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Guidance Note

Topographic, engineering, land and measured building surveying – Survey and mapping techniques

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Reference documentation

NR/L2/EDT/002, *Computer Aided Design Drawing Specification*

NR/L2/MTC/088, *Arrangements for maintenance of new and changed assets.*

NR/L2/TRK/001/C01, *Inspection and maintenance of permanent way – Geometry and gauge clearance.*

NR/L2/TRK/2102, *Design and Construction of Track.*

NR/L2/TRK/3100, *Telecommunications Maintenance Testing & Failure Investigation.*

Surveying for Engineers. Uren, J. and Price, W.F. 5th edition 2010. ISBN 9780230221574

GPS Guidelines for the use of GPS in Surveying and mapping. Published by RICS Books 2010.

RICS Guidance Note, Network RTK Best Practice Guide and Guidelines for the use of Network RTK GPS in Land surveys. Newcastle University in association with the Survey Association, Ordnance Survey, Leica Geosystems, Trimble and RICS.

Virtually Real: Terrestrial laser scanning: RICS Geomatics client guide series

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1 Purpose

This Guidance Note standard has been written to enable a general understanding of the techniques that may be used to collect survey data for Topographic surveys requiring a positional accuracy of better than $\pm 100\text{mm}$. It provides detailed project processes for track monitoring and surveying for slab tracking as Appendices.

Such surveys are not an automatic requirement for all projects.

It specifically excludes ground investigation, geotechnical, building condition, dilapidation surveys and surveys associated with Network Rail owned and operated measurement trains.

It is intended to be used in conjunction with the suite of Network Rail Topographic surveying standards. Parts of NR/L2/TRK/3100, *Topographic, engineering, land and measured building surveying – Surveying and general* specify information to enable a specification for survey data collection to be developed for the disciplines of Track (specified in NR/L3/TRK/3101), Civils (specified in NR/L2/TRK/3102, in development), OLE, Signalling and Commercial Property (all with standards to be developed).

This Guidance Note provides guidance and recommendations on the GRIP stages that the various techniques are best suited for.

2 Scope

This Guidance Note provides guidance on surveying activity for which a co-ordinated engineering, land, topographic or measured building survey is required.

The concept of a survey risk register is covered in NR/L2/TRK/3100.

NOTE 1 It is important to consider the accuracy of the survey required and also the commercial and technical risks involved with a survey.

NOTE 2 As an example consider a track renewal site where a decision has to be made as to which technique is appropriate. This very much depends on the type of route. In general terms, a decision has to be made between the options of a Hallade survey (NR/L3/TRK/3101 Section 5.11) or Total Station (electronic theodolite and EDM) detail survey (NR/L3/TRK/3101 Section 5.8). All planned works in the area need to be considered (track and other disciplines). Where adjacent tracks (alongside or close by on the same track) are to be renewed a topographic survey would be advantageous. When other works such as level crossing renewals, platform reconstructions, or certain types of drainage work, are required, then a topographic survey has distinct advantages.

When new and innovative techniques not covered by this standard are developed and are appropriate for use, these techniques can be presented to the Network Rail Senior Survey Engineer for consideration and approval.

3 Definitions

For the purposes of this standard, the terms and definitions in NR/L2/TRK/3100 apply.

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4 Surveying and GRIP

Table 1 gives guidance on the types of survey techniques and where these techniques are best suited for use within the GRIP process. Techniques may be used in a GRIP stage other than that shown in Table 1 when this meets the project needs in a more appropriate manner, with acceptance of the CSM.

Survey	GRIP Stage							
	1	2	3	4	5	6	7	8
	Output definition	Pre-feasibility	Option selection	Single Option development	Detailed design	Construction testing and commissioning	Hand back	Project closeout
Survey project strategy*		✓						
Aerial photography			✓	✓				
LiDAR			✓	✓				
Terrestrial Laser Scanning			✓	✓	✓		✓	
Ground land and topographic survey			✓	✓	✓	✓	✓	
Geographic Information Systems							✓	✓

*NOTE *As defined in NRL/2/TRK/3100 Section 5.1*

Table 1 – Surveying and GRIP

The needs at the earlier stages of the GRIP process (3 and 4) and the accuracies required are better suited to remote sensing techniques such as aerial photography and LiDAR. Other aerial mounted sensors may also be used to provide data such as infra-red imagery (vegetation management, heat loss uses).

Accuracies needed for more detail at GRIP stages 3, 4 and 5 are more suited to terrestrial laser scanning or ground topographic survey.

NOTE Terrestrial laser scanning cannot be used for survey of rails for alignment design or platform edges unless special targets are used as shown in NRL/3/TRK/3101 Figure 12.

At GRIP stage 6, during implementation and commissioning, setting out using ground survey techniques is appropriate.

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At scheme handback and project closeout, data is required for computerised information systems such as GIS and asset information systems.

5 General

A phase often used in surveying is to work from the “whole to the part”. In practice this means that surveys are conducted in two parts: *control survey* and *detail survey*.

The control survey is a framework of ground markers that are surveyed to high accuracy to find their location on the survey grid.

During the detail survey, instruments are established over the ground markers and the features that we want to record are measured. The detail survey is carried out to a lower accuracy than the control survey and adjusted to fit the survey control framework. The detail survey also has a role in detecting gross errors in the survey control.

This same idea of working from the “whole to the part” applies with surveying using other methods such as the Omnicom 3D surveyor train mounted video system. In this case, the video is the detail survey and the survey control comes from the GNSS and inertial measurements that are used to determine the location of the train.

As the survey PGM network is often not required by the engineer for their work, its purpose is often not understood. It should be considered to be like the foundations of a bridge: if it is ignored, problems will follow.

Provision of a witness diagram (an example is given in NR/L2/TRL/3100 Appendix C) should mean that the PGM can be found again and used for further work without needing to re-observe the whole framework.

For example, additions might be required to an original survey, or a design based on this survey might need to be marked out on the ground for construction. However, extending a survey PGM network because a new adjacent area is now required might cause errors when a small site is extended to cover a much larger area. The only solution in this case is to re-compute the new network and adjust the original survey to fit the new framework.

6 GNSS surveys

6.1 Introduction

This guidance details GNSS uses for high precision survey PGM control networks but different types of GNSS techniques may be used for the collection of Asset Information for input into a GIS.

Several different methodologies are in regular use but only the Static and Rapid static concepts are explained.

For PGM control surveys, ETRF89 co-ordinates are required. These are then capable of being converted into the project’s chosen survey grid.

A typical dual frequency GNSS receiver, capable of receiving signals from GPS, Glonass or Galileo is shown as Figure 1.

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Figure 1 – Dual frequency GNSS receiver

6.2 General

GNSS survey “Good practice” should be adhered to as detailed in the following documents:

- GPS Guidelines for the use of GPS in Surveying and mapping*. Published by RICS Books 2010.
- RICS Guidance Note, Network RTK Best Practice Guide and Guidelines for the use of Network RTK GPS in Land surveys*.
- Guidelines for the use of Network RTK GPS in Land surveys*, Newcastle University in association with the Survey Association (TSA), Ordnance Survey, Leica Geosystems, Trimble and RICS.
- Network RTK Best Practice Guide*.

The last two documents available for download from: www.tsa.org.uk.

Surveying using GNSS methods is best visualised by the measurement of baselines of several kilometres in length to make up a survey framework.

The accuracy of these baselines is given below for “Static” observation techniques. *GPS Guidelines for the use of GPS in Surveying and mapping* gives more information on the expected accuracies.

Table 2 shows the hierarchy of PGMs from Primary to Tertiary and information about their layout and expected accuracies.

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PGM hierarchy type	Longitudinal spacing	Plan accuracy	Maximum error over 200 m
Primary (First order)	5 to 10 km Pair >400 m	1 in 75,000 (13.3 mm per km).	±2 mm
Secondary (Second order)	1 to 2 km Pair >400 m	1 in 50,000 (20 mm per km).	±3 mm
Tertiary (Third order)	200 m	1/50 000 (20 mm per km).	±3 mm

Table 2 – PGM hierarchy

Some additional general points are highlighted as follows.

- a) Dual frequency GNSS receivers with choke ring antenna, for reduction of multipath, capable of receiving signals from GPS, Glonass or Galileo satellites, should be used.
- b) Static or fast static GNSS survey techniques should be utilised at Primary PGMs (First order). Taking note of the PGM hierarchy, these PGMs will form a properly surveyed network working from “the whole to the part”, i.e. starting from the Primary PGMs and creating Secondary PGMs (Second order) between them that keep the primary as fixed in the calculations. Tertiary PGMs (Third order) are created between the Second order, keeping both the First order and Second order fixed in the calculations.
- c) Once the 10 km primary network has been established, infilling of the secondary network every 2 km should take place, with four pairs of Second order PGMs between the 10 km pairs.
- d) When new PGMs are selected they should be selected in a GNSS friendly location, where there exists a clear view of the sky to minimise cycle slips, multipath and other interference.
- e) The most frequent source of error when observing GNSS baselines are incorrect measurement of the instrument height. Consequently, the survey contractor should use at least two different tripod set-ups at each PGM. Further checks of the relative levels of GNSS heightened PGMs should be incorporated in the surveying system. A suitable approach to be adopted is given in Clause 6.7.
- f) The PGMs should also be linked to the OS Active control stations (OS Net).

In addition to plan position, the heights are also derived from GNSS that are two to three times less accurate than that of plan co-ordinates. Height is as important and typically has to be more accurate than plan. So, although the absolute accuracy of GNSS heighting is high – each point may have a standard error of (say) 20 mm – the relative accuracy between nearby points is low.

Additionally, when using heights from GNSS, we require heights as defined by a spirit level (Spheroid) rather than heights above the mathematically defined surface,

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the geoid measured by GNSS. The differences between the spheroid and the geoid are given as a geoid model, a “grid” of latitude and longitude.

In Britain, in simplified terms, GNSS heights are converted to heights about the height datum (Newlyn) using the Ordnance Survey’s correction surface – OSGM02.

All control surveys in GNSS will be recorded in raw data and post-processed to include reference stations and precise ephemeris downloads. Active Net GNSS surveying where appropriate for detail surveying of Band 3 (± 25 mm) or lower accuracy features should include tie readings to control PGMs established by higher accuracy GNSS techniques.

6.3 Methodology

Before commencing the observations, the contractor’s lead field surveyor for the project should carry out a reconnaissance, accompanied by the Client’s Survey Manager (CSM).

Almanacs will be investigated for satellite coverage for each field survey day, to avoid periods of poor configuration. Any periods where GDOP is greater than a value of 8 will be omitted from the processing.

For every set up, a booking sheet to record antenna heights at the start and at the end of observations should be used.

To observe a PGM control network, three or more GNSS receivers will be used simultaneously, occupying at least three PGMs at any one time, using baselines of between 20 min and one hour depending on satellite coverage.

By holding one station fixed and moving the two other GNSS units on to the next two stations, a traverse run will effectively be observed with GNSS static observations, with each control point fixed relative to at least two others in the network. The primary PGMs should be occupied at least twice, once tying in the primary points only, the second run when including measurements to the Second order PGMs.

The survey contractor should provide a detailed methodology that will detail the manner of moving receivers around the network to achieve accurate results – synchronous shifting, leap-frogging, etc., for acceptance by the CSM prior to commencement of the site work.

Although all heights will be deduced from GNSS observations, so that the relative accuracies between PGMs are consistent, it is necessary to spirit level between PGMs to obtain level values that may be utilised for engineering construction.

6.4 Computations

All GNSS survey data will be downloaded to office computers.

Survey data processing should be carried out using the manufacturer’s approved processing software.

Prior to processing the control network, antenna heights should be checked twice, once on import and once during set up of processing parameters. As much data as possible collected for each PGM should be used in the processing to maximize the

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reliability. Where bad data through poor satellite coverage was encountered, the information should be removed from the adjustment. This is achieved by editing the receiver timelines.

All processing should be carried out in the ETRF89 co-ordinates. Primary points should also be linked to downloaded OS Active Rinex data, which gives a gross error check of the network.

Baselines should be processed and checked, loop closures should be computed and a network adjustment (using "least squares methods") should be performed.

The maximum error ellipse at 95% should not exceed 15 mm throughout the primary control network. This gives a measure of the accuracy of the work.

Co-ordinates should be output in the ETRF89 grid along with their precision values.

These ETRF89 values in Latitude and Longitude may now be converted into the project survey grid.

Further discussion on the intricacies of GNSS computation are beyond the scope of this document and reference should be made to *GPS Guidelines for the use of GPS in surveying and mapping*.

6.5 Deliverables

The survey report should include:

- a) an introduction;
- b) a list of equipment including makes of hardware and software, versions of firmware, etc.
- c) a method statement describing the survey technique used including the computations;
- d) a network diagram (*.dgn format);
- e) quality reports giving how many times the network has been adjusted to derive the final results;
- f) a GNSS network adjustment that provides the error ellipses for each point, and the residuals of each baseline;
- g) final listings of co-ordinates (also provide as *Excel* spreadsheet) in ETRF89 compared with any previous supplied ETRF89 values;
- h) the control station witness diagrams for new PGMs using the form NR/L2/TRK/3100/PGM witness diagram in *.pdf format.

6.6 Levelling

6.6.1 Introduction

The vertical is the direction which a plumb bob takes when it hangs freely, under the effect of gravity. A horizontal surface is a surface at right angles to it. The height of a point is defined as the distance up or down the vertical through a point associated with a reference horizontal surface or datum.

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The datum used is related to a point known as a “Bench mark” (BM). This may have a simple local value or be related to the Ordnance Survey Newlyn datum (OSBM) from which all heights in the UK are defined. Figure 2 shows a typical OSBM.



Figure 2 – OSBM

The network of OSBM have now been superseded by the use of GNSS levels (see 6.7).

At its simplest, levelling is measuring between the ground and a horizontal plane established by a spirit level or more, usually by a survey instrument known as a level. In addition to the level, a tripod to place the level on and a measuring staff is needed.

6.6.2 Instrumentation

Levels fall into several categories.

- a) the Tilting or “Dumpy” level where before each reading a spirit level bubble has to be adjusted (now generally superseded);
- b) an automatic level (like an Optic in use for track levelling) that has a suspended compensator to define the horizontal plain. The tolerances of the compensator should be satisfied when the circular bubble on the automatic level is centralised. The compensator is a free hanging prism within the telescope, of both the automatic and digital levels which creates the necessary refraction so that the line of sight of is horizontal. When used near heavy machinery, the compensator can vibrate making readings impossible. Most instruments available today fall into the latter category and are classed as “Engineer’s levels”. There is also a variation on the Engineer’s level called a “Digital level”. These instruments may be further split up depending on the accuracy needs. Engineers’ levels are usually read to 1 mm with an accuracy

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of 1 to 2 mm. A more accurate and precise level is also available that may be read to an accuracy of 0.1 mm and estimated to 0.001 mm;

- c) laser levels where a plane is defined by a rotating laser and a sensor is used on a measuring staff to provide a reading.

Conventional Engineer's levels are used to sight directly upon an E-type levelling staff and the readings are read off (as shown in Figure 5) by the observer which allows direct measurement to the order of 10 mm with estimation to the order of 1 mm. Examples of the different types of levels are shown in Figure 3.



Figure 3 – Examples of Engineer's levels

A digital level reads a bar-coded levelling staff (as shown in Figure 4). It is sighted onto the staff and focussed then, with the press of a button, it observes the bar-code in the field of view and calculates the observed measurement. This is stored in the on-the instrument's memory for download at a later time. If a value for the start BM is known, a level may be computed on the instrument. This has the huge advantage of removing booking error with the disadvantages of the extra costs associated with buying a digital level and the reliance on batteries. Some digital levels do not function correctly in low light levels or in darkness.

Figure 4 gives an example of a digital level and the associated bar-coded levelling staff alongside a conventional "E" reading staff, that the operator has to read and also estimate to get the mm value.

Figure 5 shows the view through the level telescope of a conventional levelling staff and the reading estimated to 1mm.

Figure 6 shows a range of different cross-hair arrangements for different level instruments.

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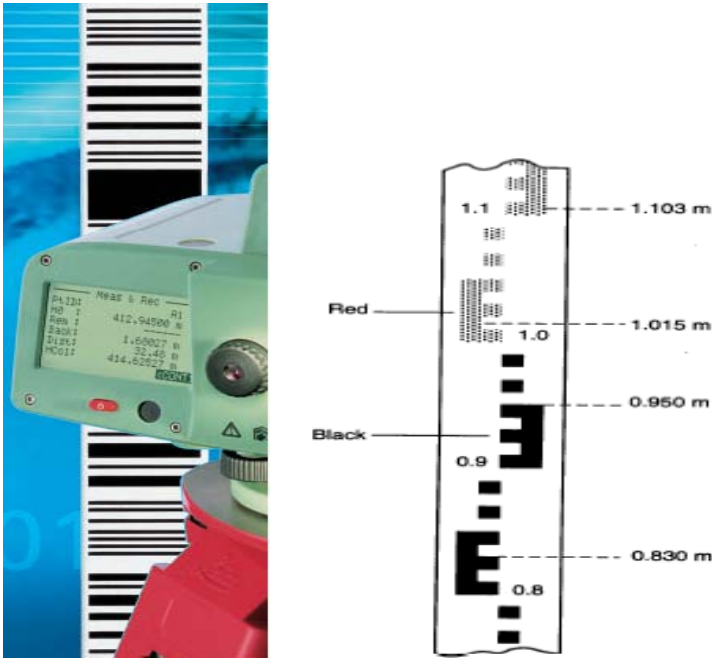


Figure 4 – Example of digital level and bar-coded staff with conventional “E”staff

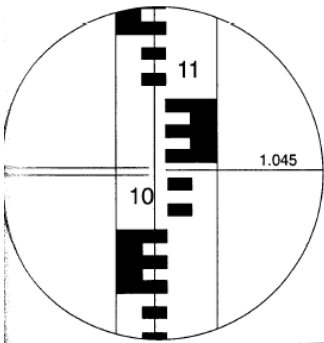


Figure 5 – Conventional levelling staff and view through the instrument

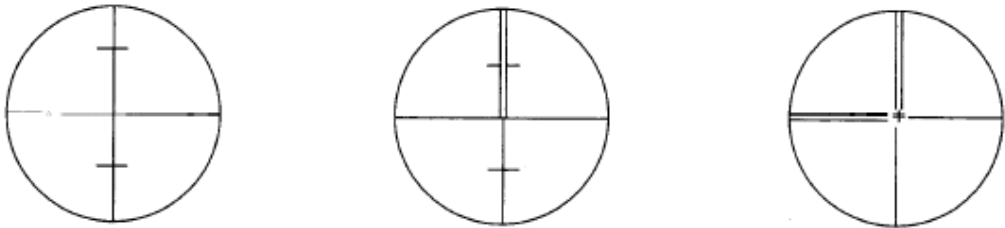


Figure 6 – Variations on cross-hairs as seen through an instrument

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In Figure 6, the two small “cross-hairs” are known as “Stadia hairs”. If readings of the staff for each are taken, the lower subtracted from the higher and multiplied by 100, the distance to the staff will be known to within 100 mm. It is a good way of estimating distances.

Table 3 gives an indication of the accuracy that may be achieved using various configurations of equipment.

Equipment	Maximum sight distance to staff	Maximum misclosure
Geodetic level and invar staves (stabilised)	60 m	$2 \text{ mm} \times \sqrt{k}$
Digital level with barcode staves	80 m	$5 \text{ mm} \times \sqrt{k}$
Automatic optical level with E-pattern staves	90 m	$15 \text{ mm} \times \sqrt{k}$
Key k is the distance levelled in kilometres.		

Table 3 –Accuracy of levelling equipment

6.6.3 Site procedure for levelling

Levelling should always use a closed loop, starting and finishing on a benchmark or known PGM. Any misclosure gives an indication of the quality of the levelling.

Benchmarks and PGMs should be at a stable location where the level will not change. Where permanent benchmarks are not present a bolt on the base of a stanchion or signal is particularly suitable for a temporary benchmark (TBM) but a cable route troughing lid or catch pit corner are not suitable.

The levelling instrument should be set up between two benchmarks in a “place of safety” that has clear sighting to both benchmarks. The maximum distance between benchmarks should be 100 to 120 m meaning that sighting with the levelling instrument's telescope would be around 50 to 60 m. A levelling run between two benchmarks should always be done twice (double levelling) to so that no gross errors in staff readings occur. In practice on site it is often possible to observe both forward (foresight – FS) and back (backsight – BS) readings twice by resetting the instrument after the first set of readings saving time.

For further information on site operation for levelling, reference should be made to a good surveying textbook such as *Surveying for Engineers* by J.Uren and W.F.Price.

6.6.4 Calculating reduced levels

There are two well known methods for calculating reduced levels from site measurements. The “rise and fall” method and the “height of collimation” (HofC) method.

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Use the “rise and fall” method of computation when control levelling, as this shows arithmetic errors. The “height of collimation” method should only be used when taking many intermediate observations and particularly on building sites for setting out.

The plane is called the height of collimation (HofC) and is defined by the height at which the telescope of the levelling instrument is set up.

The rise and fall method has the advantage of being able to compute the differences in height as you work and allows direct comparison with the second set of readings when double levelling.

Figure 7 shows an example of a site with three TBMs and two locations where an automatic level was set up. Table 4 shows what was written in the survey book.

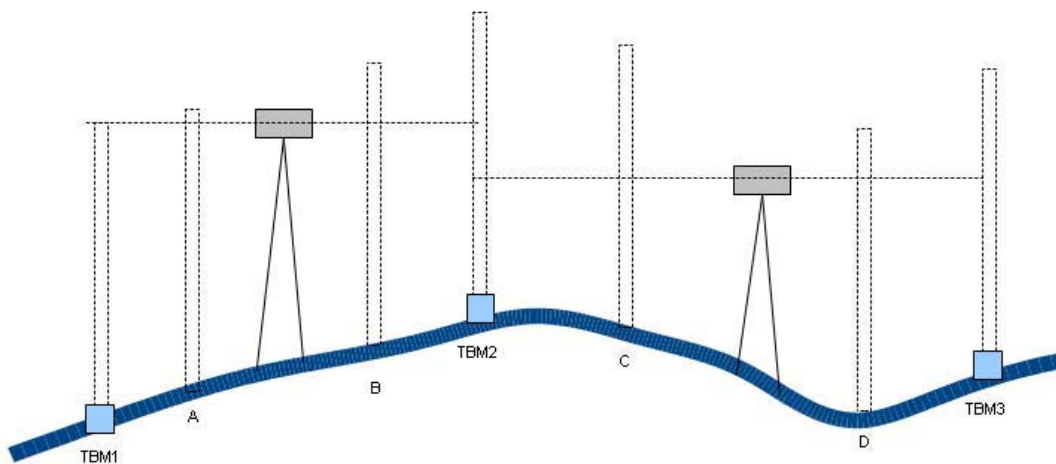


Figure 7 – Site Example with three TBMs and two level set up locations

BS	IS	FS	HofC	RL	Remarks
2.900					TBM1, Bolt at base of signal SN71
	2.401				A
	1.980				B
0.747		1.545			TBM2, Corner of LOC cabinet base
	1.228				C
	2.156				D
		1.896			TBM3, Corner of access steps

Table 4 – Survey book example

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6.6.5 H of C method

The height of collimation for each set up is the horizontal plane that is viewed through the telescope. This is shown in Figure 7 by the dotted line passing horizontally through the levelling instrument's telescope.

As in this example, we don't know the height of any of the TBMs and we are only interested in the relative heights throughout the site, we can assign an arbitrary height of 10.000 metres to TBM1. As the staff reading between TBM1 and the height of collimation is our backsight, adding the backsight value to TBM1's reduced level value will give us the height of collimation for this initial set up.

$$TBM \text{ Reduced Level} + \text{Backsight to TBM} = \text{Height of Collimation} \quad \text{equation [1]}$$

In this case, we obtain a value for height of collimation of 12.900 m. This height of collimation will remain the same for this set up. For each of the intermediate sights, we have measured the staff reading between the ground and the height of collimation. Therefore, to calculate the reduced level of the ground, we subtract the intermediate sight's value from the height of collimation.

$$\text{Height of Collimation} - \text{Intermediate Sight} = \text{Reduced Level} \quad \text{equation [2]}$$

So, for point A of this survey, we should subtract 2.401 m from 12.900 m, giving a reduced level for point A of 10.499 m. We can apply the same equation to calculate the reduced level at point B.

We have taken a foresight at TBM2 from which we can calculate the reduced level at this location in much the same way as we would treat an intermediate sight.

$$\text{Height of Collimation} - \text{Foresight to TBM} = \text{TBM Reduced Level} \quad \text{equation [3]}$$

In setting up and levelling the levelling apparatus at another location, a new height of collimation will have been set and will need calculating. As we know the reduced level of TBM2 (from using equation [3]) and we have taken a backsight to TBM2, we can use equation [1] to calculate the new height of collimation. This new value is then used to calculate reduced levels for the intermediate sights and for the next TBM.

After performing these calculations, our survey book appears as shown in Table 5.

Table 5 shows calculated reduced levels for the example site shown in Figure 7.

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BS	IS	FS	HofC	RL	Remarks
2.900			12.900	10.000	TBM1, bolt at base of signal SN71
	2.401			10.499	A
	1.980			10.920	B
0.747		1.545	12.102	11.355	TBM2, corner of LOC cabinet base
	1.228			10.874	C
	2.156			9.946	D
		1.896		10.206	TBM3, corner of access steps
3.647		3.441	0.206	0.206	Check calculations

Table 5 – Calculated reduced levels for the example site

Checks on the calculations may be done by subtracting the sum of the Foresights (FS) from the sum of the Backsights (BS) and comparing this value with difference in Reduced level (RL) between start RL and final RL as shown in Table 5 in *italics*.

It is not possible to work out the misclosure error in this case as the true heights of the TBMs are not known. However, by repeating the observations (flybacks or double levelling) a misclosure may be obtained. In a situation where the benchmarks had known heights the misclosure could be calculated from comparing actual and calculate reduced levels.

The backsight and foresight values read on the flybacks and calculated heights of collimation and reduced levels are shown in Table 6.

BS	FS	HofC	RL	Remarks
1.865		12.071	10.206	TBM3
1.627	0.716	12.982	11.355	TBM2
	2.984		9.998	TBM1

Table 6 – Readings taken during a fly back run and the calculations

There is a misclosure of –2 mm as the calculation of the reduced level of TBM1 for the flyback is 2 mm less the originally assigned reduced level. To ascertain if this is an acceptable Misclosure, equation [4] should be used.

Comparison of the reduced levels of the TBMs may be obtained and normal practice is compute mean results.

$$\text{Acceptable misclosure} = \pm 3\sqrt{n} \text{ mm} \quad \text{equation [4]}$$

where

n is the number of set ups.

As this site involved two set ups, the allowable misclosure is $\pm 4\text{mm}$ so the survey is acceptable. Due to the misclosure, an adjustment should be made to the reduced

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levels originally calculated. This is applied by making an equal and cumulative adjustment to each set up. In this case a –2 mm misclosure of two set ups can be rectified by adding 1 mm to each set up as follows.

Table 7 shows the adjustments necessary to correct for the misclosure and the adjusted reduced levels for this example site.

Initial RL	Adjustment	Adj RL	Remarks
10.000		10.000	TBM1
10.499	+0.001	10.500	A
10.920	+0.001	10.921	B
11.355	+0.001	11.356	TBM2
10.874	+0.002	10.876	C
9.946	+0.002	9.948	D
10.206	+0.002	10.208	TBM3

Table 7 – Adjustments to correct for misclosure and final adjusted reduced levels

6.6.6 “Rise and fall” method

Date 26/07/92		Levels		taken for Road Alignment			
From Chn. 2040 at Bridge N°6				To Chn 3040 at Bridge N°8			
Back sight	Inter-mediate	Fore sight	Rise	Fall	Reduced level	Distance	Remarks
2.856					35.688	0m	TBM on Bridge Abut. (N°6)
	1.432		1.424		37.112	20m	Chn. 2060 m
		3.543		2.111	35.001	30m	C.P. at Chn. 2070 m

Figure 8 – “Rise and fall” example

In Figure 8, the intermediate sighting is subtracted from the backsight and the resultant rise is written in the rise column on the same line as the observation.

$$2.856 - 1.432 = 1.424$$

In the next line, the intermediate sight is subtracted from the foresight to arrive at a fall of 2.111. It is written in the fall column on the same line as the observation.

To derive the reduced level, the TBM value of 35.688 has first the rise of 1.424 added to it to arrive at 37.112 and then the fall of 2.111 taken from that value to get 35.001.

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The advantage of the “rise and fall” method is that checks are easily obtained of the calculation by summing all the rises and all the falls and taking one from the other. This value should then equal the final reduced level taken away from the start reduced level. This is also shown in Table 5.

6.6.7 Errors with levelling and ways to avoid them

The collimation error of the instrument can be checked for using the two-peg test (see 6.6.8).

Parallax of the instrument should be checked for and eliminated whenever a staff reading is taken. This is achieved by unfocusing the instrument telescope and focusing the eyepiece so that cross-hairs are sharply focused before refocusing the main telescope.

The staff itself can introduce errors if the writing on it is worn, or the sections in a multi-section staff do not fit together well, or the foot of the staff is damaged. It is very important that the operator on the staff check that when putting up an extra section of the staff that the staff goes up fully and locks into position.

If the staff is not vertical, the observer will not be able to take an accurate reading from it. In this case, rocking the staff to and fro or using a specially designed spirit level bubble is necessary.

The level of some surfaces will change depending upon weather conditions. For example, a tarmac surface will expand in heat and muddy ground, when wet, can encourage the tripod feet to sink into it. This effect can be minimised by treading the tripod legs well into the surface and leaving a minimum of time between setting up the apparatus and taking readings.

Errors in recording readings can be avoided by sticking to a rigorous double checking procedure. Using a digital level will remove this common source of error.

Although errors in TBMs or change points (CP) are given by the misclosure of the levelling, intermediate sights are not checked. If they are observed using a digital level, the chances of an error are small.

6.6.8 Collimation error and the two-peg test

Under use and transportation, the parts of a level are usually disturbed. For instance, the compensator or the cross-hairs moving out of alignment might lead to the line of collimation not being truly horizontal. Many levels have a collimation error associated with them, which should be tested so that it is within an acceptable tolerance.

The collimation error is found by carrying out a two-peg test. The two-peg test should be done at least weekly with the results recorded, or more frequently when the accuracy of the results is imperative.

A two-peg test should be carried out on fairly level ground. First, two points, *A* and *B*, should be defined a known distance, *L* (metres) apart. *L* should be at least 40 m. These points can be defined either by using pegs in soft ground or paint on hard ground. Another point, *C*, is defined exactly halfway between these points and the

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levelling instrument is set up at this point. Pacing out these distances is accurate enough.

The levelling staff is placed at each location and the readings are recorded. The reading at B is S_1 and the reading at A is S_2 .

The level is set up at a new point, D , that is $L/10$ (metres) away from point B , as shown in Figure 9. The levelling staff is placed at points A and B and the readings recorded. This time, the reading at B is S_3 and the reading at A is S_4 .

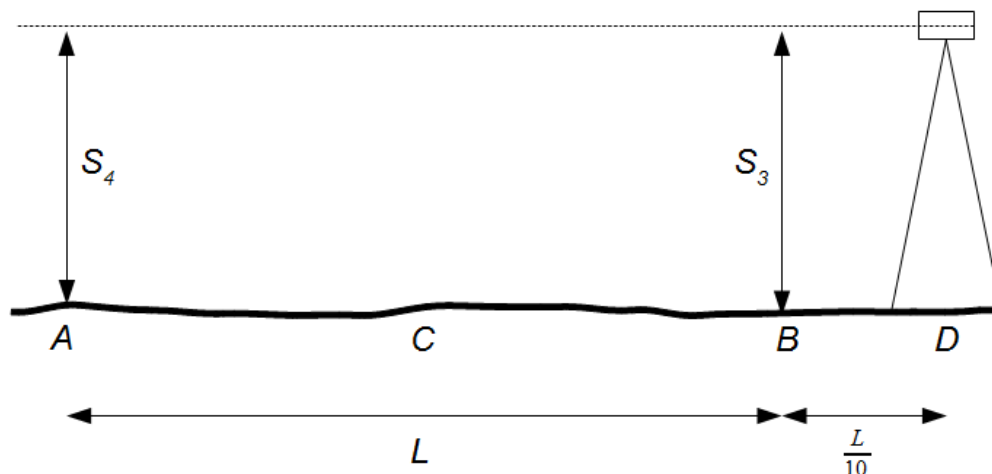


Figure 9 – Two-peg test

Using the values for S_1 , S_2 , S_3 and S_4 , the collimation error, e , can be calculated using equation [5]

$$\text{Collimation error } e = (S_1 - S_2) - (S_3 - S_4) \text{ per } L \text{ metres} \quad \text{equation [5]}$$

If the collimation error is found to be less than ± 1 mm per 20 m, the instrument is accepted as being in tolerance. If the collimation error is greater than this, then the instrument should be calibrated by a trained technician under laboratory conditions.

6.6.9 Laser levelling

Laser levelling instruments are also known as rotating lasers. They generate a plane of laser light and may be supported on various mountings from tripods to column and wall brackets.

Usually they are Laser classes 1 or 2 and present only a minimal hazard to the user.