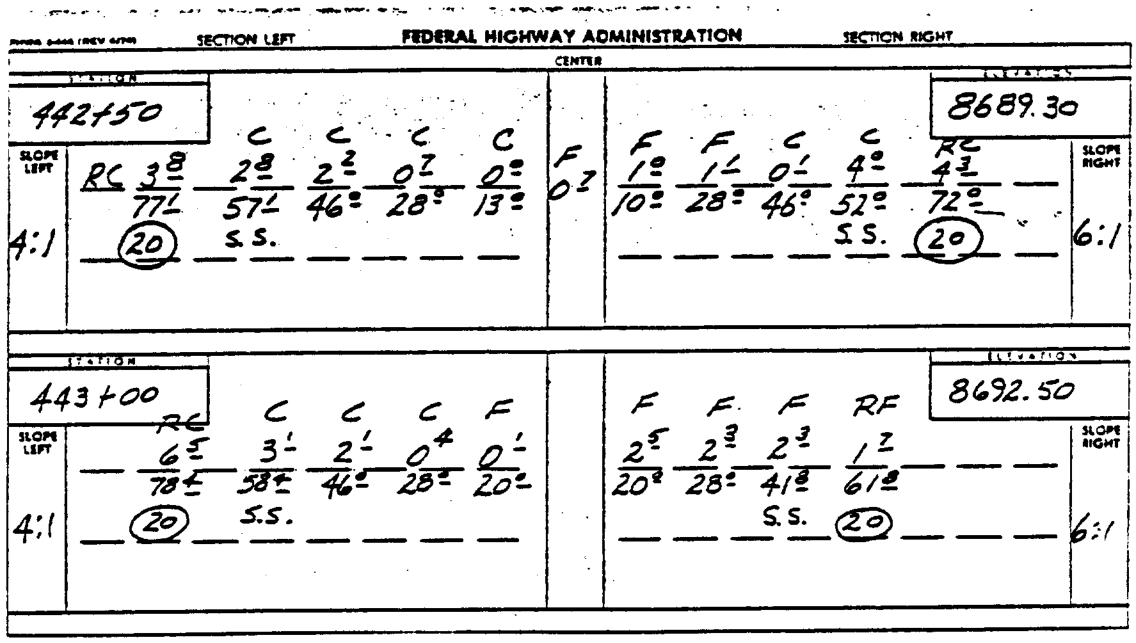
Cut and fill staking is one of the oldest methods of cross sectioning terrain. One of the fears engineers have about cut and fill staking is that if the grade is changed, new cross sections have to be taken for the grade change area. This is not a problem today. When cut and fill notes are submitted to ADP, the computer can reduce the shots to elevations and distances. The only new data needed in a grade change area are slope stakes (point inserts). It is mandatory that this type of note has left and right catch points. The computer also edits the catch points. Some of the checks are:

1. The slope stake is checked for multiplication errors.

2. A comparison of grades used in staking with those previously used to build the template at centerline.

3. A check is made on the ground elevation with the grade elevation and compares the result with the amount of cut or fill at centerline.

When cut and fill terrain is taken, the left page of the slope stake book has all the information needed to describe the templates and hinge points. If the engineer changes this information, such as grade, widening, or supers, he must code the change on the proper coding form. Do not make these changes on the left slope stake page as these changes will not be keypunched. A cut or fill shot does not have to be taken at template break point, but the shot has to be in reference to the last template break point. All shots from the hinge point and beyond are in reference to the hinge point.



Point and Block Inserts

When a project has been cross sectioned and the catch points need to be set (usually set from reference hub slope stake books), there are two ways that catch points can be submitted to ADP:

1. Point Insert

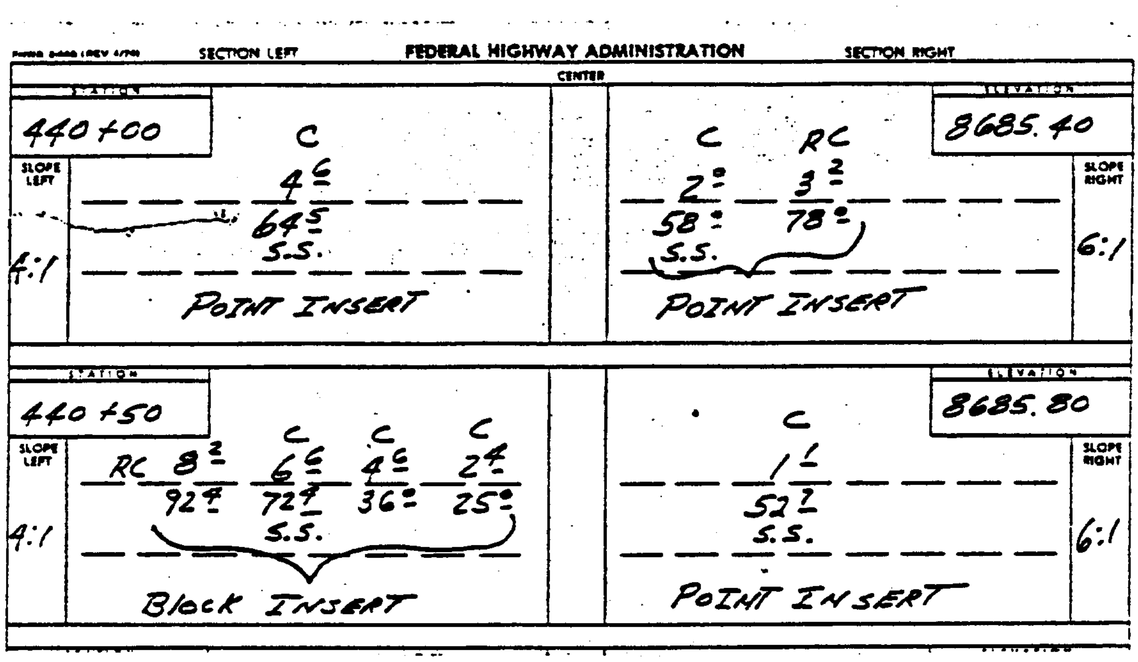
2. Block Insert

Both of these inserts have to be in cut and fill notation. The computer edits them both for:

1. Slope stake multiplication errors

2. A difference between the grade used to slope stake and the coded grade

The point and block inserts can be mixed for one station. The engineer should clearly mark block inserts that have less than five shots. All shots with four or less will be assumed point inserts unless otherwise stated.



Point Insert

The point insert allows the engineer to submit catch points with one or two reference stakes. These shots are converted from cut and fill notation into elevation and distances. They are then inserted into previously taken terrain (types 40-48). A maximum of four points can be inserted, one of which has to be the slope stake per side.

Block Insert

The block insert differs from the point insert in that it replaces one shot (or a group of shots) in the previously taken terrain. All old terrain is replaced between first and last shots on the block insert. Block inserts should only be used to replace a half of a section or less. If the complete section needs to be replaced, use cut and fill or one of the other methods available (types 40-48). A maximum of 44 points can be used to replace the previously taken terrain on each side of centerline.

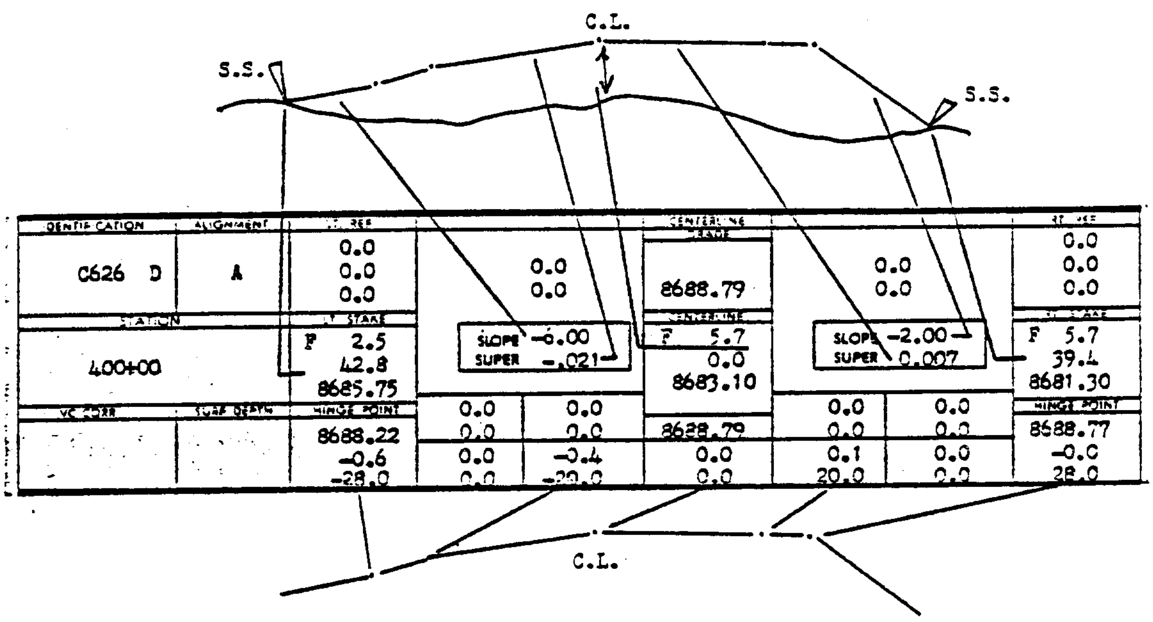
DO'S AND DONT'S FOR USING THE SLOPE STAKE BOOK

1. DON'T erase or mark over.
2. DON'T get excited when water removes carbon; it will come back when it dries out.
3. DON'T show more than one slope in Lt. and Rt. slope columns.
4. DO use a 4H pencil or harder.
5. DO PRESS FIRMLY WHEN PRINTING.
6. DO use backing behind white copy when making notations.
7. DO remove both white copies frequently, at the end of each day and after careful proofreading.
8. DO print legibly.
9. DO X-out errors. Circle or double bracket , (( )), numbers not to be keypunched.
10. ADP prefers originals to be sent in--IF WHITE CARBONS ARE SENT IN, MAKE SURE THEY ARE LEGIBLE.
11. DO show all catches with S.S. code. (One on each side.)
12. DO show catch slopes in slope column.
13. DO show all daylights with daylight code--DD or DS.

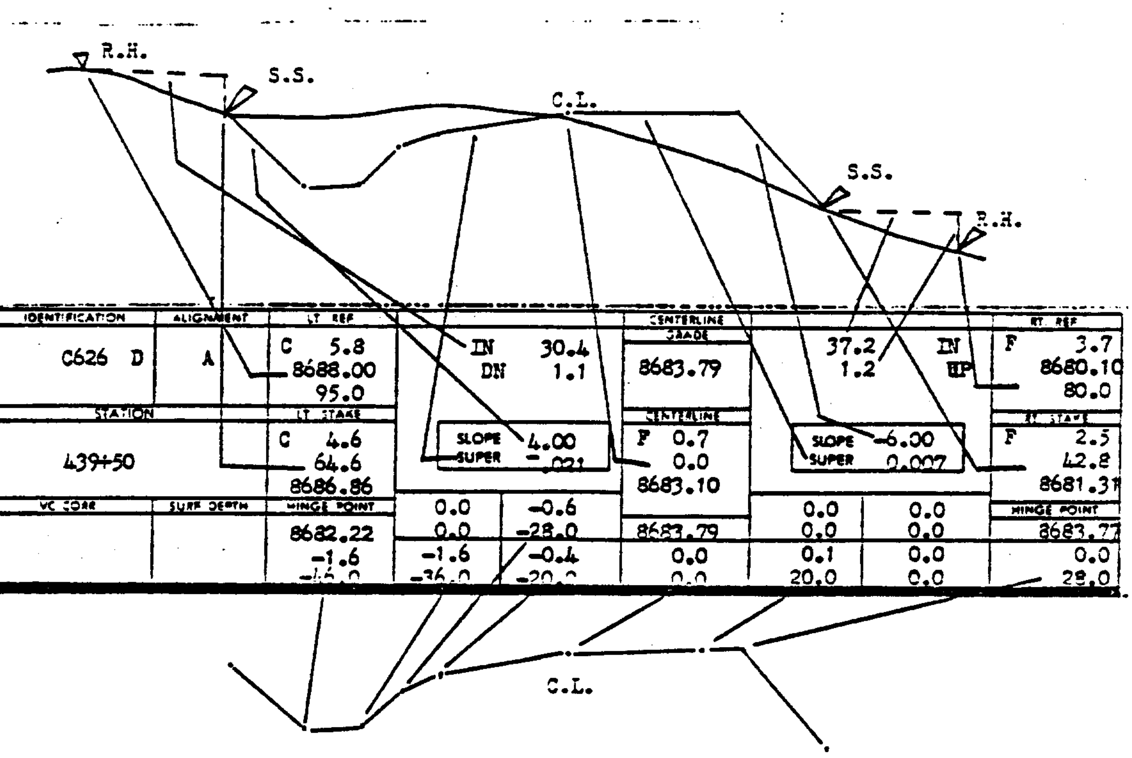
OTHER DO'S

1. Print TP below shot to indicate turning point, and distance starts over.
2. If reference hub stake-out is desired, show RH below Reference shot.
3. If Topog. is taken offset from centerline, show CL beneath true centerline.
4. If offset centerline method if used, and ELEV. of centerline is used, show "T" alongside elevation, (7463.32T).
4. If fixed slope is desired, show slope in slope column with a "C" or "F" to indicate whether slope is in Cut or Fill.

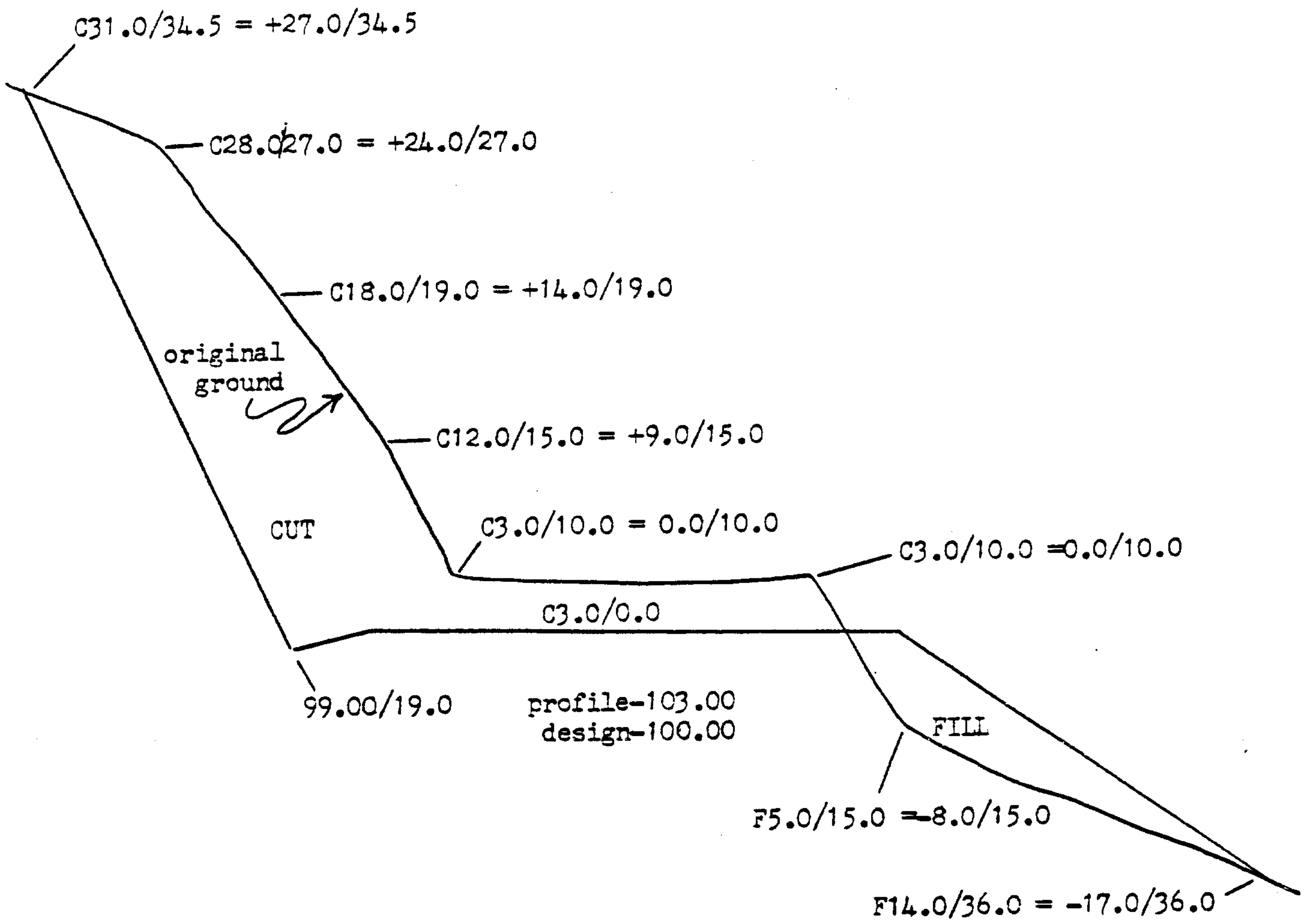
THE FOLLOWING EXAMPLE SHOWS THE TEMPLATE, ORIGINAL GROUND, AND HOW THE REGULAR SLOPE STAKE BOOK VALUES RELATE



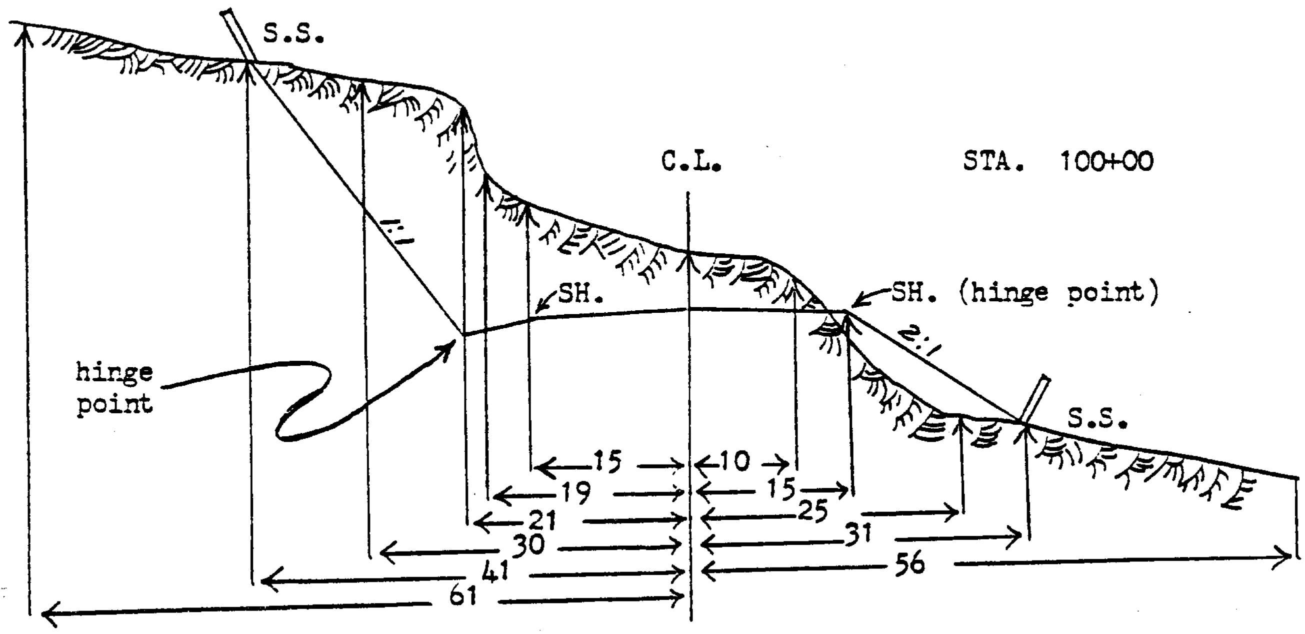
THE FOLLOWING EXAMPLE SHOWS THE TEMPLATE, ORIGINAL GROUND, AND HOW THE REFERENCE HUB SLOPE STAKE BOOK RELATES

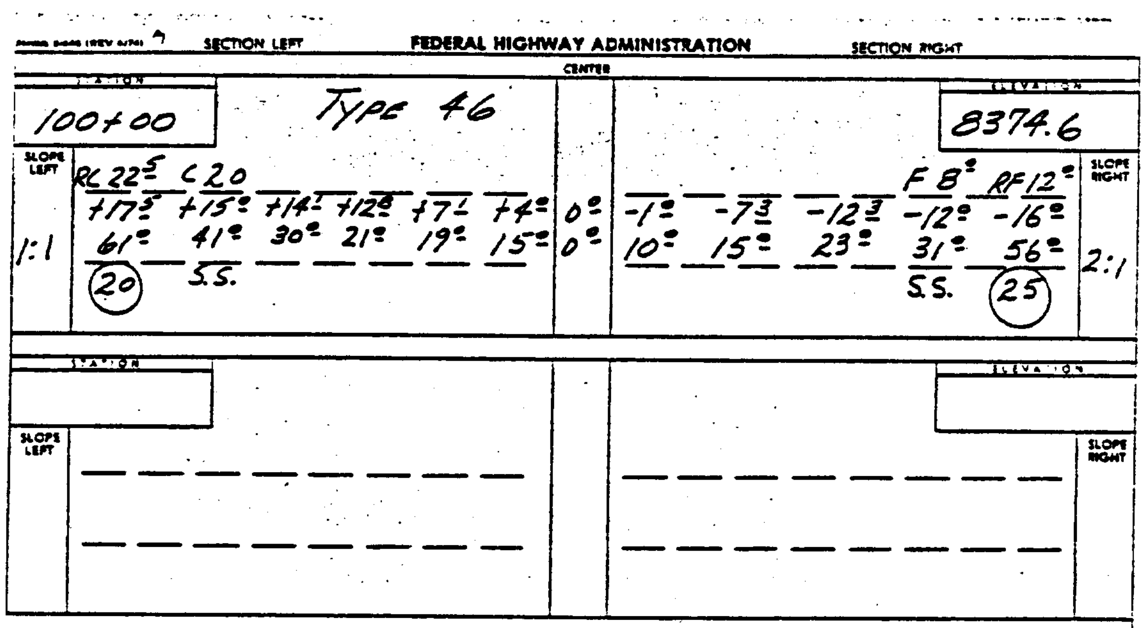


RELATIONSHIP BETWEEN CUT/FILL AND +/-

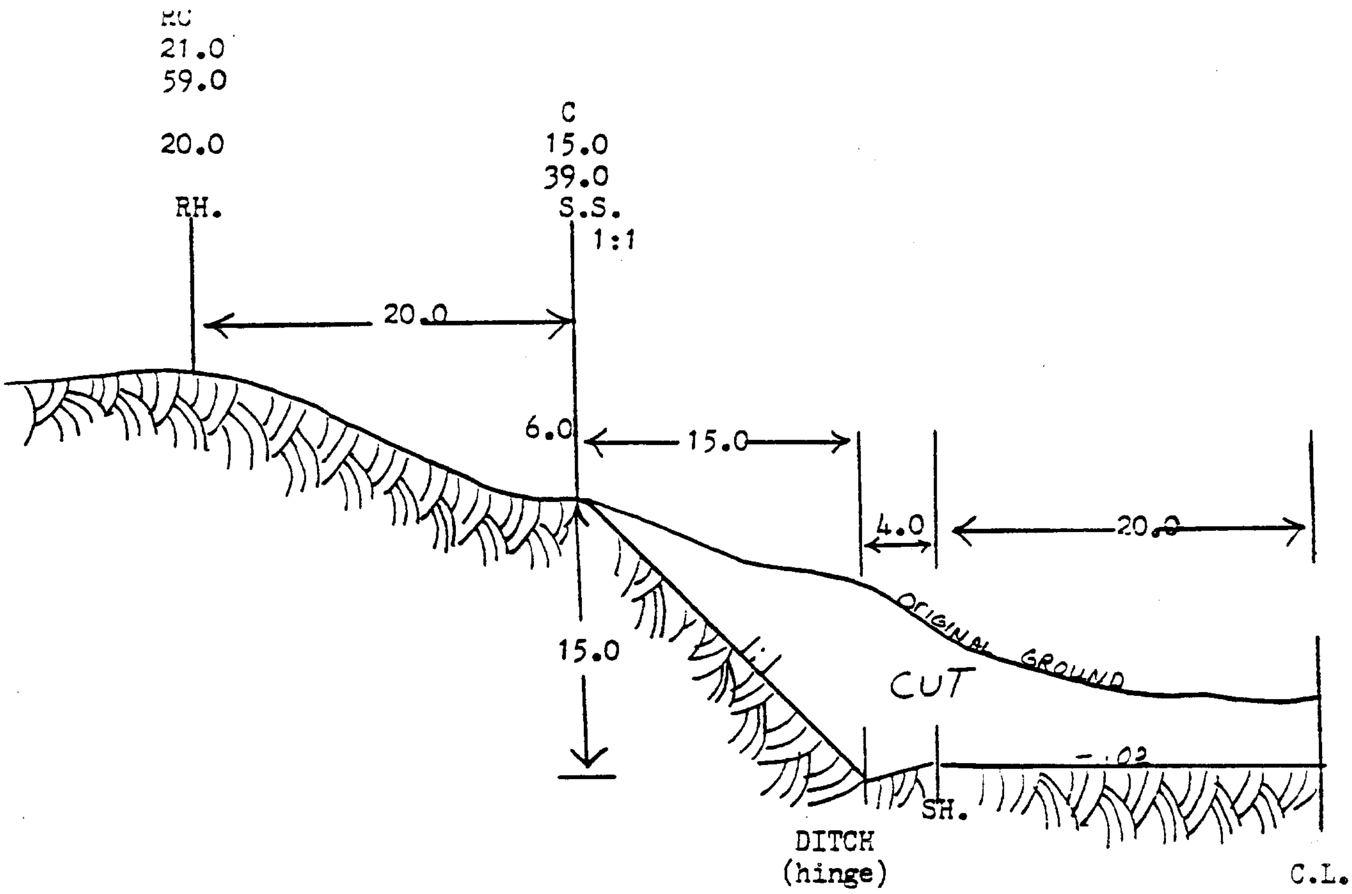


DEFINITION OF SLOPE STAKE HINGE POINTS

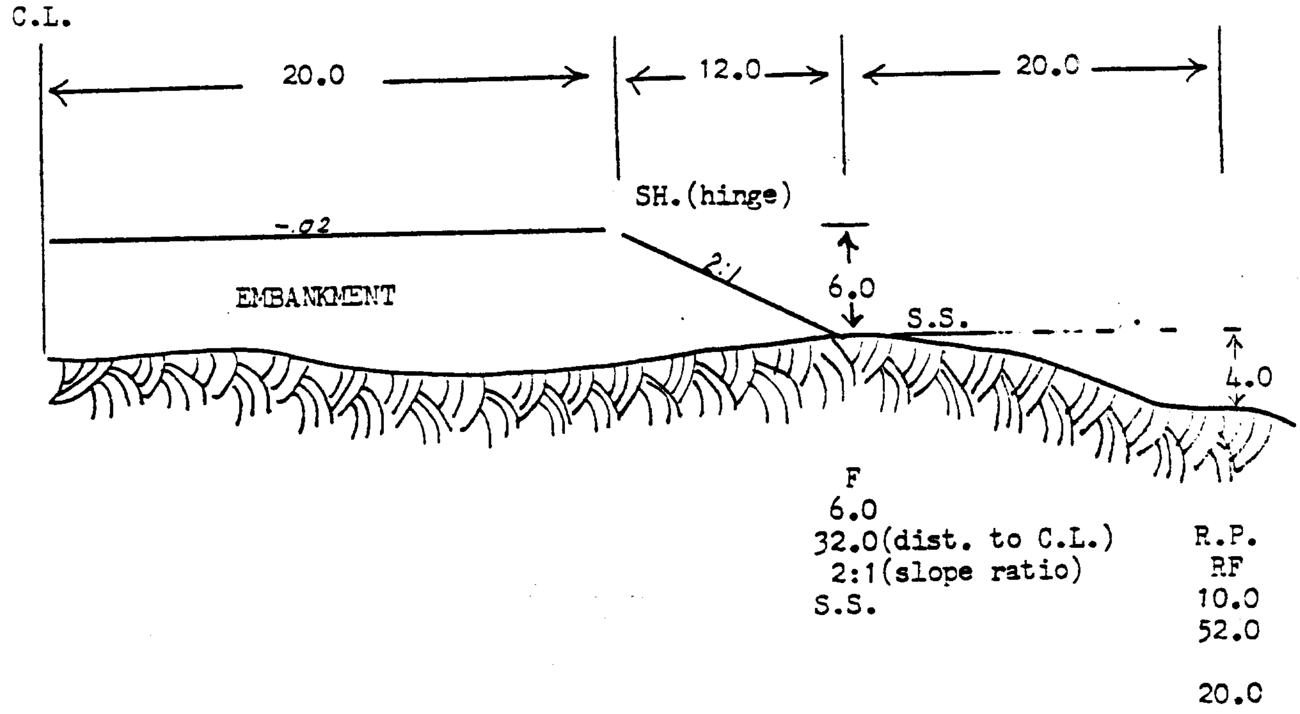




SLOPE STAKE RELATION (CUT)

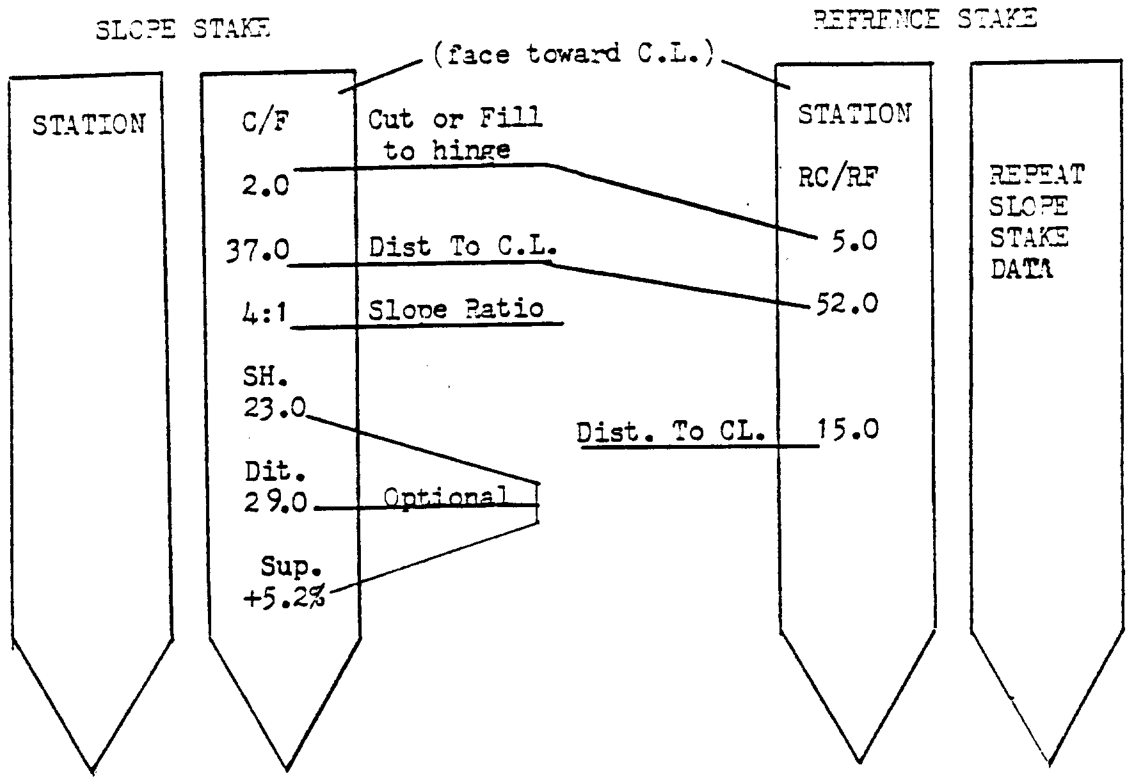


SLOPE STAKE RELATION (FILL)

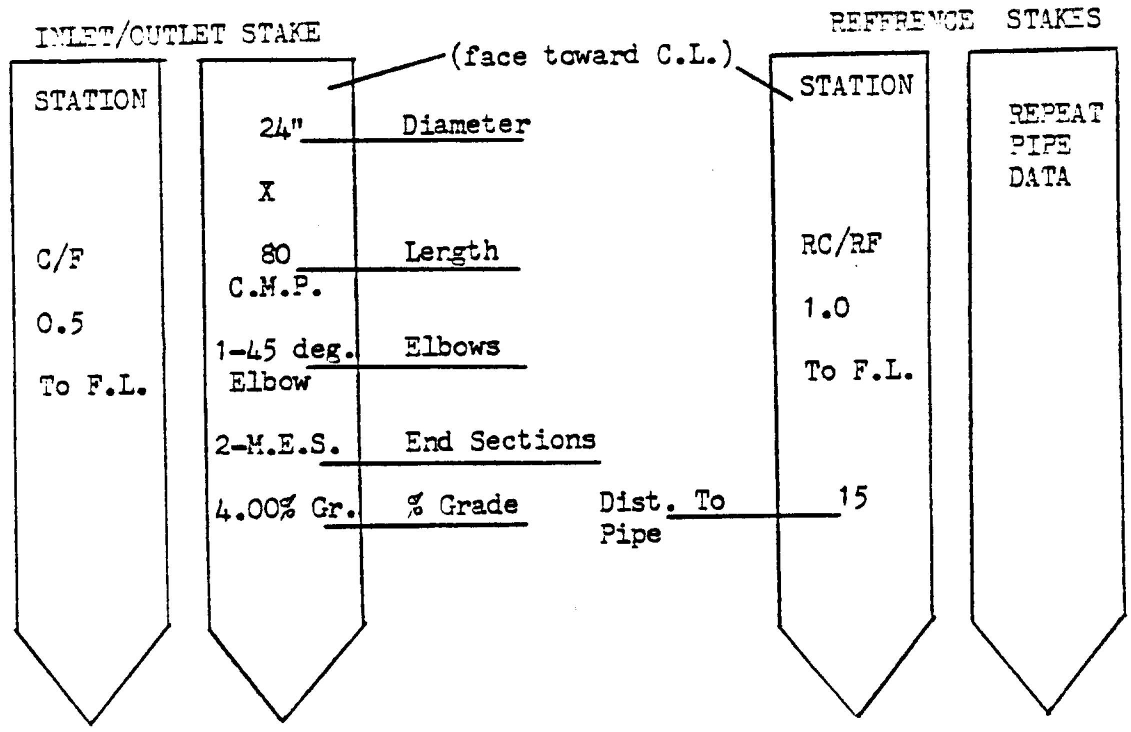


NOTE: All stakes should be pounded solidly in the ground. Do not pound stake so deep that it has to be pulled out to be read.

RC/C - RF/F METHOD FOR SLOPE STAKE WRITE UP



RC/C - RF/F METHOD FOR CULVERT STAKE WRITE UP



Culvert Staking

Field determination of culvert locations may not necessarily agree with locations shown on the plans. Minor adjustments may be required for proper drainage, particularly where natural water courses exist. Always be aware of field conditions, roadway template, and roadway grade when staking culverts.

When staking culverts in embankment areas, it is important to assure the outlet is long enough and positioned to direct the water away from the toe of the fill.

When staking culverts in cut areas, care should be taken to assure the culvert has adequate cover. Refer to standard details on the plans. Take care to avoid placing culverts too low which results in drop inlet and catch basin installations that will be traffic hazards.

Take special care in calculating inlet and outlet elevations and culvert grade. Have someone check all computations. If time allows, plot cross sections for culvert locations.

The information placed on all culvert and reference stakes should be legible, complete, and adequate to complete the installation.

CULVERT STAKING DATA
(example for culvert book set up)

STAKED		PLANNED
	STATION	
	DIAMETER	
	LENGTH	
	GAGE	
	SKEW	
	ELBOWS	
	END SECTIONS	
	ANCHOR ASSEMBLIES	
	INLET/OUTLET TREATMENT	
	STAKED BY	
	DATE	

INLET ELEVATION________ DROP__________

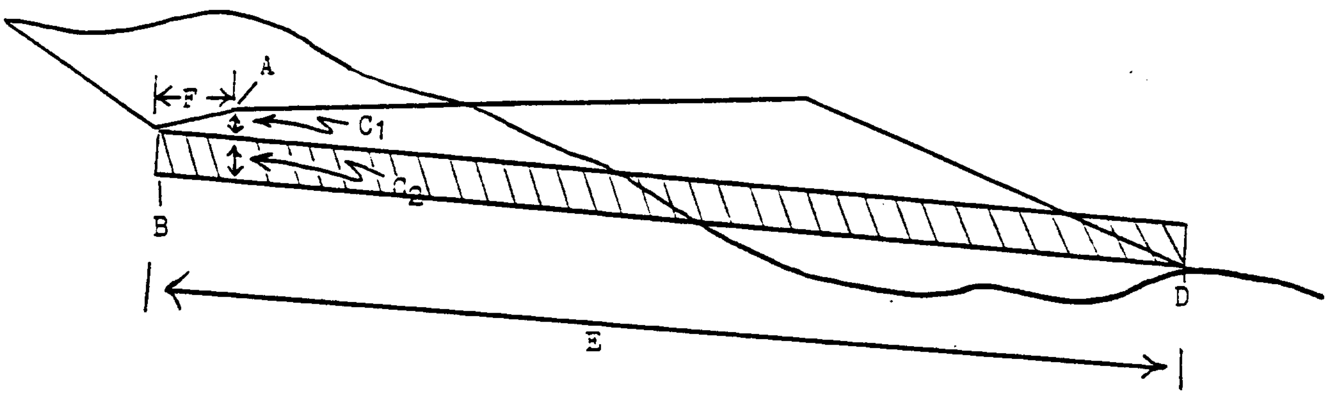
OUTLET ELEVATION____________________ % GRADE/RATIO___________

INLET, RC/RF__________

OUTLET, RC/RF__________

NOTE: Show inlet, outlet, and percent grade or percent ratio computations to be rechecked.

CULVERTS



Elevation B = A−C

% Grade = $\frac{B-D}{H.D.}$ (horizontal distance)

% Ratio = $\frac{B-D}{E}$ (slope distance)

A − Shoulder El. (given)
B − Inlet El.
C_1 − Minimum Cover (see Plans)
C_2 − Diameter of Culvert (given)
C − $C_1 + C_2$
D − Outlet El.
E − Length of Culvert (slope distance)
*F − Distance from shoulder to inlet

*Elevation adjustment should be made to elev. B using dist. F in culvert with steep grades

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EXAMPLE:

A = 7030.24 (given)

B = FIND

C_1 = 18" (1.5')

C_2 = 24" (2.0')

C = 42" (3.5')

D = 7021.37 (field computed)

E = 80 (field measured)

G = FIND

R = FIND

B = 7030.24 (A) − 3.5 (C) = 7026.74 (inlet el.)

$$R = \frac{\underset{(B)}{7026.74} - \underset{(D)}{7021.37}}{\underset{(E)}{80}} = 6.71\%\ \text{Ratio}$$

G = to compute % grade, need to convert E to horiz. dist.

$$\text{Horiz. Dist.} = \sqrt{(\underset{(E)}{80} + \underset{(B-D)}{5.37})(\underset{(E)}{80} - \underset{(B-D)}{5.37})}$$

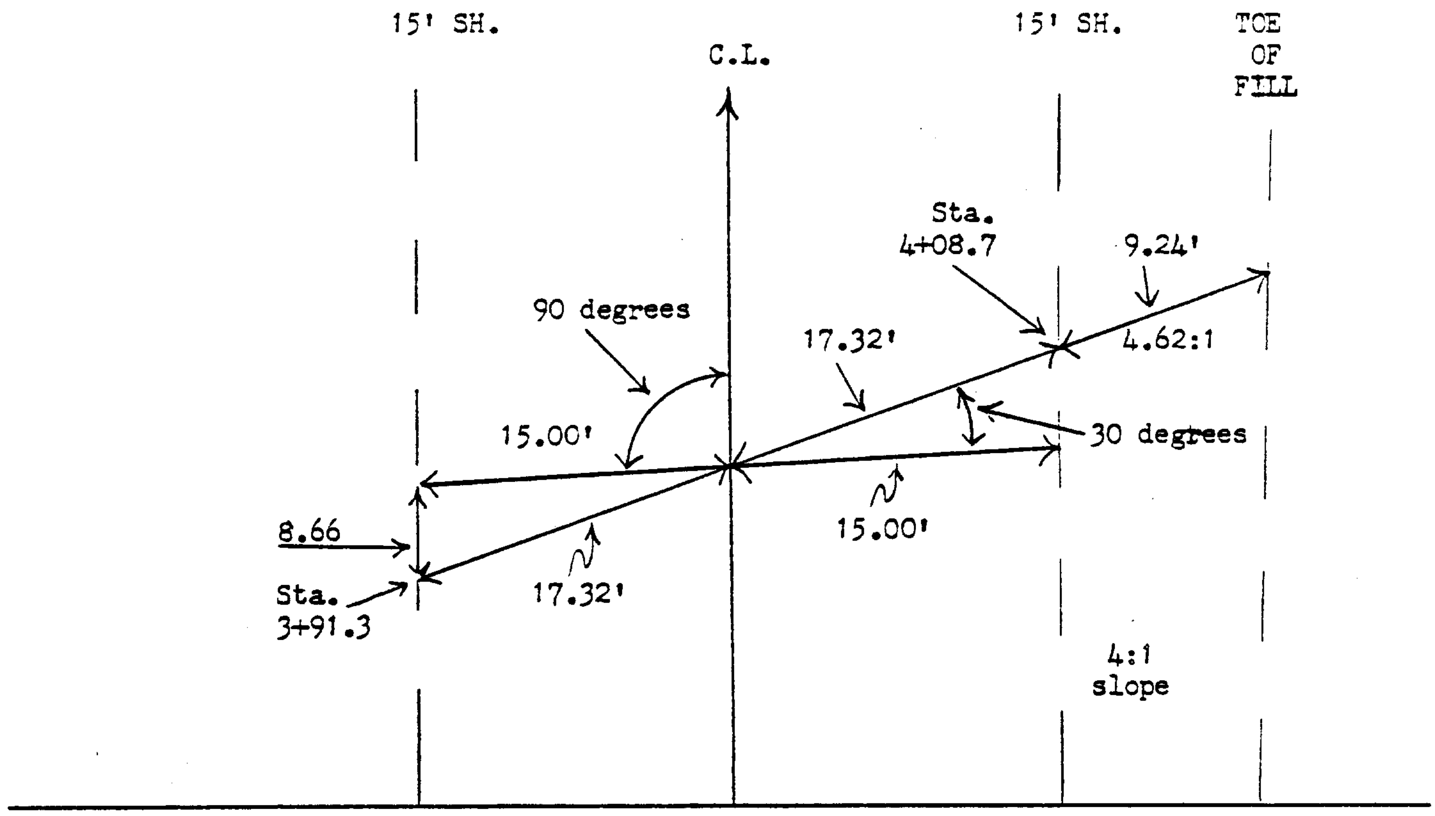
$$\sqrt{(85.37)(74.68)}$$

$$\sqrt{6371.16}$$

H.D. = 79.82

$$G = \frac{\underset{(B)}{7026.74} - \underset{(D)}{7021.37}}{\underset{(H.D.)}{79.82}} = 6.73\%\ \text{Grade}$$

SKEWED CULVERTS



CALCULATIONS: (required for computing inlet and outlet elevation and culvert plotting)

SKEWED SH. WIDTH: $\frac{15.00'}{\text{COS. 30 degrees}} = 17.32'$

SKEWED DIST. AH. AND BK.: SIN 30 degrees x 17.32' = 8.66'

SLOPE RATIO: $\frac{4*}{\text{COS. 30 degrees}} = 4.62$

SKEW ANGLE: $\frac{8.66'}{17.32'} \times \text{SIN}^{-1} = 30$ degrees

* staked slope ratio of slopes ahead and back of culvert station

SUMMARY OF ITEMS
STATION________

(example of culvert book set up)

ITEM 203(3), Unclassified Excavation

Date	Quantity	Ent. Sum. Bk./Pg.	Reference Bk. No./Pg.
6-20-81	56 C.Y.	6-20-81/42	5/23

ITEM 206(1), Structural Excavation

Date	Quantity	Ent. Sum. Bk./Pg.	Reference Bk. No./Pg.
6-23-81	26.14 C.Y.	6-23-81/13	5/19

ITEM 603(4), 24-Inch Steel End Sections

Date	Quantity	Ent. Sum. Bk./Pg.	Ht.#	Inspector	Bk./Pg.
6/23/81	1	6-23-81/40	27BC641	John Doe	5/27

ITEM 603(31E), 24-Inch Culvert Pipe

Date	Length	Ht.#	Inspector	Ent. Sum. Bk./Pg.	Bk./Pg.
6/23/81	24	64AH123	John Doe	6-30-81	5/48

(enter all pieces of culvert and date placed)

Total-------108 Lin. Ft.

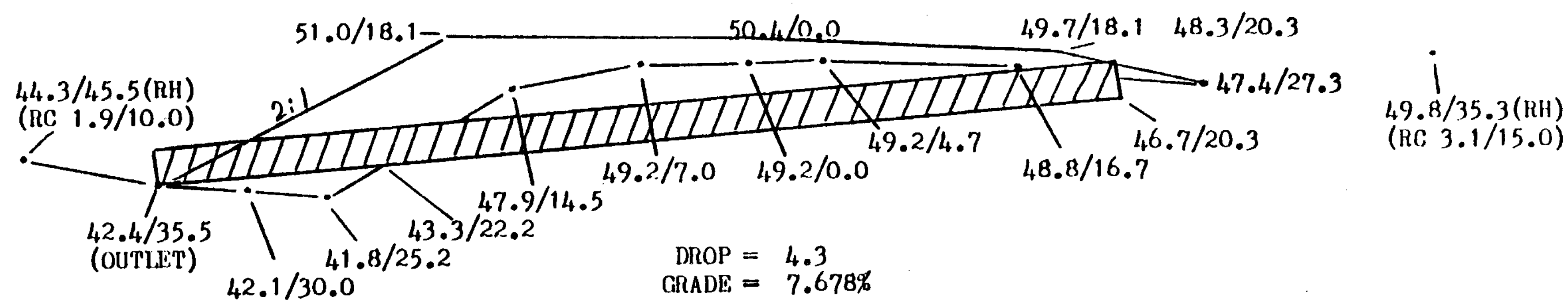
NOTE: Attempt to enter all items in numerical order

CULVERT CROSS SECTIONS AND QUANTITIES

B.M. #7 = El. 7149.00
Rod Shot = + 3.64
7152.64 (reference el.)

STATION						LT.		X-SECTION			RT.				
	El =	44.3	42.4	42.1	41.8	47.9	49.2	49.2	49.2	48.8	48.3	48.1	47.3	47.4	49.8
4+00	X-Sec. =	- 8.3 / 45.5 (RH)	-10.2 / 35.5	-10.5 / 30.0 (OUTLET)	-10.8 / 25.2	- 4.7 / 14.5	- 3.4 / 7.0	- 3.4 / 0.0 C.L.	- 3.4 / 4.7	- 3.8 / 16.7	- 4.3 / 20.3 (INLET)	- 4.5 / 21.5	- 5.4 / 23.5	- 5.2 / 27.3	- 2.8 / 35.3 (RH)

PLOTTED CROSS SECTION



DROP = 4.3
GRADE = 7.678%

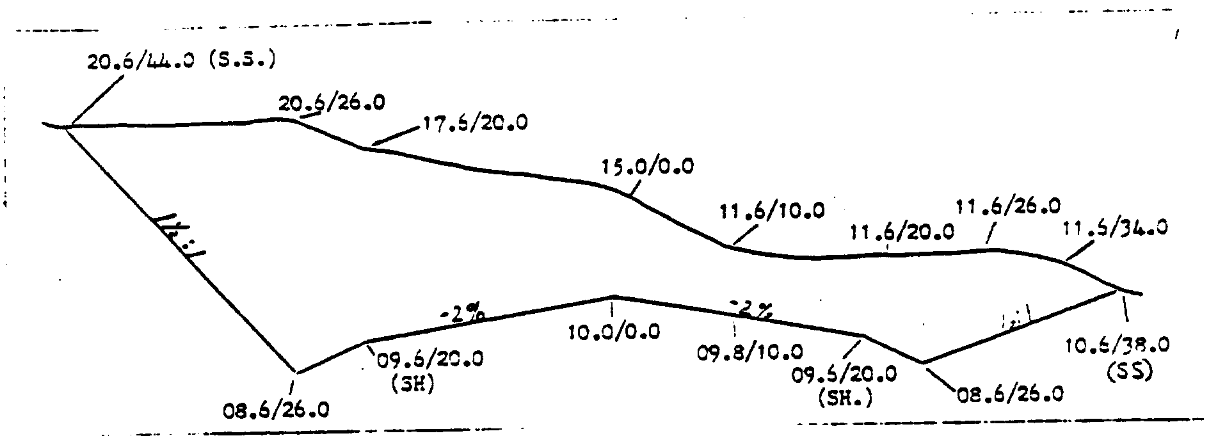
STRUCTURAL EXCAVATION

$$\frac{43.3}{0.0}\ \frac{47.9}{7.7}\ \frac{49.2}{15.2}\ \frac{49.2}{22.2}\ \frac{49.2}{26.9}\ \frac{48.8}{38.9}\ \frac{48.3}{42.5}\ \frac{46.7}{42.5}\ \frac{43.3}{0.0} = \frac{\frac{282.33 \times 5^*}{2}}{27} = 26.14 \text{ C.Y.}$$

* 24-inch culvert, pay 18 inches either side of pipe

Computed by ____________________
Checked by ____________________

CUT AND FILL QUANTITIES COMPUTATION



	C	C	C	C	C	C	C	C	C	C
0.0	12.0	12.0	8.0	5.0	1.8	2.0	3.0	3.0	2.0	0.0
26.0	44.0	26.0	20.0	0.0	10.0	20.0	26.0	34.0	38.0	26.0
	SS	DIT.	SH	C.L.		SH	DIT.			

Diff. = 0.0 + 24.0 + 26.0 + 30.0 + 20.0 + 16.0 + 14.0 + 12.0 - 8.0

(Diff. x Cut) + (Diff. x Cut) + (Diff. x Cut) - (Diff. x Cut) = $\frac{776.00 \text{ Double End A}}{2}$

= Sq. Ft. (388.00)

ELEVATION QUANTITIES COMPUTATIONS

ORIGINAL GROUND | TEMP. ELEV.

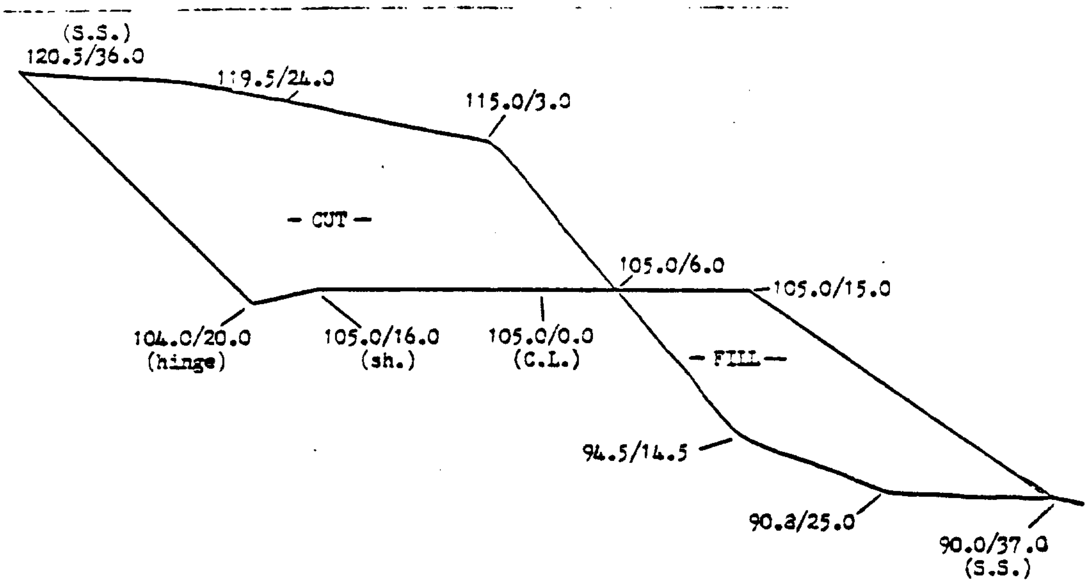
20.6	20.6	20.6	17.6	15.0	11.6	11.6	10.6	08.6	09.6	10.0	09.6
44.0	44.0	26.0	20.0	0.0	10.0	34.0	38.0	26.0	20.0	0.0	20.0

Diff.=
18.0 + 24.0 + 26.0 + 30.0 + 34.0 + 28.0 - 8.0 - 18.0 - 26.0 - 40.0 - 26.0

388.00 Sq. Ft. = $\frac{776.00}{2}$ = $\frac{20.6}{44.0}$ $\frac{20.6}{44.0}$ $\frac{08.6}{26.0}$

- 18.0 - 24.0

QUANTITY COMPUTATIONS USING ELEVATIONS

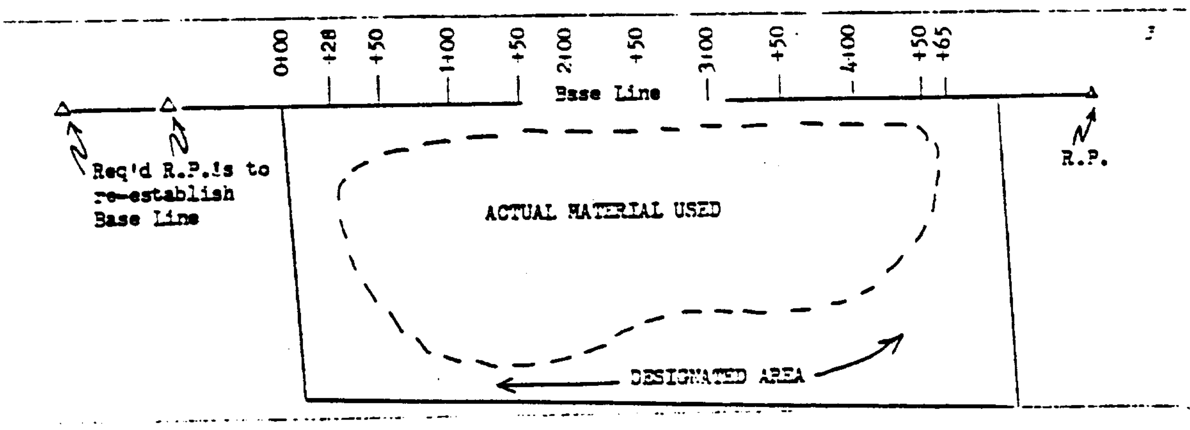


$$\text{CUT} = \frac{120.5}{0.0}\ \frac{119.5}{12.0}\ \frac{115.5}{33.0}\ \frac{105.0}{42.0}\ \frac{105.0}{20.0}\ \frac{104.0}{16.0}\ \frac{120.5}{0.0} = \frac{721.50}{2} = \underline{375.75\ \text{S.F.}}$$

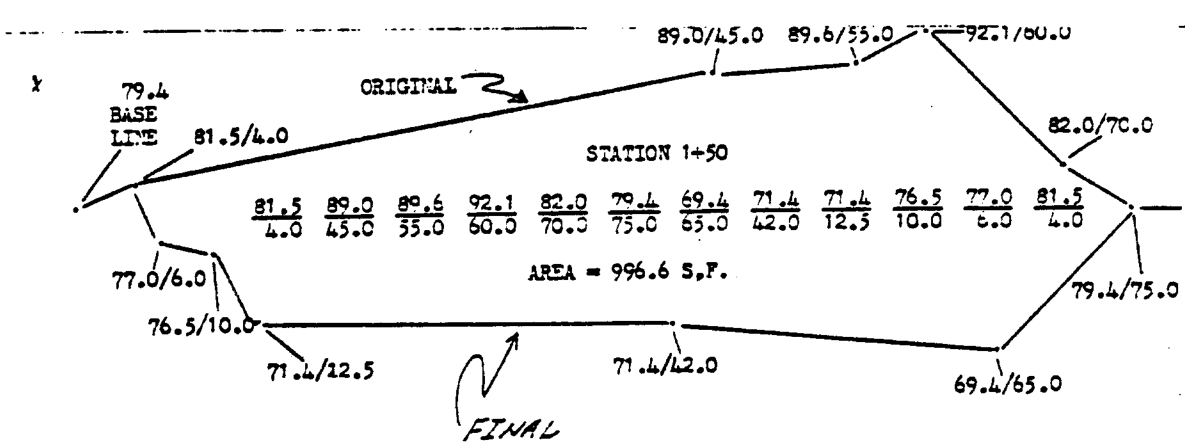
$$\text{FILL} = \frac{105.0}{0.0}\ \frac{105.0}{9.0}\ \frac{90.0}{31.0}\ \frac{90.8}{19.0}\ \frac{94.5}{8.5}\ \frac{105.0}{0.0} = \frac{369.0}{2} = \underline{184.50\ \text{S.F.}}$$

NOTE: Elevation computation eliminates the need for break shots at hinge points. If cut and fill method of computations is used, breaks will have to be interpulated along original ground wherever template changes.

BORROW PIT CROSS SECTIONING (EXAMPLE)



STATION	BASE LINE ELEVATION	CROSS SECTION (type 46, method optional)						
1+50 (original)	8679.40	81.5 / + 2.1 / 4.0	89.0 / + 9.6 / 45.0	89.6 / +10.2 / 55.0	92.1 / +12.7 / 60.0	82.0 / + 2.6 / 70.0	79.4 / 0.0 / 75.0	79.4 / 0.0 / 80.0
1+50 (final)	8679.40	77.0 / - 2.4 / 6.0	76.5 / - 2.9 / 10.0	71.4 / - 8.0 / 12.5	71.4 / - 8.0 / 42.0	69.4 / -10.0 / 65.0	79.4 / 0.0 / 75.0	



Bridge Stakeout

Bridge stakeout is the surveying network of points established to locate all points required in the construction of the bridge and the reference points required to reproduce these locations.

The points required for horizontal control would be the beginning and the ending of the bridge and the location of the abutments and bents along the centerline of the road. Other points could be required depending upon the type of the bridge and its location. These points would be located on the ground from the alignment of the roadway. These points should be referenced by the use of reference points set out on each or either side of the centerline. When setting the reference points, it must be remembered that these points will be used to reproduce the bridge location points during construction. For the construction of a bridge, the contractor will require a large area for equipment and materials used in the bridge construction. Al reference points must be set where they will not be disturbed and they can be set up over with a transit to reproduce the desired point during different stages of the bridge construction.

Other points at the bridge location that would be required would be the centerline of the abutments and bents and their footings. These points must also be referenced so they can be reproduced.

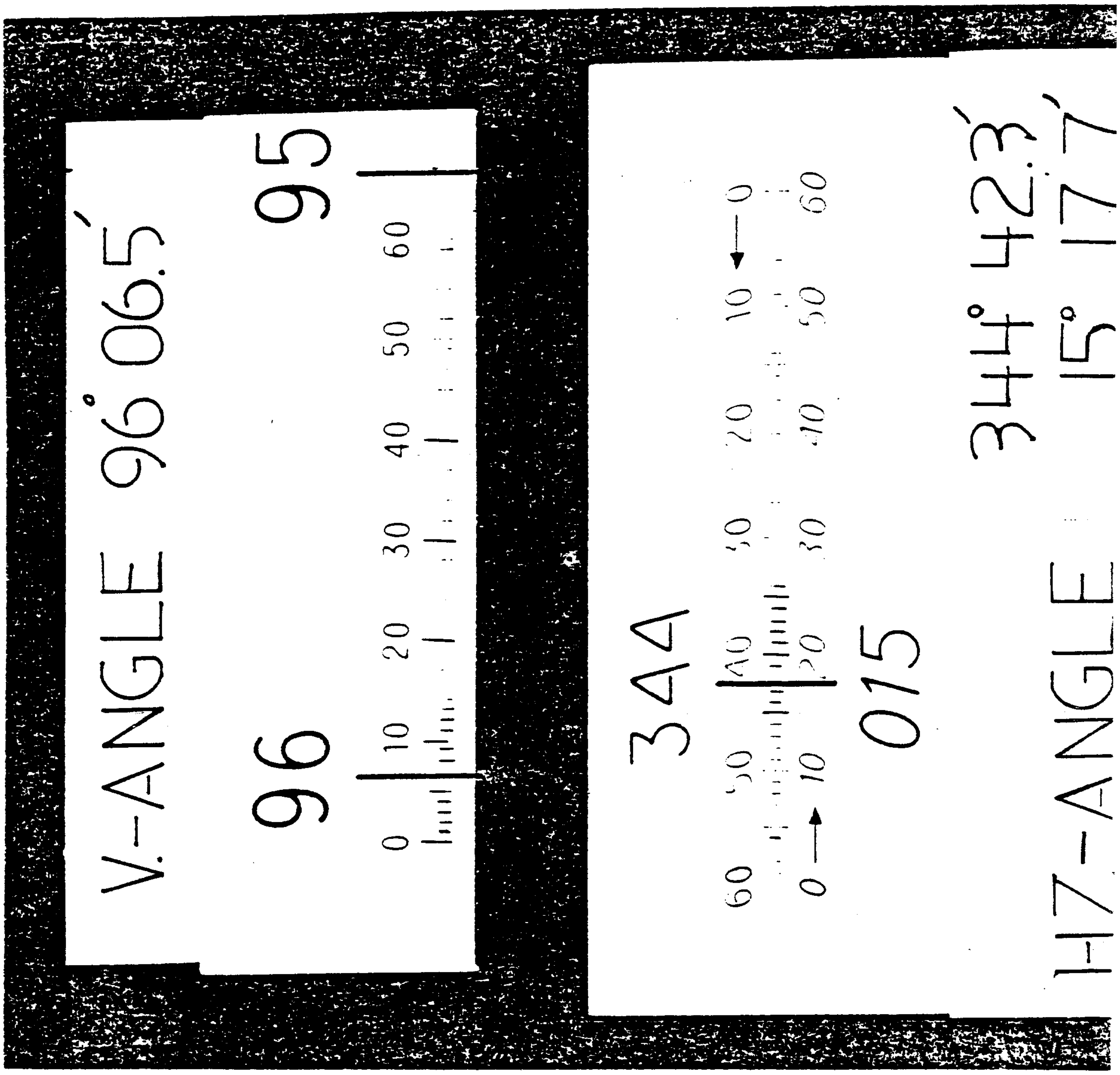
Due to the many situations encountered in bridge construction, it would be impossible to describe the proper stakeout. Only a general idea can be given. When preparing to lay out the control network, the party chief should study the plans and determine where his reference points could be located clear of construction and most advantageous for reproducing the bridge control points. As an example, if the bridge was above the surrounding landscape it would be most advantageous to set the reference points on as high ground as possible or far enough back so that the transitman could see as high up on the finished elevation of the bridge as possible. Angle chaining or breaking chain on slope type measurements aren't very accurate. If this was required to reproduce the desired points, it might be advantageous to cross reference them so they could be located by the intersection of two straight lines. It might also be very hard to chain out too accurately the intersection of the centerlines of the bridge and an abutment if the contractor was pouring concrete into these abutment forms.

Another possibility would be setting out an offset line with the required points located on it.

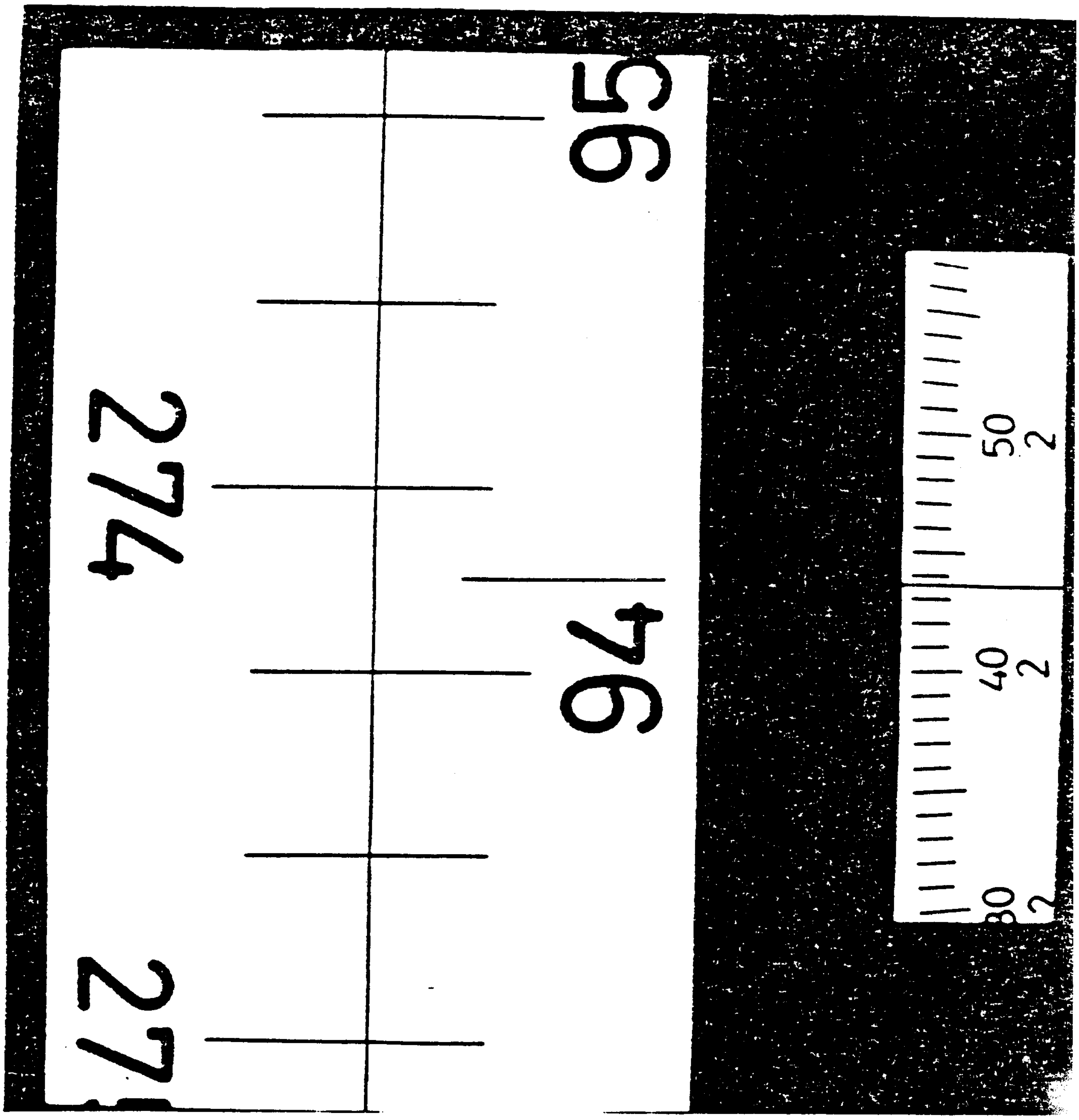
It is generally required that the location and alignment of the keyways, beam seats, and other construction elements require being located and checked for location while the concrete or other material is being placed. You do not have time to go back to the roadway alignment points and rerun in the bridge control points. In other words, you must have your control network set up so that you can check location and alignment in a hurry.

For horizontal control a base benchmark should be placed convenient to the bridge but clear of the construction work so that it will not be disturbed. Temporary benchmarks should be located in advantageous points for checking the elevation of abutment and bent footings, top of bents, finish grade, and so forth. These temporary benchmarks should be placed so that the elevation of the points can be determined without having to make more than one setup with the level.

Bridge location surveying and stakeout should be done to a much greater accuracy than regular highway surveying. It is possible to change the location or elevation of a highway several inches or even a foot or more without adding a large amount of grading work, but once you have set the bridge points, the bridge location has been set on the ground. The bridge will have to stay in this location as it would cost too much to move it. After the footings or foundation parts of the bridge have been set and constructed, the bridge location cannot be changed. The only way to be sure of that is to be able to reproduce the bridge control points in exactly the same location every time it is required during construction.



T-16
DOUBLE VERNIER



T-2

HZ-ANGLE 94° 12′ 43″6

New T-16

Horizontal Adjustment

1. Set instrument up level.

2. Take a sight at least 200 feet away to observe horizontal readings.

3. Turn 180^{o} to set sight, observe readings.

4. Reverse scope and repeat 2 and 3; readings should be within 0-00-30 a minute.

5. To adjust, remove cover at eye piece. The adjustment is carried out by means of the two horizontal capstan headed adjustment screws. Turn screws either left or right equal amounts, taking out 1/4 of the error.

6. Repeat steps 2 through 4 to check adjustments.

New T-16 with Automatic Collimation

1. Set instrument up on tripod, level all positions. If bubble will not remain centered within one division for all positions, adjust as follows:

    a. Turn the instrument clockwise through 180°. Note position of the bubble. Bring the bubble to a point halfway between this position and central position by turning foot screw. With an adjusting pin, carefully turn the adjustment screw until the bubble is centered.

    b. Repeat until the bubble remains centered within one division for all positions of the instrument.

Optical Plummet

1. Place a piece of paper on the floor with tape. Mark a fine point on the paper (1).

2. Point (1) set T-16 on tripod over paper and cross hairs on point (1). Now turn instrument 180° and mark position on paper very carefully, which would be point (2). If the points coincide, the plummet is in adjustment.

3. If out more than 0.005, it must be adjusted.

4. To adjust, mark a point (3) halfway between point (1) and point (2). By turning the foot screws, set cross hairs on point (3). Loosen four screws in the plate around the optical plummet eye piece, carefully move plummet and plate until cross hairs are exactly on point (1). Tighten four screws, and repeat test to check adjustment.

Vertical Angle

1. Set instrument up on tripod.

2. Take a sight on a high, well-defined point at least 200 feet away.

3. Observe angle, then reverse scope and set on the same point. Add the readings together. They should read 360° if in adjustment.

4. If the difference is more than 0-01-00, adjust.

    a. To adjust, open the cover above the mirror with a screw driver. Turn the adjustment scew until proper reading is obtained.

5. Repeat steps 2 through 4 to check adjustment.

Level Peg Check

1. Set two solid points 200 feet apart.

2. Set level halfway between points.

3. Take readings on point A, then on Point B (three wire or center wire). Subtract readings from each other, and this should be true difference between points.

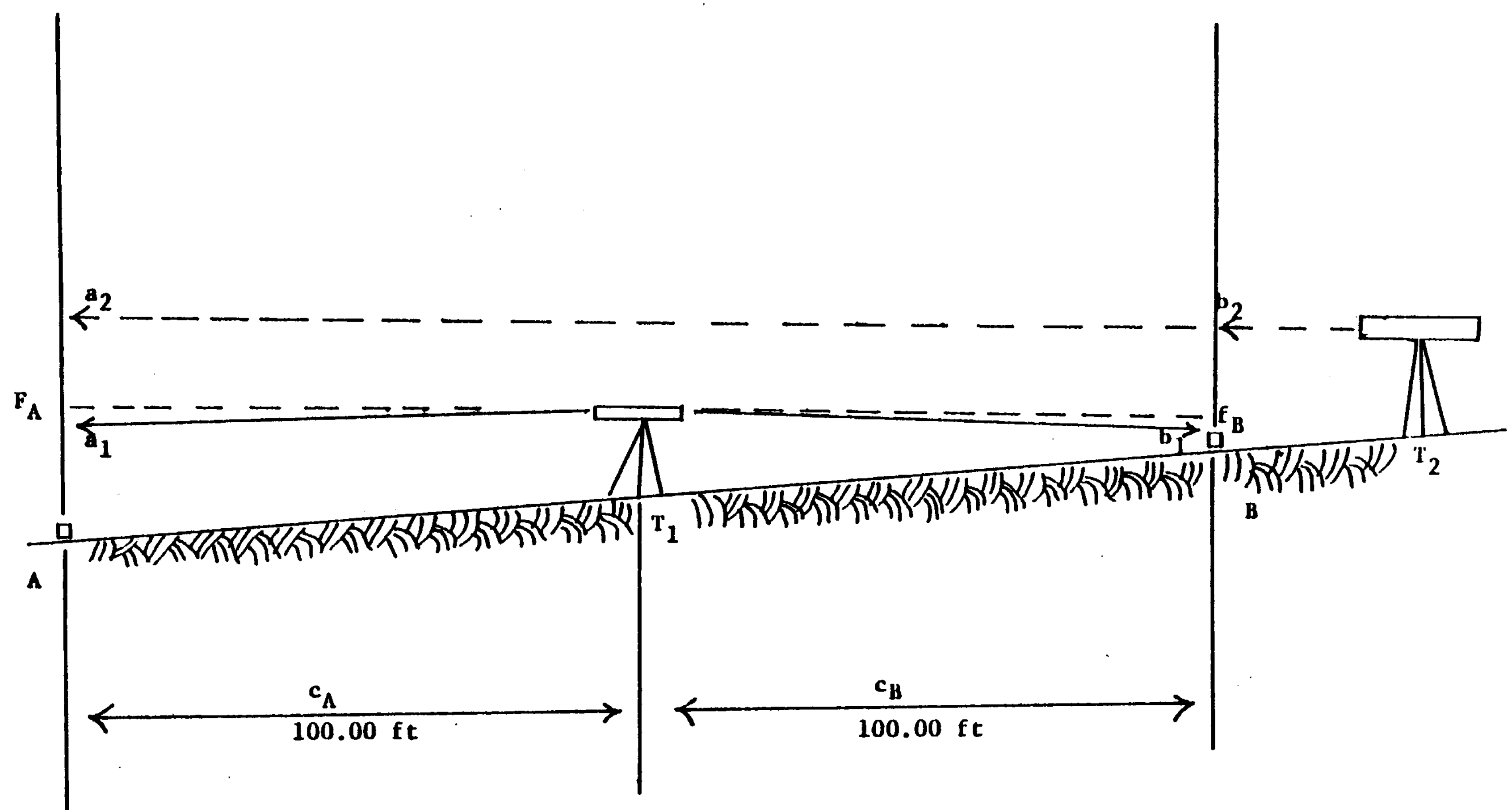
4. Move level + 12 feet past point A or B.

5. Set level up and take readings on short sight.

6. Take readings on long sight--subtract readings. If the level is in adjustment, the difference should be the same as in step 3, within 0.002.

7. Adjust level on long sight by turning the capstan screw left or right, depending on what reading is needed.

8. Repeat procedures 2 through 6 as a check to see if adjustment was in the right direction.

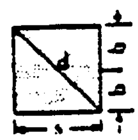


$$\begin{array}{r} a_1 = 2.423 \\ -\ b_1 = 0.936 \\ \hline d = 1.487 \end{array}$$

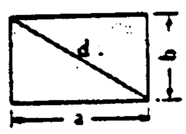
$$\begin{array}{r} b_2 = 1.462 \\ +\ d = 1.487 \\ \hline \text{thus } a_2 \text{ should be } 2.949 \end{array}$$

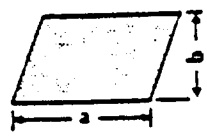
TABLE IV-36 AREAS OF PLANE FIGURES

Square



Diagonal $= d = s\sqrt{2}$.
Area $= s^2 = 4b^2 = 0.5d^2$.
Example. $s = 6$; $b = 3$. Area $= (6)^2 = 36$ Ans.
$d = 6 \times 1.414 = 8.484$ Ans.

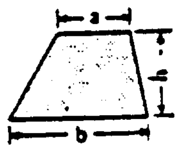




Rectangle and Parallelogram

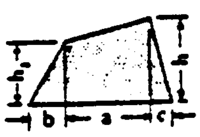
Area $= ab$ or $b\sqrt{d^2 - b^2}$
Example. $a = 6$; $b = 3$.
Area $= 3 \times 6 = 18$ Ans

Trapezoid



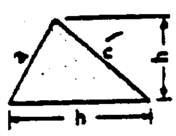
Area $= \frac{1}{2}h(a + b)$
Example. $a = 2$; $b = 4$; $h = 3$.
Area $= \frac{1}{2} \times 3(2 + 4) = 9$. Ans.

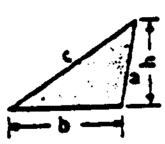
Trapezium



Area $= \frac{1}{2}[a(h + h^1) + bh^1 + ch]$
Example. $a = 4$; $b = 2$; $c = 2$; $h = 3$; $h^1 = 2$.
Area $= \frac{1}{2}[4(3 + 2) + (2 \times 2) + (2 \times 3)] = 15$. Ans.

Triangles



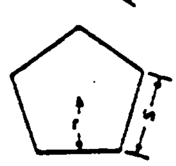


Both formulas apply to both figures
Area $= \frac{1}{2}bh$.
Example. $h = 3$; $b = 5$.
Area $= \frac{1}{2}(3 \times 5) = 7\frac{1}{2}$. Ans.

Area $= \sqrt{S(S - a)(S - b)(S - c)}$ when $S = \frac{a + b + c}{2}$

Example. $a = 2$; $b = 3$; $c = 4$.
$S = \frac{2 + 3 + 4}{2} = 4.5$
Area $= \sqrt{4.5(4.5 - 2)(4.5 - 3)(4.5 - 4)} = 2.9$. Ans.

Regular Polygons



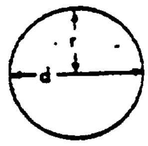
	Sides		
Area	5 sides	$= 1.720477\ S^2$	$= 3.63271\ r^2$
	6 "	$= 2.598150\ S^2$	$= 3.46410\ r^2$
	7 "	$= 3.633875\ S^2$	$= 3.37101\ r^2$
	8 "	$= 4.828427\ S^2$	$= 3.31368\ r^2$
	9 "	$= 6.181875\ S^2$	$= 3.27573\ r^2$
	10 "	$= 7.694250\ S^2$	$= 3.24920\ r^2$
	11 "	$= 9.365675\ S^2$	$= 3.22993\ r^2$
	12 "	$= 11.196300\ S^2$	$= 3.21539\ r^2$

n = number of sides; r = short radius;
S = length of side; R = long radius.

$$\text{Area} = \frac{n}{4} S^2 \cot.\frac{180^\circ}{n} = \frac{n}{2} R^2 \sin.\frac{360^\circ}{n}$$
$$= nr^2 \tan\frac{180^\circ}{n}$$

TABLE IV-36 (continued)

Circle



$\pi = 3.1416$; A = area; d = diameter; p = circumference or periphery; r = radius.

$p = \pi d = 3.1416d$. $\quad p = 2\sqrt{\pi A} = 3.54\sqrt{A}$

$p = 2\pi r = 6.2832r$. $\quad p = \frac{2A}{r} = \frac{4A}{d}$

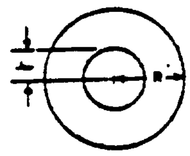
$d = \frac{p}{\pi} = \frac{p}{3.1416} \quad d = 2\sqrt{\frac{A}{\pi}} = 1.128\sqrt{A}$

$r = \frac{p}{2\pi} = \frac{p}{6.2832} \quad r = \sqrt{\frac{A}{\pi}} = 0.564\sqrt{A}$

$A = \frac{\pi d^2}{4} = 0.7854d^2 \quad A = \frac{p^2}{4\pi} = \frac{p^2}{12.57}$

$A = \pi r^2 = 3.1416r^2 \quad A = \frac{pr}{2} = \frac{pd}{4}$

Circular Ring



Area $= \pi(R^2 - r^2) = 3.1416(R^2 - r^2)$

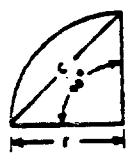
Area $= 0.7854(D^2 - d^2) = 0.7854(D - d)(D + d)$

Area = difference in areas between the inner and outer circles.

Example. $R = 4$; $r = 2$.

Area $= 3.1416(4^2 - 2^2) = 37.6992$. Ans.

Quadrant

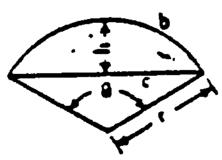


Area $= \frac{\pi r^2}{4} = 0.7854r^2 = 0.3927c^2$.

Example. $r = 3$. c = chord.

Area $= .7854 \times 3^2 = 7.0686$. Ans.

Segment



b = length of arc. θ = angle in degrees

c = chord $= \sqrt{4(2hr - h^2)}$

Area $= \frac{1}{2}[br - c(r - h)]$

$= \pi r^2 \frac{\theta}{360} - \frac{c(r - h)}{2}$

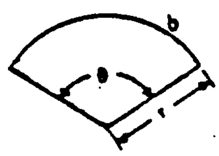
When θ is greater than 180° then $\frac{c}{2} \times$ difference between r and h is added to the fraction $\frac{\pi r^2 \theta}{360}$

Example. $r = 3$; $\theta = 120°$; $h = 1.5$

Area $= 3.1416 \times 3^2 \times \frac{120}{360} - \frac{5.196(3 - 1.5)}{2}$

$= 5.5278$. Ans.

Sector



Area $= \frac{br}{2} = \pi r^2 \frac{\theta}{360°}$

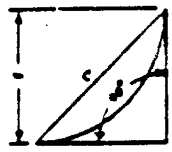
θ = angle in degrees; b = length of arc.

Example. $r = 3$; $\theta = 120°$

Area $= 3.1416 \times 3^2 \times \frac{120}{360} = 9.4248$. Ans.

— —

TABLE IV-36 (continued)



Spandrel

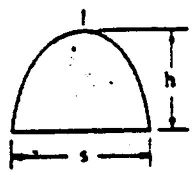
Area $= 0.2146r^2 = 0.1073c^2$
Example. $r = 3$
Area $= 0.2146 \times 3^2 = 1.9314$. Ans

Parabola

l = length of curved line = periphery $- s$

$$l = \frac{s^2}{8h}\left[\sqrt{c(1+c)} + 2.0326 \times \log(\sqrt{c}+\sqrt{1+c})\right]$$

in which $c = \left(\frac{4h}{s}\right)^2$

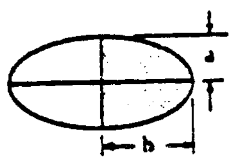


Area $= \frac{2}{3}sh$
Example. $s = 3$; $h = 4$
Area $= \frac{2}{3} \times 3 \times 4 = 8$. Ans.

Ellipse

Area $= \pi ab = 3.1416ab$

Circum. $= 2\pi\sqrt{\frac{a^2 + b^2}{2}}$ (close approximation)



Example. $a = 3$; $b = 4$.
Area $= 3.1416 \times 3 \times 4 = 37.6992$. Ans.

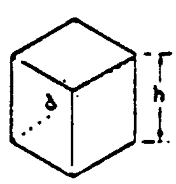
Circum. $= 2 \times 3.1416\sqrt{\frac{(3)^2 + (4)^2}{2}}$
$= 6.2832 \times 3.5355 = 22.21$ Ans.

TABLE IV-37 VOLUME AND SURFACE AREA OF SOLIDS

Symbols

V = Volume
S = Lateral Surface Area
T = Total Surface Area
B = Area of Base
P = Perimeter Perpendicular to Sides
P_b = Perimeter of Base
A = Area of Section Perpendicular to Sides
l = Lateral Length
h = Perpendicular Height
d = Diagonal Length

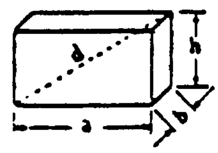
Cube



$V = h^3$
$T = 6h^2$
$S = 4h^2$
$d = h\sqrt{3}$

TABLE IV-37 (Continued)

Rectangular Prism



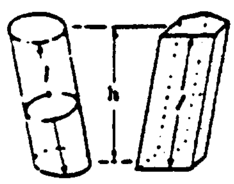
$V = abh$

$T = 2(ab + ah + bh)$

$S = 2(ah + bh)$

$d = \sqrt{a^2 + b^2 + h^2}$

Prism or Cylinder, Right or Oblique, Parallel Ends



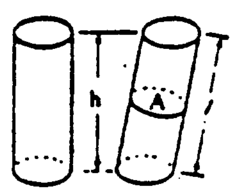
$V = Al$

$S = Pl$

$T = Pl + 2B$

(Note $A = B$, $P = P_h$ and $l = h$ for right cylinders and prisms

Cylinder, Right or Oblique, Circular or Otherwise, Parallel Ends



$V = Bh$ (Right Cylinder)

$V = Al$ (Oblique Cylinder)

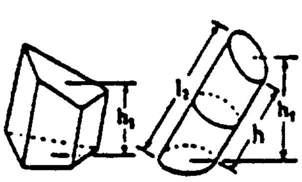
$S = P_h h$ (Right Cylinder)

$S = Pl$ (Oblique Cylinder)

$T = P_h h + 2B$ (Right Cylinder)

$T = Pl + 2B$ (Oblique Cylinder)

Frustrum of Prism or Cylinder

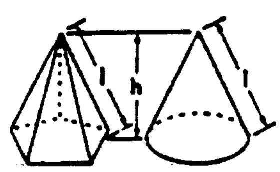


$V = Bh_1$ (where h_1 is perpendicular height from base to c.g. of top)

or, for cylinder

$V = \frac{A}{2}(l_1 + l_2)$

Pyramid or Cone, Right and Regular

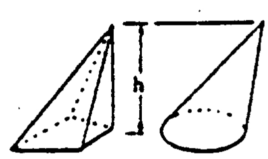


$V = \frac{Bh}{3}$

$S = \frac{P_b l}{2}$

$T = \frac{P_b l}{2} + B$

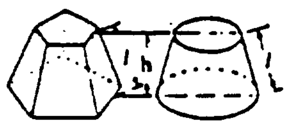
Pyramid or Cone, Right or Oblique, Regular or Irregular



$V = \frac{Bh}{3}$

TABLE IV-37 (Continued)

Frustrum of Pyramid or Cone, Right and Regular, Parallel Ends



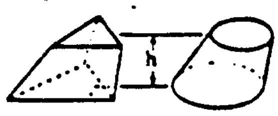
$$V = \frac{h}{3}(B + B_1 + \sqrt{BB_1})$$

$$S = \frac{l}{2}(P_b + P_T)$$

$$T = \frac{l}{2}(P_b + P_T) + B + B_1$$

where: B_1 = Area of Top
P_T = Perimeter of Top

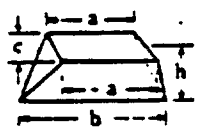
Frustrum of Any Pyramid or Cone, Parallel Ends



$$V = \frac{h}{3}(B + B_1 + \sqrt{BB_1})$$

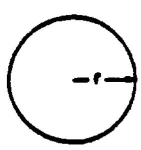
where: B_1 = Area of Top

Wedge, Regular



$$V = \frac{ch}{6}(2a + b)$$

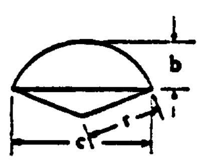
Sphere



$$V = \frac{4\pi r^3}{3}$$

$$S = 4\pi r^2$$

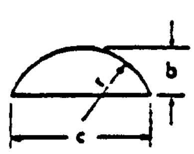
Spherical Sector



$$S = \frac{1}{2}\pi r(4b + c)$$

$$V = \frac{2}{3}\pi r^2 b$$

Spherical Segment



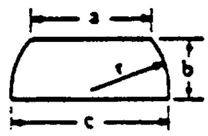
$$S = 2\pi rb = \frac{1}{4}\pi(4b^2 + c^2)$$

$$V = \frac{1}{3}\pi b^2(3r - b)$$

$$= \frac{1}{24}\pi b(3c^2 + 4b^2)$$

TABLE IV-37 (Continued)

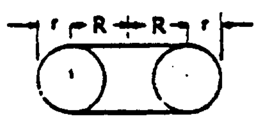
Spherical Zone



$$S = 2\pi rb$$

$$V = \frac{1}{24}\pi b(3a^2 + 3c^2 + 4b^2)$$

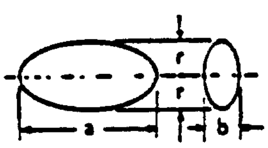
Circular Ring



$$S = 4\pi^2 Rr$$

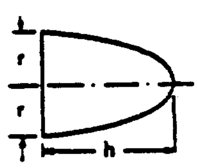
$$V = 2\pi^2 Rr^2$$

Ellipsoid



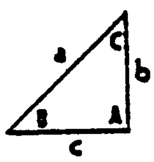
$$V = \frac{1}{3}\pi rab$$

Paraboloid



$$V = \frac{1}{2}\pi r^2 h$$

TABLE IV-38 TRIGONOMETRIC RELATIONS AND SOLUTIONS OF RIGHT ANGLE TRIANGLES



As shown in the illustration, the sides of the right angled triangle are designated *a*, *b* and *c*. The angles opposite each of these sides are designated *A*, *B* and *C* respectively.

Angle *A*, opposite the hypotenuse *a* is the right angle and is therefore always one of the known quantities.

Sides and Angles Known	Formulas for Sides and Angles to be Found		
Sides a and b.....	$c = \sqrt{a^2 - b^2}$	$\sin B = \frac{b}{a}$	$C = 90° - B$
Sides a and c.....	$b = \sqrt{a^2 - c^2}$	$\sin C = \frac{c}{a}$	$B = 90° - C$
Sides b and c.....	$a = \sqrt{b^2 + c^2}$	$\tan B = \frac{b}{c}$	$C = 90° - B$
Side a; angle B....	$b = a \times \sin B$	$c = a \times \cos B$	$C = 90° - B$
Side a; angle C....	$b = a \times \cos C$	$c = a \times \sin C$	$B = 90° - C$
Side b; angle B....	$a = \frac{b}{\sin B}$	$c = b \times \cot B$	$C = 90° - B$
Side b; angle C....	$a = \frac{b}{\cos C}$	$c = b \times \tan C$	$B = 90° - C$
Side c; angle B....	$a = \frac{c}{\cos B}$	$b = c \times \tan B$	$C = 90° - B$
Side c; angle C....	$a = \frac{c}{\sin C}$	$b = c \times \cot C$	$B = 90° - C$

TABLES OF WEIGHTS AND MEASURES

Linear Measure

1 inch			=	2.54	centimeters
12 inches	=	1 foot	=	0.3048	meter
3 feet	=	1 yard	=	0.9144	meter
5½ yards or 16½ feet	=	1 rod (or pole or perch)	=	5.029	meters
40 rods	=	1 furlong	=	201.17	meters
8 furlongs or 1,760 yards or 5,280 feet	=	1 (statute) mile	=	1,609.3	meters
3 miles	=	1 (land) league	=	4.83	kilometers

Square Measure

1 square inch			=	6.452	square centimeters
144 square inches	=	1 square foot	=	929	square centimeters
9 square feet	=	1 square yard	=	0.8361	square meter
30¼ square yards	=	1 square rod (or square pole or square perch)	=	25.29	square meters
160 square rods or 4,840 square yards or 43,560 square feet	=	1 acre	=	0.4047	hectare
640 acres	=	1 square mile	=	259	hectares or 2.59 square kilometers

Cubic Measure

1 cubic inch			=	16.387 cubic centimeters
1,728 cubic inches	=	1 cubic foot	=	0.0283 cubic meter
27 cubic feet	=	1 cubic yard	=	0.7646 cubic meter
(in units for cordwood, etc.)				
16 cubic feet	=	1 cord foot		
8 cord feet	=	1 cord	=	3.625 cubic meters

Chain Measure

(for Gunter's, or surveyor's, chain)

7.92 inches	=	1 link	=	20.12	centimeters
100 links or 66 feet	=	1 chain	=	20.12	meters
10 chains	=	1 furlong	=	201.17	meters
80 chains	=	1 mile	=	1,609.3	meters

(for engineer's chain)

1 foot	=	1 link	=	0.3048	meter
100 feet	=	1 chain	=	30.48	meters
52.8 chains	=	1 mile	=	1,609.3	meters

Surveyor's (Square) Measure

625 square links	=	1 square pole	=	25.29	square meters
16 square poles	=	1 square chain	=	404.7	square meters
10 square chains	=	1 acre	=	0.4047	hectare
640 acres	=	1 square mile or 1 section	=	259	hectares or 2.59 square kilometers
36 square miles	=	1 township	=	9,324.0	hectares or 93.24 square kilometers

Nautical Measure

6 feet	=	1 fathom	=	1.829 meters
100 fathoms	=	1 cable's length (ordinary) (In the U.S. Navy 120 fathoms or 720 feet = 1 cable's length; in the British Navy, 608 feet = 1 cable's length.)		
10 cables' lengths	=	1 nautical mile (6,076.10333 feet, by international agreement in 1954)	=	1.852 kilometers
1 nautical mile (Also called geographical, sea, or air mile, and, in Great Britain, Admiralty mile.)	=	1.1508 statute miles (the length of a minute of longitude at the equator)		
3 nautical miles	=	1 marine league (3.45 statute miles)	=	5.56 kilometers
60 nautical miles	=	1 degree of a great circle of the earth		

Dry Measure

1 pint			=	33.60 cubic inches	=	0.5505 liter
2 pints	=	1 quart	=	67.20 cubic inches	=	1.1012 liters
8 quarts	=	1 peck	=	537.61 cubic inches	=	8.8096 liters
4 pecks	=	1 bushel	=	2,150.42 cubic inches	=	35.2383 liters

1 British dry quart = 1.032 U.S. dry quarts.

According to United States government standards, the following are the weights avoirdupois for single bushels of the specified grains: for wheat, 60 pounds; for barley, 48 pounds; for oats, 32 pounds; for rye, 56 pounds; for corn, 56 pounds. Some States have specifications varying from these.

Liquid Measure

1 gill	=	4 fluid ounces (see next table)	=	7.219 cubic inches	=	0.1183 liter
4 gills	=	1 pint	=	28.875 cubic inches	=	0.4732 liter
2 pints	=	1 quart	=	57.75 cubic inches	=	0.9463 liter
4 quarts	=	1 gallon	=	231 cubic inches	=	3.7853 liters

The British imperial gallon (4 imperial quarts) = 277.42 cubic inches = 4.546 liters. The barrel in Great Britain equals 36 imperial gallons, in the United States, usually 31½ gallons.

Apothecaries' Fluid Measure

1 minim		= 0.0038 cubic inch	= 0.0616 milliliter
60 minims	= 1 fluid dram	= 0.2256 cubic inch	= 3.6966 milliliters
8 fluid drams	= 1 fluid ounce	= 1.8047 cubic inches	= 0.0296 liter
16 fluid ounces	= 1 pint	= 28.875 cubic inches	= 0.4732 liter

See table immediately preceding for quart and gallon equivalents.
The British pint = 20 fluid ounces.

Circular (or Angular) Measure

60 seconds (″)	= 1 minute (′)
60 minutes	= 1 degree (°)
90 degrees	= 1 quadrant or 1 right angle
4 quadrants or 360 degrees	= 1 circle

Avoirdupois Weight

(The grain, equal to 0.0648 gram, is the same in all three tables of weight)

1 dram or 27.34 grains		= 1.772 grams
16 drams or 437.5 grains	= 1 ounce	= 28.3495 grams
16 ounces or 7,000 grains	= 1 pound	= 453.59 grams
100 pounds	= 1 hundredweight	= 45.36 kilograms
2,000 pounds	= 1 ton	= 907.18 kilograms

In Great Britain, 14 pounds (6.35 kilograms) = 1 stone, 112 pounds (50.80 kilograms) = 1 hundredweight, and 2,240 pounds (1,016.05 kilograms) = 1 long ton.

Troy Weight

(The grain, equal to 0.0648 gram, is the same in all three tables of weight)

3.086 grains	= 1 carat	= 200 milligrams
24 grains	= 1 pennyweight	= 1.5552 grams
20 pennyweights or 480 grains	= 1 ounce	= 31.1035 grams
12 ounces or 5,760 grains	= 1 pound	= 373.24 grams

Apothecaries' Weight

(The grain, equal to 0.0648 gram, is the same in all three tables of weight)

20 grains	= 1 scruple	= 1.296 grams
3 scruples	= 1 dram	= 3.888 grams
8 drams or 480 grains	= 1 ounce	= 31.1035 grams
12 ounces or 5,760 grains	= 1 pound	= 373.24 grams

THE METRIC SYSTEM

Linear Measure

10 millimeters	= 1 centimeter	= 0.3937 inch
10 centimeters	= 1 decimeter	= 3.937 inches
10 decimeters	= 1 meter	= 39.37 inches or 3.28 feet
10 meters	= 1 decameter	= 393.7 inches
10 decameters	= 1 hectometer	= 328 feet 1 inch
10 hectometers	= 1 kilometer	= 0.621 mile
10 kilometers	= 1 myriameter	= 6.21 miles

Square Measure

100 square millimeters	= 1 square centimeter	= 0.15499 square inch
100 square centimeters	= 1 square decimeter	= 15.499 square inches
100 square decimeters	= 1 square meter	= 1,549.9 square inches or 1.196 square yards
100 square meters	= 1 square decameter	= 119.6 square yards
100 square decameters	= 1 square hectometer	= 2.471 acres
100 square hectometers	= 1 square kilometer	= 0.386 square mile

Land Measure

1 square meter	= 1 centiare	= 1,549.9 square inches
100 centiares	= 1 are	= 119.6 square yards
100 ares	= 1 hectare	= 2.471 acres
100 hectares	= 1 square kilometer	= 0.386 square mile

Volume Measure

1,000 cubic millimeters	= 1 cubic centimeter	= .06102 cubic inch
1,000 cubic centimeters	= 1 cubic decimeter	= 61.02 cubic inches
1,000 cubic decimeters	= 1 cubic meter (the unit is called a *stere* in measuring firewood)	= 35.314 cubic feet

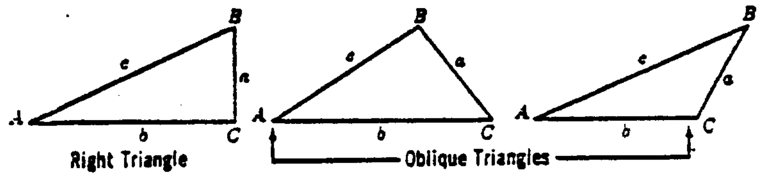
Capacity Measure

10 milliliters	= 1 centiliter	= .338 fluid ounce
10 centiliters	= 1 deciliter	= 3.38 fluid ounces
10 deciliters	= 1 liter	= 1.0567 liquid quarts or 0.9081 dry quart
10 liters	= 1 decaliter	= 2.64 gallons or 0.284 bushel
10 decaliters	= 1 hectoliter	= 26.418 gallons or 2.838 bushels
10 hectoliters	= 1 kiloliter	= 264.18 gallons or 35.315 cubic feet

Weights

10 milligrams	= 1 centigram	= 0.1543 grain
10 centigrams	= 1 decigram	= 1.5432 grains
10 decigrams	= 1 gram	= 15.432 grains
10 grams	= 1 decagram	= 0.3527 ounce
10 decagrams	= 1 hectogram	= 3.5274 ounces
10 hectograms	= 1 kilogram	= 2.2046 pounds
10 kilograms	= 1 myriagram	= 22.046 pounds
10 myriagrams	= 1 quintal	= 220.46 pounds
10 quintals	= 1 metric ton	= 2,204.6 pounds

SOLUTION OF TRIANGLES



Solution of Right Triangles

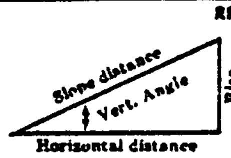
For Angle A. $\sin = \frac{a}{c}$, $\cos = \frac{b}{c}$, $\tan = \frac{a}{b}$, $\cot = \frac{b}{a}$, $\sec = \frac{c}{b}$, $\operatorname{cosec} = \frac{c}{a}$

Given	Required	
a, b	A, B, c	$\tan A = \frac{a}{b} = \cot B$, $c = \sqrt{a^2+b^2} = a\sqrt{1+\frac{b^2}{a^2}}$
a, c	A, B, b	$\sin A = \frac{a}{c} = \cos B$, $b = \sqrt{(c+a)(c-a)} = c\sqrt{1-\frac{a^2}{c^2}}$
A, a	B, b, c	$B = 90° - A$, $b = a \cot A$, $c = \frac{a}{\sin A}$
A, b	B, a, c	$B = 90° - A$, $a = b \tan A$, $c = \frac{b}{\cos A}$
A, c	B, a, b	$B = 90° - A$, $a = c \sin A$, $b = c \cos A$

Solution of Oblique Triangles

Given	Required	
A, B, a	b, c, C	$b = \frac{a \sin B}{\sin A}$, $C = 180° - (A+B)$, $c = \frac{a \sin C}{\sin A}$
A, a, b	B, c, C	$\sin B = \frac{b \sin A}{a}$, $C = 180° - (A+B)$, $c = \frac{a \sin C}{\sin A}$
a, b, C	A, B, c	$A+B = 180° - C$, $\tan \frac{1}{2}(A-B) = \frac{(a-b)\tan \frac{1}{2}(A+B)}{a+b}$, $c = \frac{a \sin C}{\sin A}$
a, b, c	A, B, C	$s = \frac{a+b+c}{2}$, $\sin \frac{1}{2}A = \sqrt{\frac{(s-b)(s-c)}{b\,c}}$, $\sin \frac{1}{2}B = \sqrt{\frac{(s-a)(s-c)}{a\,c}}$, $C = 180° - (A+B)$
a, b, c	Area	$s = \frac{a+b+c}{2}$, area $= \sqrt{s(s-a)(s-b)(s-c)}$
A, b, c	Area	area $= \frac{b\,c \sin A}{2}$
A, B, C, a	Area	area $= \frac{a^2 \sin B \sin C}{2 \sin A}$

REDUCTION TO HORIZONTAL



Horizontal distance=Slope distance multiplied by the cosine of the vertical angle. Thus: slope distance=319.4 ft. Vert. angle=5° 10′. From Table, Page X, cos 5° 10′=.9959. Horizontal distance=319.4×.9959=318.09 ft.
Horizontal distance also=Slope distance minus slope distance times (1—cosine of vertical angle). With the same figures as in the preceding example, the following result is obtained. Cosine 5° 10′=.9959. 1—.9959=.0041, 319.4×.0041=1.31. 319.4—1.31=318.09 ft.

When the rise is known, the horizontal distance is approximately: the slope distance less the square of the rise divided by twice the slope distance. Thus: rise=14 ft., slope distance=302.6 ft. Horizontal distance $= 302.6 - \frac{14\times14}{2\times302.6} = 302.6 - 0.32 = 302.28$ ft.

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