SECTION 8 - LEVELLING

8.1 Introduction
Levelling in its various forms has been developed and refined to meet the differing degrees of precision called for in the many circumstances requiring some form of level information.

The requirements of an organisation, perhaps a large construction company, may demand a very high degree of precision in some particular aspects of its work, e.g. long tunnel work and the monitoring of structure movements. At the other end of the scale, is the relatively simple problem of providing levels for the base course of a brick house.

Each and all of these procedures have been recorded separately in print with varying degrees of useful theoretical and practical information, and some refinements, as with the guild of yester-year, have tended to be passed on verbally from the widely experienced to the novice. This information is nowhere available in a single document.

The guidelines and procedures set out in this Section represent the quintessence of the practical aspects of levelling for most purposes in Victoria. Due to its entirely specialised nature and limited applications, no attempt has been made to embrace first order levelling.

The term “reduced level” has a long history of accepted usage in surveying, engineering, architecture and the building industry. This terminology relates to the distance above or below a specific datum surface, and had real meaning when datums were widely scattered and not related to one another. With the advent of the Australian Height Datum there is an increasing tendency to discuss “heights” rather than “levels”, e.g. finished floor height. See AS 1100, Part 101-1984.

8.2 The Australian Height Datum - (AHD)
On 5th May 1971 the Division of National Mapping, on behalf of the National Mapping Council of Australia, carried out a simultaneous adjustment of 97,320 kilometres of two way levelling, holding mean sea level for 1966-1968 fixed at zero at thirty tide gauges around the coast of the Australian continent.

The levelling network in Tasmania was adjusted on 20th February 1979, and established the AHD (Tasmania). The network, which consists of seventy-two sections between fifty seven junction points, is based on mean sea level for 1972 fixed at zero at the tide gauges at Hobart and Burnie.

The tide gauge network used for the establishment of AHD is being maintained by the Division of National Mapping, as these recordings are expected to be the initial indicator of tectonic disturbances or variations in mean sea level.

Since the 1971 adjustment, minor additions have been made to the total network, and the present state of the network in Victoria is shown in Fig. 8.3. 

Heights and contour intervals shown on maps produced at large and medium scales by Government departments and authorities in recent years are all in terms of AHD. The Survey Coordination (Surveys) Regulations 1992 require that all altimetric surveys shall be connected either directly or indirectly to AHD.
DIVISION OF SURVEY AND MAPPING

THIRD ORDER

LEVELLING SECTIONS

AHD

MAY 1992

LEGEND

1/2
2
4
8
COMPLETED
ORDER INCOMPLETE

Fig. 8.3a
The denser spread of the basic network in Victoria, and the considerable extension and connection of permanent marks and surveys to this network, does much to facilitate the implementation of survey co-ordination, and encourage the use of AHD for all purposes.

Some indication of the availability of AHD brought about by survey co-ordination is provided in Fig. 8.4. It will be noted this information, perforce, covers only the major areas of interest of the various organisations. There is a great deal more detailed information available in local authority offices. Those intending work in any locality should consult with those responsible for the services provided in that area, e.g. roads, water, sewerage, electricity, gas etc. (See Section 3).

Work should always be based on more than one permanent mark or bench mark to guard against the occasion, when unbeknown to the surveyor, a mark may have moved, or been moved.

8.3 Other Level Datums

8.3.1 General

This Section discusses a number of level datums which existed prior to the introduction of the Australian Height Datum. In some instances, a relationship has been provided between datums, primarily to illustrate the magnitude of the differences involved.

From this, it will be evident that great care is necessary in examining and relating level information produced over a very long period.

8.3.2 Definitions and Abbreviations

The various abbreviations used in this section shall have the meanings as set out:

AHD - Australian Height Datum.
BM - bench mark. It is a permanent mark of known height above the datum surface.
CRB - Country Roads Board (now VicRoads).
Datum - the particular reference surface to which the altitude of each point is reduced. To be compatible, all levels must be referred to the one datum.
Datum For Levels - is a phrase which may also refer to the specific bench mark included in the circuit of levels, the published reduced level of which has been accepted as defining the reference surface.

DLS - Department of Lands & Survey - (now Land Victoria)
MMBW - Melbourne and Metropolitan Board of Works (now disaggregated)
LWW - Low Water Williamstown.
PMA - Port of Melbourne Authority. (now Victorian Channels Authority)

RL - Reduced Level. This is the distance a point lies above or below a specific datum surface.
SRWSC - State Rivers & Water Supply Commission (now disaggregated).

Note: In this section, in particular, it is important to retain the names of the organisations as they existed during the period under discussion. This will assist in simplifying recognition and relationship of older work which exists as record and archival material under these names.

8.3.3 Some Common Datums

A datum for levels may be found variously referred to as:

- Admiralty Chart Datum
- Australian Height Datum
- Chelsea Sewerage Authority Datum
- CRB Datum
- Harbour Trust Datum
- MMBW Datum
- Railway Datum
- State Datum
- State Rivers Datum and many more.

Reference may be made also to:

- Arbitrary Datum
- Local Datum

In Victoria, many of the early datums were based on determinations of low water spring tides made by Hodgkinson in 1853 and 1857, and by Capt. Cox RN in 1864, and in 1884. The 1884 figure was widely used under a variety of names. In 1971, the Australian Height Datum based on mean sea level and expressed in metres was introduced to eventually supersede all the others. However, a knowledge of the earlier datums is essential when dealing with old records or extending old work. Consequently, they will be described in some detail.

8.3.4 State Datum

Mean Low Water Spring Tides was determined at Williamstown in 1884. This was related to the "broad arrow" cut in the sill (now covered) of the Williamstown Time Ball Tower. For 87 years, this mark and its value, 7.81 feet, was recognised as the basic bench mark defining State Datum. Apart from the Melbourne Harbour Trust, all Victorian Government Departments and Authorities accepted this datum and
abandoned their earlier ones. In 1891 it was adopted by the MMBW.

Although now not used for its original purpose, the tower structure now serves as a second order trigonometric station.

As used and extended by Government Authorities this one datum was known variously as:
- Board of Works Datum
- Harbour Trust Datum
- Lands Dept. Datum
- MMBW Datum
- Public Works Dept. Datum
- State Datum etc.

While all these datums had the one origin, unavoidable errors and other mistakes were made in transferring the levels to distant areas. Thus absolute agreement could not always be expected between the bench marks and levels of the different authorities.

### 8.3.5 MMBW Datum

Prior to the introduction of Australian Height Datum and the conversion of levels to metres, the MMBW Datum as it was called, was the only one widely used in the metropolitan area. It is still needed when working with old records, or in areas where reference to imperial plans is required. This has the same origin as State Datum. Reduced levels were quoted in feet and decimals. It should be referred to as MMBW Datum (Imperial) to clearly distinguish it from AHD which is now the MMBW Datum for levels in metric units. Over the years, some 3000 bench marks were established to the old datum throughout the metropolitan area.

This datum became widely used by municipal councils and other authorities for their works and by private surveyors as datum for levels on plans of subdivisions, and for other surveys.

**WARNING:** Levels purporting to be on this datum should always be verified as many were deduced from contours or surface levels without a bench mark check.

### 8.3.6 Harbour Trust Datum

The Port of Melbourne Authority (formerly Melbourne Harbour Trust) always used State Datum for levels on land as distinct from Admiralty Chart Datum used for tides and soundings. Prior to 1970 these levels were expressed in feet and decimals and thus were compatible with levels on MMBW Datum (Imperial). On conversion to metric units, the same State Datum was retained by the Port of Melbourne Authority; whereas, in MMBW practice, all metric levels are based on AHD.

**WARNING:** When using Port of Melbourne Authority records, or connecting to their works, special care must be taken.

PMA levels in metres are above Low Water Williamstown.

MMBW levels in metres always relate to AHD. (Mean Sea Level)

MMBW levels, in feet always refer to Low Water Williamstown.

### 8.3.7 Admiralty Chart Datum - Port of Melbourne

This was based on Low Water Williamstown as determined by Capt. Cox, RN, in 1864 (as opposed to the 1884 value adopted as State Datum). It was used by the Melbourne Harbour Trust for hydrographic work in Port Phillip Bay. Until 1970, RL 0.00 on Admiralty Chart Datum equalled RL 0.19 on State Datum (Imperial measurements). Coinciding with the conversion to metric units, there was a small shift to make Chart Datum coincide with State Datum and MMBW Datum (Imperial). This shift is insignificant when referring to soundings and tide levels, and the Port of Melbourne Authority makes no distinction between the old and the new Admiralty Chart Datum.

From all RLs on the new Admiralty Chart Datum, 0.524 metres must be subtracted to bring them onto AHD. This also applies to tidal predictions and records for the Melbourne area published in the Port of Melbourne Handbook. The following information deduced from the 1978 edition has been referred to AHD.

<table>
<thead>
<tr>
<th>At Williamstown</th>
<th>RL in metres AHD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean High Water</td>
<td>0.30</td>
</tr>
<tr>
<td>Mean Sea Level</td>
<td>0.04</td>
</tr>
<tr>
<td>Mean Low Water</td>
<td>minus 0.22</td>
</tr>
</tbody>
</table>

For the Year 1976

| Highest Tide on 1 Aug. 1976 | 0.99 |
| Lowest Tide on 5 Mar. 1976  | minus 0.65 |

Record Tides

| Highest ever on 1 Dec. 1934 | 1.52 |
| Lowest ever on 19 Sept. 1926 | minus 0.88 |

It is to be noted that all tidal predictions throughout Australia are now given in metres above the local chart datum. This is generally based on a local determination of low water and means there is a different datum for each port.

### 8.3.8 Chart Datums in Victoria

Chart Datum is understood to be the datum of soundings on the latest edition of the largest scale Australian or Admiralty chart of the place.

Previously Chart Datum was referred to Ordinary Low Water Spring Tide or to Indian Spring Low Water, values derived from records over some time.

Chart Datum now and in the future will refer to Lowest Astronomical Tide (LAT). This is the lowest level of tide that can be predicted over a nineteen year lunar cycle. This does not compute absolute extremes as meteorological conditions also influence tide levels and there will be some occasions when the tide level will fall below LAT, though these will be infrequent.
Victoria has 5 standard ports, Melbourne, Geelong, Portland, Port Phillip Heads and Westernport. The relationship between the Datum shown on the relevant Chart and Australian Height Datum is shown:

<table>
<thead>
<tr>
<th>Place</th>
<th>Chart</th>
<th>Datum Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Melbourne</td>
<td>AUS 154</td>
<td>Chart Datum is - (minus) 0.524m AHD</td>
</tr>
<tr>
<td>Geelong (Inner Harbour)</td>
<td>AUS 157</td>
<td>Chart Datum is - (minus) 0.580m AHD</td>
</tr>
<tr>
<td>Portland</td>
<td>AUS 140</td>
<td>Chart Datum is - (minus) 0.507m AHD</td>
</tr>
<tr>
<td>Port Phillip Heads (Pt. Lonsdale)</td>
<td>AUS 158</td>
<td>Chart Datum is - (minus) 0.860m AHD</td>
</tr>
<tr>
<td>Westernport</td>
<td>AUS 156</td>
<td>Chart Datum is - (minus) 1.600m AHD</td>
</tr>
</tbody>
</table>

All other ports in Victoria are termed secondary ports and have local datums.

### 8.3.9 Marine Datums - General Note

In hydrographic work, the Australian Height Datum is generally not used. The use of mean sea level as a datum for soundings in navigable tidal waters is not practical; minimum depth is the practical requirement. For harbour charts, a local determination of low water is invariably adopted as datum and the same datum is used by the port authority for their associated works on land.

Differences from AHD will vary from place to place; consultation with local survey staff is essential. Eleven different tide levels are named and described in Australian National Tide Tables. They can be fixed only by reference to an analysis of extensive tide gauge readings, and if required the local authorities should be consulted. Some land boundaries are defined in these and similar terms. In Victoria there are three Marine Authorities:

- Victorian Channels Authority
- Department of Natural Resources & Environment
- Parks Victoria

### 8.3.10 Railway Datum

This was carried throughout the State by successive extensions of the railway system over a period of more than 100 years. Much of this took place prior to 1884 when the Railway Department adopted State Datum and there is no knowledge as to whether old records were amended. In an operating system, accurate RLs would be relatively unimportant. Alterations to design, incomplete “as constructed” records, lengthy time intervals between extensions, and changes of personnel would all have served to allow hidden errors to be incorporated in the record of levels. Published RL’s of stations and other railway structures were accepted in good faith by generations of surveyors as the best available datum for a whole range of levelling work. In the circumstances, it is not surprising that eventually cross-country links disclosed major discrepancies. In one recorded case this amounted to fifteen feet.

### 8.3.11 State Rivers (SRWSC) Datum

The State Rivers Datum in any one area was invariably derived from an adjacent Railway Datum. With the fan-like nature of the railways radiating from Melbourne, it was inevitable that cross-checks from one railway to another via SRWSC levelling, disclosed discrepancies of varying magnitude; some disproportionately large.

As a consequence, more than 75 different datums were established, with the boundaries usually made to coincide with natural features such as rivers.

During metric conversion in the early 1970’s the various datums were adjusted to Australian Height Datum at the same time as being converted to metric. The extent of the adjustment required varied between minus 1.5 metres and plus 0.5 metres. Occasionally the adjustment necessary within a datum area varied considerably.

Some of the larger area datums were those of:

- "Calivil"- comprising much of the Goulburn Murray Irrigation District area west of the Campaspe River through to the Loddon River and beyond.
- Waranga - east of the Campaspe River to the Goulburn River.
- Kerang - north of the Calivil Datum bounded by the Murray River and reaching as far as Swan Hill.
- Murray Valley - North East of the Goulburn River to as far as Yarrawonga.
- Glenmaggie- covering a large area of Gippsland
- Coliban - between Goornong and Kyneton and west to Carisbrook.
- Donald Main - west of Charlton.

To give some idea of the earlier proliferation of datums, the comparisons made by the SRWSC in 1959 along the area covered by the River Murray Lock Survey between Echuca and Wentworth are worth noting. The values are in feet.

<table>
<thead>
<tr>
<th>Comparison of Level Datum</th>
<th>Value (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RMLS Echuca to Wentworth</td>
<td>100.00</td>
</tr>
<tr>
<td>First Mildura Irrigation Trust</td>
<td>95.40</td>
</tr>
<tr>
<td>SRWSC Redcliffs</td>
<td>95.60</td>
</tr>
<tr>
<td>Robinvile - Victorian Railways</td>
<td>95.75</td>
</tr>
<tr>
<td>Robinvile Soldier Settlement</td>
<td>96.00</td>
</tr>
<tr>
<td>NSW Water Conservation Datum Echuca</td>
<td>100.00</td>
</tr>
<tr>
<td>NSW Water Conservation Datum Robinvile</td>
<td>101.30</td>
</tr>
<tr>
<td>NSW Water Conservation Datum Mildura</td>
<td>102.00</td>
</tr>
<tr>
<td>Nyah SRWSC</td>
<td>95.77</td>
</tr>
<tr>
<td>Swan Hill SRWSC</td>
<td>95.33</td>
</tr>
<tr>
<td>Echuca (East of Campaspe) SRWSC</td>
<td>96.25</td>
</tr>
<tr>
<td>Echuca (West of Campaspe) SRWSC</td>
<td>96.85</td>
</tr>
<tr>
<td>Echuca Victorian Railways</td>
<td>95.55</td>
</tr>
</tbody>
</table>
8.3.12 CRB Datum
CRB surveys before 1974 used datums established by other agencies where these datums were available. A great deal of earlier work was done on assumed datums.

Reference should be made to the Road Construction Authority where attempts to interpret old work are being made.

Since 1974, consistency of datum has been obtained by exclusive use of AHD, derived from MMBW levelling in the Melbourne Metropolitan area or Department of Crown Lands and Survey elsewhere.

8.3.13 Chelsea Sewerage Authority Datum
This was established by private surveyors using MMBW benchmarks placed for use in the investigation and design of the South Eastern Outfall. These had been derived from the "best fit" over a number of MMBW bench marks in the Keysborough area which subsequently had their published values amended. This was further complicated by the general instability of bench marks, due to movement of poorly consolidated alluvial silts because of water table variations. The end result was a datum shift of 100 mm which was only disclosed when AHD was brought into the area.

Similar case histories could be compiled relating to datum for levels in many areas. Field checks should be arranged to reveal any undisclosed datum shift, of a major nature, or any minimal shift due to the unavoidable drift in two vintages of levels based on the same bench mark.

8.3.14 Metric Conversion & the Australian Height Datum
In general, levels in metres refer to Australian Height Datum. Exceptions are levels published by Port of Melbourne Authority which are in metres related to the old State Datum (Low Water Williamstown). This is also the local Admiralty Chart Datum for soundings and tidal predications given in metres.

WARNING. In the early stages of metric conversion some authorities (including CRB) and other organisations changed their imperial to metric units without shifting to Australian Height Datum. These facts are not well documented and caution should be exercised in using information from doubtful sources.

During metric conversion, the MMBW took the opportunity to introduce at the same time a datum change from Low Water Williamstown (1884 Datum) to Australian Height Datum. This data shift applies to all levelling values published by, or derived from, MMBW levelling network prior to 1971 and must not be applied to imperial levelling carried out by the Department of Crown Lands and Surveys.

In the area north and west of the line, the imperial levels were reduced by 1.72 feet to bring them onto the mean sea level datum before conversion to metres. Thus, (RL in feet LWW - 1.72) x 0.3048 = (RL in metres AHD)

RL 100.00 feet LWW = RL 29.955 metres AHD.

In the area south and east of the line, imperial levels have to be reduced by 1.92 before conversion to metres.

Thus, (RL in feet LWW - 1.92) x 0.3048 = 29.895 metres AHD

RL 100.00 feet LWW = RL 29.895 metres AHD.

A summary of the relationships of the various datums used in the Melbourne area for over a century, and some other pertinent data is given in Fig. 8.5.

8.3.15 Adoption of Datum
The current value of every bench mark to be used in the establishment of a datum for levelling should be verified before commencing work. Each bench mark used should be fully described in the level book with its reference number, location, current height value, date provided and authority for the value.

When an area is subject to further levelling, it is necessary to have compatibility of datum with the earlier work. If there is no means of obtaining an up-dated verified value of the originating bench mark, then connections to other bench marks in the vicinity are required to prove stability, or establish a difference to be applied to the previously used values.

In the absence of bench marks, old levels on identifiable structures or even natural surface locations must be re-observed.

If a comparison between the new and old heights discloses a datum difference, it should be recorded as follows:

e.g. “Add (or subtract) 1.105 to all heights to bring them onto AHD (or other) based on BM 6124 RL = 97.745”. 

Additionally, it can be useful to identify the successive values of the originating bench mark by calendar dates.

In the transformation of level values from one datum to another, it is essential that reference should not be made to a datum being “higher or lower” than AHD (or another).

8.3.16 Origin of Datum for Heights Shown on Plans
Every plan showing heights shall clearly state the origin for the datum of levels presented. This statement must include an adequate description of each bench mark used in the determination of the datum, and the calendar date of the value's obtained for each bench mark. This is important, as BM values may be subject to revision from time to time.

A notation of this type is shown on the example Survey and Feature Plan included in Part 1 of this Handbook.
## DATUM AND LEVELS
### AT MELBOURNE (WILLIAMSTOWN)
### (STANDARD PORT)

<table>
<thead>
<tr>
<th>AHD METRES</th>
<th>CHART DATUM METRES</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.322</td>
<td>3.846</td>
</tr>
<tr>
<td>1.520</td>
<td>2.044</td>
</tr>
<tr>
<td>0.52</td>
<td>1.04</td>
</tr>
<tr>
<td>0.42</td>
<td>0.94</td>
</tr>
<tr>
<td>0.12</td>
<td>0.64</td>
</tr>
<tr>
<td>0.000</td>
<td>0.524</td>
</tr>
<tr>
<td>-0.08</td>
<td>0.44</td>
</tr>
<tr>
<td>-0.38</td>
<td>0.14</td>
</tr>
<tr>
<td>-0.48</td>
<td>0.04</td>
</tr>
<tr>
<td>-0.524</td>
<td>0.000</td>
</tr>
<tr>
<td>-0.880</td>
<td>-0.356</td>
</tr>
</tbody>
</table>

**BM NICHE ABOVE ARROW TIME BALL TOWER**

**MMBW 6000 - DLS**

**HIGHEST RECORDED TIDE 01/12/1934**

**HIGHEST ASTRONOMICAL TIDE (H.A.T.)**

**MEAN HIGHER HIGH WATER (M.H.H.W.)**

**MEAN LOWER HIGH WATER (M.L.H.W.)**

**MEAN HIGHER LOW WATER (M.H.L.W.)**

**MEAN LOWER LOW WATER (M.L.L.W.)**

**LOWEST ASTRONOMICAL TIDE (L.A.T.)**

**FORMER M.M.B.W. DATUM (IMPERIAL)**

**LOWEST RECORDED TIDE 19/09/1926**

**NOTE:** Values shown are prepared from the best available information
8.4 Bench Marks

The quality and permanence of all levelling is heavily dependent on the location and construction of the bench marks used. Wherever possible, marks should be sited where they can reasonably be expected to be free from accidental interference. Because of the continuing problems of ground movement, and the setting of structures, careful consideration must be given to the location and type of mark to be constructed, having regard to local soil and rock characteristics, and the capital value and permanence of the project involved.

8.4.1 Bench Mark Stability

Experience and research indicate that stability of bench marks can be called into question, unless the correct type of bench mark is used having regard to a number of factors. One of the most important factors is ground movement, and this is illustrated to some extent in Fig. 8.6. From this it will be appreciated that some knowledge of soil mechanics and the local geology will assist in improving and maintaining stability of bench marks. Some general remarks follow:

Clay Soils. In clay soils a seasonal vertical movement of more than 60 mm may occur in the surface layers. Movement can usually be detected up to a depth of two metres. It is caused by the expansion and contraction of the clay due to change in moisture content and can be minimised by action to control this change.

Alluvial Sediments. In alluvial sediments of river deltas and flood plains, areas are subject to long term settlement due to compaction and the lowering of the water table. Adjacent bench marks may move by similar amounts, causing a hidden datum error. Bench marks in these areas are known to have subsided by as much as one metre.

Construction Sites. At construction sites care must be taken to establish bench marks in locations of maximum stability. Slumping adjacent to an excavation, settlement due to dewatering, and compaction and other movements caused by heavy vehicles or new structures must all be anticipated. If there is any doubt, bench marks should be established in groups of three or more about 100 metres apart for convenient monitoring to disclose and isolate any movement.

8.4.2 Bench Mark Specifications

A bench mark should be designed for the greatest stability which can be attained economically, taking into account its probable use, its required life expectancy, and the geology of its location.

The major factor affecting BM stability is the seasonal change in the moisture content of the upper 2 or 3 metres of the soil, so prime consideration must be given to minimising this change, or isolating the BM from its influence.

Various methods for bench mark construction follow, but for maximum BM stability, the methods described in (a) and (b) may be used:

(a) Rod Driven to Refusal. Special stainless steel-clad rods which can be coupled together, can be driven into the soil, close to refusal using a power hammer. The minimum depth should be three metres. The top two metres should be encased in a plastic tube, surrounded at the top by cast in situ concrete, with caulking between the rod and plastic to prevent water entry. The top of the rod should be 300 mm below the surface and protected by a metal cover over the plastic tube casing.

(b) Rod Anchored in Sub-Surface Rock. If there is solid rock close to the surface, a hole should be drilled down to and 600 mm into it. A stainless steel rod can then be grouted in at the base and protected by plastic tube and concrete at the surface. Care must be taken to ensure that an isolated boulder or “floater” is not used as an anchor.

(c) Plaque in Concrete. A bronze plaque or steel rod may be set in concrete which has been cast in situ in a hole not less than one metre deep. Precast marks are not favoured unless firmly encased by in-situ concrete.

(d) Plaque Grouted into Surface Rock. Where there is a clean outcrop of undecomposed rock, a plaque may be grouted into an appropriate size drill hole using epoxy or other cement.

(e) Marks on Structures. Buildings on deep foundations, power line transmission towers, bridge abutments, water channel regulators etc., can be used to site bench marks. Bronze plaques, mushroom head rivets or drive nails can be affixed to many of these structures.

(f) Temporary and Lower Order Bench Marks. Marks in non-stable structures, e.g., rivets in kerbs and nails in posts, are located for convenience rather than permanence. If no better sites are available it is best to place them in groups of three or more to enable some check on identity and movement. The old traditional bench cut in the root of a tree should not be used (for ecological reasons) in urban areas, but may be used in timbered rural areas.

8.4.2.1 Survey Coordination Requirements

In the Survey Co-ordination (Surveys) Regulations 1981, specifications for permanent mark construction are prescribed, and in addition, Regulation 4 (2) requires that permanent marks be capable of serving the purposes of both horizontal and vertical reference. Section 5, Survey Marks, contains some details of survey marks in general, many of which will be found to be serving as bench marks.

8.4.2.2 Examples of Bench Marks in Use

A number of bench mark designs have been in use in Victoria for a long period. The designs adopted have generally been influenced by survival factors, accessibility and the ultimate use of the level values assigned to the marks.

The best known and widely used bench marks in the Melbourne Metropolitan area are those which have been placed by the MMBW. These are illustrated in Fig. 8.7.
VERTICAL MOVEMENT OF MISCELLANEOUS SURVEY MARKS
ESTABLISHED ADJACENT TO SET 4 FLYNN
LATROBE VALLEY AREA
LID AND CAP TO BE REMOVED TO EXPOSE BENCH MARK

Fig. 8.7
It must be noted that the vertical distance between the bench mark tube and the top surface of the cover lid is NOT constant, and users must remove the lid for access to the correct BM and value.

Inside and outside the metropolitan area, a variety of bench marks can be encountered. Some of these have been in use for many years and pre-dated the Survey Co-ordination Act when first introduced in 1940.

These range from bench marks on trees, steel (star) posts with or without concrete collars, high tensile drive nails in concrete structures, deck spikes in wooden poles, steel pipes, pre-fabricated concrete marks to the more recent and elaborate marks constructed for greater stability and surface monitoring surveys.

Bench marks on trees are shown diagrammatically in Fig. 8.8 and pictorially in Fig. 8.9 (a) and Fig. 8.9 (b).

Fig. 8.10 shows a permanent mark used as a bench mark, and Fig. 8.11 depicts a “tombstone” mark formerly used by the SRWSC in the Mallee district.

For many years, the Commission used a bronze plaque set in concrete almost solely for use as bench marks. This plaque has a raised dome in the centre and is illustrated in Fig. 8.12.

Figures 8.13, 8.14, and 8.15 are designs which have been used by the State Electricity Commission of Victoria for control of construction and the very deep bench mark for monitoring ground movement in the LaTrobe Valley.

Most of the marks used by the Division of Survey and Mapping in the Victorian contribution to the National Levelling Adjustment conform with Fig. 8.16. Fig. 8.17 shows a typical installation unit.

8.4.3 Colour Coded Marker (Witness) Posts

For ease of identification of bench marks in particular, organisations maintaining extensive level networks frequently use colour coded marker posts placed conveniently near bench marks. The key is:

- Light blue - Albury-Wodonga Development Corporation
- Red orange - Dandenong Valley Authority
- Light blue & orange top - Division of Survey & Mapping
- White & orange top - Division of Forests
- Green & orange top - Gas & Fuel Corporation of Victoria
- Blue - Geelong Waterworks & Sewerage Trust
- Red & white - Melbourne & Metropolitan Board of Works
- White - Road Construction Authority
- Red - Rural Water Commission of Victoria
- Yellow - State Electricity Commission of Victoria

8.5 Procedures

8.5.1 Definitions

The subject of levelling is complicated by confusing terminology. The word Level may refer to:

- an instrument for defining a horizontal line
- the act of setting up that instrument
- the act of using that instrument to determine the relative heights of points
- the height above datum of a point so determined
Fig. 8.12

Fig. 8.13

Fig. 8.14

Fig. 8.15
the altitude of a surface having all points at the same height above the datum
its use as an identifier for a specific surface.
To avoid ambiguity, an expanded list of definitions is necessary:

Horizontal plane is a plane surface generated by a line rotating normal to the direction of gravity at a point. It is tangential to the level passing through that point.

Horizontal line means a line that lies within a horizontal plane.

Level surface means a surface which, at all places is normal to the direction of gravity. Note that this is not a plane but a curved surface.

Level line is a line that lies within a level surface.

Correction for Curvature of the Earth is a dimension equal to the perpendicular distance between a level surface and one end of a horizontal line which is tangential to the level surface at the other end. It amounts to 78 mm in one kilometre and varies as the square of the distance.

Mean Sea Level is a level surface which is the mean level of the sea derived from an analysis of tide gauge readings taken over a period of years.

The mean level of the sea varies throughout the year, and is not necessarily consistent from year to year. Variations of 0.1 metre can be expected even when based on several years observation. Mean sea level may remain as much as 0.3 m above or below the average for as long as 1 month.

Datum means the particular reference surface to which the altitude of each point is reduced. It may be a national datum, a local datum or an arbitrary datum used only for one project. The term “datum for levels” may also refer to a specific bench mark, included in a circuit of levels, the published reduced level of which has been accepted as defining the reference surface. This information must be recorded in the level book. See also Sections 8.2 and 8.3.

The reduced level (RL) of a point is its distance above or below a specific datum surface measured along the direction of gravity.

Spot level means a point on a surface (usually ground level) together with its reduced level which will be determined by field observation and marked on a plan.

Bench Mark is a stable defined point, the reduced level of which is known and can be adopted as a datum for further levelling. The abbreviation is BM, and a temporary bench mark is known as a TBM.

Change Point is a stable point on which the staff is held while the levelling instrument is transferred from one position to the next. In field notes, it is abbreviated to CP.

Line of Collimation is the line of sight of a surveying telescope as defined by the cross hairs.

Height of Collimation is the reduced level of the horizontal line of sight of a levelling instrument in adjustment and correctly set up.
Atmospheric Refraction is the bending of a ray of light as it passes through layers of air of differing densities. This and related phenomena such as “shimmer”, “mirage” and “looming” is discussed in Section 8.5.3.2.

Reciprocal Levelling is a term used in two distinctly different senses. In textbooks it refers to a special form of differential levelling where the effect of grossly unequal backsights and foresights is minimised by repeating the observation at the other end of the line. It is described in detail in Section 8.5.13.

The term is also used to describe the procedure in which a theodolite is used to observe simultaneous vertical angles at each end of a measured line and from these, calculate the difference in level. This should be referred to as Reciprocal Trigonometric Levelling and is outlined in Section 8.5.16.

8.5.2 Choice of Techniques

On a survey plan produced for use by other surveyors or in an allied discipline, the position of a point in the third or vertical dimension is depicted on a two dimensional plan by a level value reduced to an accepted datum surface.

The absolute reduced level of a point in isolation is rarely required to a high degree of accuracy. However, its precise level relative to adjacent points is frequently critical - the consequences of error in the determination and use of this information can be catastrophic.

The theory and mathematics of levelling are such that there is a tendency to underestimate the overall complexity of levelling. In practice, it is difficult to avoid errors and consistently achieve the required accuracy. Proficiency is acquired through experience, and maintained by constant attention.

It is essential to understand the various procedures available, and be aware of their limitations.

8.5.2.1 Range of Techniques

Several techniques are available for obtaining height differences. The technique chosen will depend upon the accuracy required, the topography, and the extent of the area to be covered. The techniques in outline are:

(a) Differential Levelling

Differential levelling is the measurement of a vertical distance below (or occasionally above) a horizontal line of sight defined by a surveying telescope. The difference in altitude between successive measured points and, subsequently, their reduced levels can be calculated. See Section 8.5.5.

(b) Satellite based levelling

Satellite based technology such as GPS can provide accurate spheroidal height differences between points. To reduce these spheroidal height differences to the geoid surface (AHD) however, the geoid separation at each point must be well known. The separations can be obtained from a grid of Australia wide gravity values known as AUSGEOID 98.

(c) Trigonometric Levelling

Trigonometric levelling is the method in which an observed angle of inclination and a measured slope distance or calculated horizontal distance is used to calculate the vertical distance. See Section 8.5.16.

(d) Barometric Levelling

Barometric levelling is based on the measurement of atmospheric pressure using a sensitive gauge such as an aneroid barometer. The difference in level between places at which readings are taken can then be derived. See Section 8.5.17.

(e) Photogrammetry

Photogrammetry is the method widely used for the production of maps and plans from aerial photographs. With adequate field control, derived by differential or trigonometrical levelling, contours which have a specified accuracy of half the contour interval can be produced. However, there is an upper limit of definition beyond which differential levelling methods must be used.

Levels which have been derived from photogrammetric contours must not be used for detail design purposes.

Photogrammetry is not discussed in detail in this handbook.

(e) Profile Heighting

Surface profiles can be produced using special equipment:

(i) By measuring the depth of water using “Echo Sounding” equipment on a moving boat, a bottom surface profile can be produced for a body of water;
(ii) Using airborne radar or laser equipment, a land surface profile can be produced.

These techniques have an application in some circumstances.

8.5.3 Phenomena Affecting Levelling

8.5.3.1 Atmospheric Refraction

Differential and trigonometrical levelling methods involve the passage of a ray of light through the atmosphere.

Because the density of air changes with temperature, humidity and height above sea level, the light ray will bend as it passes from one air layer to another of different density. This effect is known as refraction, and it can cause significant errors in the levels obtained. By understanding the factors which control refraction, a levelling technique which will give the required accuracy may be chosen.
8.5.3.2 Abnormal Atmospheric Conditions

Three indicators of abnormal atmospheric conditions which affect levelling are:
(a) Shimmer, which is the apparent quivering motion of an object, due to the continual movement of the interface between air layers of different densities;
(b) Mirage, which is a transposed image, resulting from the total reflection of the light rays at an air layer interface. The common ‘water on the road’ phenomenon is the reflection of the blue sky, apparently in motion due to shimmer, and
(c) Looming, which is the apparent vertical elongation of an image, caused by a mirage effect.

When any one of these indicators is present, marked deviations of the line of sight will occur, thus reducing the accuracy of the levelling work.

8.5.3.3 Other Factors

Air layers close to the ground and other surfaces are influenced by reflected heat, especially in calm, cloudless conditions. Different surfaces e.g. bare earth, rock, block paving, water etc. each have a different effect on the temperature of adjacent air layers and thus on the light rays passing through them. Cloud cover will reduce the temperature variation, and wind tends to mix the air to a uniform density. Atmospheric conditions for levelling tend to be most stable at thermal noon. This is usually one or two hours after Local Noon, before the temperature begins to drop.

8.5.3.4 Coefficient of Refraction

The value of the coefficient of refraction may vary widely. Research indicates that the commonly used correction value for curvature and refraction of 0.67m x (distance in kilometers)², based on a coefficient of refraction: k = 0.07, is generally only reached or approached on long lines of sight passing through the upper layers of the air. Values may vary widely, up to and beyond 0.5m (km)².

Refraction effects may exceed greatly those due to curvature on lines of sight near the ground, and may be positive or negative. Values for k of between +.109 and -.050 have been established for simultaneous reciprocal observations - the value of k is not necessarily the same at each end of the line.

8.5.3.5 Effect on Field Work

Field techniques may be adopted to cancel out or minimise the effect of normal refraction, but the presence of undetected abnormal refraction is always possible.

Critical observations should be made only when conditions are thought to be ideal.

While the longer sights are most influenced by refraction, it may also affect the shorter sights of differential levelling. This must be taken into consideration.

8.5.4 Arbitrary Datum

An arbitrary datum may sometimes be used as a temporary measure. Bench marks must be established and documented to enable connection to AHD at any time in the future. The reduced level chosen for the arbitrary datum should differ from AHD by several hundred metres to avoid any confusion.

8.5.4.2 Bench Mark Verification

The bench mark adopted to define the datum for a particular project must be fully described in the level book, with its registration number, type, location, published reduced level and date. The published RL must be verified by a second benchmark or another point of known RL to confirm the identification and to disclose any error or disturbance. If a high degree of compatibility with earlier levels is required the same datum benchmark must be used and verified.

If the old datum BM can no longer be found or proven, structures levelled in the earlier work must be re-observed to prove compatibility. In construction work, it is essential to ensure that the level datum is identical with that used for the design survey.

8.5.5 Differential Levelling

8.5.5.1 Basic Procedure

The fundamental differential levelling instrument is a surveying telescope with a spirit bubble tube attached. It is mounted on a tripod and can be rotated to sight in any direction. Modern instruments are self levelling with some form of pendulum prism in the optical system. However, the spirit bubble sight tube model is useful in forming an understanding of the operation and adjustment of all types of instrument.

A graduated staff is required for the vertical measurements. Observations are recorded in a standard format level book.

The staff is held vertically on a point of known RL. The instrument is set up some distance away and levelled. A reading is taken of the staff graduation intersected by the cross hair of the instrument telescope. The staff is then moved to a new position and another reading is taken. The difference between the two readings is the difference in level between the two points.

The RL of the second staff station is reduced. The instrument may then be moved to a new position and the procedure repeated.

A modern development in levelling instrumentation is the digital level with its specially coded staff. The instrument uses a compensating prism for the line of sight and has a microprocessor to carry out the image processing of the visible portion of the coded staff pattern.
Readings are stored electronically thus eliminating the booking function from the levelling operation.

For further information on digital levelling see Section 8.5.19 on page 8-37

8.5.5.2 General Considerations

In deciding on a procedure for observing and recording levels, it is essential to take into account the purpose of the survey and consequently the accuracy required. The work should be recorded so that the field methods used, the datum adopted, the accuracy of the results and the method of adjustment will be readily apparent from an examination of the field book. The number of decimal places used, and the manner of recording the levels must not give an exaggerated impression of their accuracy.

A standard method of booking and reduction should be followed in all but exceptional circumstances. A mixed number of decimals must not be used. For instance, the reading of change points to three decimals and intermediate sights to two decimals would result in RLs of intermediate staff stations being quoted to three decimals. This can be misleading unless the RLs of the intermediate stations are rounded. By determining the height difference between two bench marks, using two independent runs, an accuracy check is obtained. The nett rise or fall for each run is derived from the difference between the total backsights and total foresights. In every other case the rise and fall method of reduction with all aritmetic checks must be used. The ‘height of collimation’ method of reduction must not be used.

8.5.5.3 Accuracy

The National Mapping Council of Australia has published standard specifications and recommended practices for vertical control surveys. These surveys must conform to the following standards of accuracy:

First Order: Differential levelling. Misclosure not to exceed 4 mm/k where ‘k’ is the distance in kilometres of a circuit or between bench marks.

Second Order: Differential levelling. Misclosure not to exceed 8.4 mm/k.

Third Order: Differential levelling. Misclosure not to exceed 12 mm/k.

Fourth Order: Sufficient to control contouring of the area to be mapped either by differential, trigonometric or barometric levelling.

8.5.5.4 Levelling Practice

Differential levelling falls into six general categories:

First Order Levelling. Very little first order levelling is carried out in Victoria due to the high cost, and the fact that movement of bench marks caused by crustal instability can exceed the accuracy obtainable.

Second Order Levelling for the establishment of primary bench marks.

Third Order Levelling for the establishment of other registered bench marks.

Control Levelling to evaluate temporary bench marks for minor projects.

Millimetre Levelling for precision engineering checks.

Cenimetre Levelling for general topographic detail.

In general the procedures closely follow the National standard and are described in some detail in the following sections.

8.5.5.5 Levelling Staves

For lowest orders of precision, telescopic staves are commonly used, but folding staves may be preferred for third order precision. For the more precise work demanded in monitoring structures, deformation surveys and basic control, rigid staves of precise manufacture with invar faces are to be preferred. In confined spaces shorter ‘industrial’ staves may fulfil requirements.

As rigid and folding staves are typically of wooden construction with painted markings, care should be exercised to prevent wear or damage to the markings and support structure. Damaged wooden staves will be more prone to changes in length, as they are more sensitive to changes in humidity.

Wooden telescopic staves have been all but superseded by aluminium, PVC resin or fibreglass construction, in varying cross-section. Non-metal construction may be preferred for use in some areas on safety grounds such as near power lines. Recent trends have been towards the ‘slimline’ or ‘lightweight’ aluminium models with smaller cross-section and more leaves (sections) than the traditional model. Markings on a telescopic staff may be renewed using special plastic strips that can be obtained for the purpose but care is required not to introduce index errors during the replacement process.

Use of a cover of canvas or similar material will prolong the life of staves, particularly those being moved frequently by vehicle or other means. A number of accessories are usually available for staves such as struts, staff bubble, alternative baseplates for the staff and ground-plates. Calibration of staves is a necessity for certain classes of levelling covered by the Survey Co-ordination (Surveys) Regulations 1981 and particularly for work performed for very precise engineering requirements and basic control levelling. See Section 2.2.
Aluminium staves have a coefficient of expansion several times that of the traditional wood and this coefficient is usually supplied with new staves for use in the more precise surveys if required. Present day staff markings for surveys made in metric units have been largely influenced by guidelines published by the Surveyor-General in Survey Coordination Circular 1973/14 which state:

(a) For second and lower order of precision, the recommended graduation interval is 0.01 metre, and
(b) The European "E" type graduation, when the top of the "E" indicates the 0.01 unit, and the bottom of the "E" indicates the 0.05 unit, should be used.

In 1972 the Standards Association of Australia (SAA) produced an Australian Standard (AS) Specification for Levelling Staffs which was issued as AS 1298-1972 and subsequently reissued with one minor amendment relating to the use of the AS Mark. The scope of this standard was to "set out the requirements for figuring on levelling staffs". The style of "figuring" adopted by SAA is essentially a modified European E pattern with graduation marks in monochrome (black) and an unique system for the reading of numerals.

In 1973, the Country Roads Board, now VicRoads made a wide ranging investigation of staff face markings, and the findings emerged in favour of a modified European E pattern with red and black graduation marks and the system for reading of numerals as adopted in the British E pattern. This pattern, which is in sympathy with the recommendations of the Surveyor-General and the graduations adopted by SAA, has become known in Victoria as the CRB pattern. It is widely used in this State for levelling purposes either as the true CRB pattern or as the pattern defined by the German Industrial Standard DIN 18703.

The other popular pattern results from the large number of staves of Japanese manufacture. The Japanese 'E' pattern, another of the so-called 'open ' E' patterns, has a degree of similarity to the CRB and German patterns, with the main variance being that the bottom of the 'E' indicates the 0.01 unit. The only Australian manufacturer of levelling staves has adopted a pattern very similar to the Japanese pattern.

The unmodified European E pattern is used to a much lesser extent. Some European wooden folding staves used for higher precision utilise this system. Care must be taken when utilising different staves with these two similar different patterns particularly if one is familiar with one and observes the staff in an automatic process. Reference to whether the numerals occupy the same portion of each decimetre length as the 'E' or not will assist determination whether the staff is of Japanese or CRB/ German pattern.

Staves conforming to AS 1298 and bearing the AS Mark have not yet been manufactured.

Invar staffs used in precision surveys have no pattern, but carry 10 mm graduations divided into 5 mm sections, with a series of numerals for identification. Fig. 8.18 provides some comparative information.

Apart from the difference in graduation pattern, digital staves should be treated with the same care as any levelling staff.

The coded patterns are specific to the level manufacturer as different signal processing algorithms are used. A typical bar-code pattern is shown in Fig. 8.18a.

8.5.5.6 Staff Bubbles

There are various types of staff bubbles available to assist in keeping the staff in an upright position. Staff bubbles with provision for adjustment of the bubbles should be used for the more important work.

Staves for Digital Levelling

![Fig. 8.18a](image)
Fig. 8.18
8.5.6 Second Order Levelling

8.5.6.1 Instruments
A high quality auto-collimating level fitted with a parallel plate micrometer and non-telescopic tripod should be used. Digital levels of the appropriate model are also also suitable.

Two invar or other precision staves designated 'A' or 'B' are essential. For identification, calibration etc., they must be individually numbered. They must be calibrated initially before use and again at approximately three monthly intervals when in use. Calibration facilities are discussed in Section 2.

8.5.6.2 Setting up the Level
The eyepiece of the telescope should be adjusted for parallax of the cross hairs.

Maximum Tripod Stability should be ensured by applying downward pressure on the legs or by other action appropriate to the situation. As far as possible, avoid sun on the legs or a set-up on two different surfaces such as soil and concrete.

Compensator Checks should always be made as part of the set-up procedure of an automatic level. Extreme care must be exercised to make sure the compensator swings freely at all times.
To test the compensator, there are several options:

- While looking through the telescope, turn a foot-
screw, watch the line of sight change and then settle
back to the original line
- Apply a little pressure to or lightly tap the tripod
and observe as above
- Tap gently on the side of the level and observe
the line of sight shake then settle.

If the compensator is stuck, it might be released by a slight tap on the instrument. This release is only temporary and a level, once suspect, should not be used without workshop attention.

Instrument Stations should be numbered consecu-
tively. At odd numbered instrument stations the cir-
cular bubble should be centred with the telescope
pointing in the direction of the backsight (Staff A) and
at even stations pointing to the foresight. (Also Staff
A.) On the return flight, an even-odd sequence should
be adopted.

Prior to the first reading at any one instrument station the telescope should be rotated 360° about the vertical axis and in a clockwise direction. Subsequent point-
ings are also made by rotation in a clockwise direction.
This ensures that the compensator comes to rest the
same way each time, and that any bias which may
develop will cancel out on the equal back and fore-
sights.

8.5.6.3 Use of the Staff
The base of each staff should be inspected at every change point and cleaned if necessary.

The change point should be a metal ground plate firmly placed to avoid movement, a steel spike driven into soil, a nail in bitumen, or a similar well defined object not subject to displacement. On the change point, the staff should be held vertical by reference to the circular bubble. This should be checked by plumb bob each day for accuracy.

8.5.6.4 Collimation Check
At the start of each day the level must be checked for collimation error using the "twopg" method, and the readings recorded in the level book. This may be con-
veniently done immediately after the first set-up: while the two staff holders stand fast, the instrument
is transferred close to the foresight and the check
readings made. If the collimation errors exceed 1 in
20 000, adjustment should be made, and noted in the
level book with the subsequent re-test observations. See also 8.5.14.1

8.5.6.5 Length of Sight
The length of sight must not exceed 30 metres. In less
than perfect conditions it must be shortened suffi-
ciently to permit the reading of the staff to the required
accuracy.

Back sights and foresights should be made equal by rough chaining. This eliminates the effect of any re-
sidual collimation error, curvature of the earth and
normal atmospheric refraction. To achieve this in hilly
terrain, a zigzag course may be necessary, with the
downhill sight normal to the contour and the equal
length uphill sight on a more gentle rising grade.

Between bench marks the sum of the foresight distances
should not differ from the sum of the backsight dis-
tances by more than 15 metres. This may be achieved
by adjusting the sight lengths of the last few observ-
ations. Where this is not possible, an explanatory
note should be added to the record.

8.5.6.6 Refraction
To avoid some of the effects of abnormal refraction, the line of sight at any point must be not less than
half a metre above the ground surface. Errors due to
constant uniform refraction are cancelled out by using
equal length backsights and foresights. Discrepancies
due to uniformly changing refraction tend to be elimi-
nated by the technique of always sighting Staff "A"
first.

8.5.6.7 Procedures
(a) First Levelling Run. Both staves must be set and ready before reading commences and the time interval between readings must be kept to a mini-
mum. Staff "A" (on the BM) must always be read first and Staff "B" second.

This leap-frog system of progression ensures that the
backsight is read first from one instrument station
and the foresight first from the next. This
tends to cancel out errors due to soil rebound and
changing refraction which otherwise would be
cumulative. The same staff must be used on each
bench mark even if this means an extra change
point and instrument set-up at the end of a BM-
to-BM run. This is to eliminate the effect of index
error in the staff.
(b) Second Leveling Run The second levelling of the BM-to-BM section should proceed in the reverse direction to the first, preferably by a different survey party equipped with a second complete set of equipment.

Alternatively, the one party using the same equipment may make the reverse run with staves and staff holders interchanged (Staff “B” on the BMs). This is intended to disclose possible staff graduation errors, which could affect both runs by the same amount and produce an apparent good close. It may also prevent the staff holder from twice making the same mistake, e.g. misidentifying the bench mark. Errors and mistakes of this nature would only show up as a circuit misclose and are extremely difficult to isolate.

Ideally, the reverse run should be made at a different time of day: observing out in the morning and back in the afternoon, or out one day and back the next.

8.5.6.8 Readings
If the level is fitted with a V-slot diaphragm it should be used. The staff readings made by using a parallel plate micrometer must be to the fourth decimal of a metre. Coincidence between the diaphragm cross hair and the image of the staff graduation must be achieved by clockwise motion of the micrometer drum. This takes up slack in the linkage and ensures that any bias is constant on backsights and foresights and cancelled out.

8.5.6.9 Recordings
All recordings must be made in ink.

The initial entry must be a heading listing the job reference number, route details etc, together with the names of personnel in the party, the serial numbers of the instruments used, the staff identification numbers (with the date of calibration), and the commencing and finishing dates of the survey. See Annex 1 for an example.

Each day’s entry must show any change in the previously listed information, the observing conditions, air temperature at the beginning, midway and at the end of the days operations, and details of the collimation test. The time at the beginning and end of each run between bench marks must be noted.

The staff readings must be entered in the appropriate backsight and foresight columns with staff “A” or “B” designated in a separate column. The bench marks must be fully recorded with reference number, description and location. Change points should be briefly described.

No erasures are permitted. An incorrect entry should be struck out in such a way that it may be deciphered if necessary, and the correct entry placed adjacent to it and initialled. Where the amendment is based on check readings these must be recorded in the level book and cross referenced.

The record of each BM-to-BM run must be ruled off and separated from the next entry. If circumstances prevent the completion of a proposed BM-to-BM run, a temporary bench mark must be established for the termination of the section and the entry ruled off at that point.

Every page must be initialed and dated. The final page must carry the surveyor’s name in block letters, signature and date.

With digital levelling the staff readings are electronically recorded, however the route between successive BM’s and sketches of the individual BM’s should be drawn in a Level Book which should also record the daily collimation check.

8.5.6.10 Reduction
The BM-to-BM rise or fall must be deduced from the totals of the backsight readings and foresight readings, and checked independently by another person, using an unmarked photocopy of the original observations.

Note that “adding down the columns” produces a different sequence of figures and is a good check on “adding up”.

Change point to change point rises and falls need not be deduced except to isolate an obvious misreading disclosed at the completion of a double run over the same change points. Such computations are unnecessary as intermediate reduced levels are not required.

The BM-to-BM rise or fall deduced from the return run should be transcribed adjacent to the outward entry and the mean value calculated and clearly marked as such. Circuit misclosures may be deduced and recorded in the level book.

Reduced levels must never be entered in the level book close to the field observations. The deduction and adoption of reduced levels is a separate operation which may involve data brought in from far beyond the immediate levelling circuit. Reduced levels are normally finalized in the office in conjunction with other pertinent information.

8.5.6.11 Accuracy
The height differences obtained from two flights of levels between bench marks must not differ by more than 4 mm \( \pm \)k for 1st order or 8 mm \( \pm \)k for 2nd order accuracy as the case may be (where \( \text{"k"} \) is the distance in kilometres).

If this limit is exceeded (due to the accumulation of small errors) both runs must be repeated before an adoption is made. This is not necessary if a gross misreading has been isolated and corrected.

The word “OMIT” should be written over the struck entries but they must remain legible.

8.5.6.12 Procedure Summary
The BM-to-BM run shall be carried out as follows:

Outward run - staff sequence A - B - A - B

Staff A on both BMs

Return run - staff sequence B - A - B - A

Staff B on both BMs

In either case at every station:

Point to “A” and level up,

Rotate telescope 360°

Read “A” then read “B”.

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8.5.7 Third Order Levelling

Third order levelling is generally used for the establishment of bench marks where the allowable circuit misclosure does not exceed 12 mm \( \sqrt{K} \), ("K" being the distance in kilometres). In these circumstances all sights are made to and from change points, and no intermediate sights are taken. The staff is read to the nearest millimetre, if necessary by estimation.

When it is intended that the bench marks will be recorded and the RLs published, second order methods with two staves should be used, even though third order accuracy is acceptable. The Victorian Third Order Levelling network is made up of first order levelling effected in the early stages and of third order levelling to this standard. When the bench marks are intended for temporary or local use a "one staff" technique may be adequate.

The same order of accuracy is required for certain engineering works such as water tank or machinery foundation checks, where the tolerance of concrete or steel placement is defined by the project specification. However, in these cases, the use of intermediate sights is the most practical approach, and a somewhat different procedure is used. See section 8.5.8.

8.5.7.1 Control Levelling

If the required levels are located some distance apart, each point should be a change point in a closed circuit of levels, or BM-to-BM flight, and no intermediate sights should be observed. (An example would be the establishment of third order bench marks required for the control of lower order levelling.) Two staff equipment and procedures should be used when economically feasible, but are not essential. The following procedures are based on the use of one staff only.

8.5.7.2 Equipment

A good automatic or digital level should be used. Staves should be numbered and calibrated before use and recalibrated at approximately half yearly intervals. Staff bubbles must be checked regularly.

8.5.7.3 Collimation

A test for collimation error must be made, at the start of each day. The test reading should be recorded in the level book and adjustment made if the error exceeds 1 in 20 000. The re-test after adjustment should be recorded.

8.5.7.4 Setting Up the Level

The tripod should be set up for maximum stability and the levelling-up procedure as detailed in Section 8.5.6.2 must be adopted. The telescope should be pointed in the direction of the backsight at odd numbered instrument stations and towards the proposed foresight at the even stations.

8.5.7.5 Length of Sight

The length of sight should not exceed 60 metres. In less than perfect conditions it must be shortened to permit the reading of the staff to the required accuracy. Backsights and foresights should be made equal by the staff holder pacing from the back staff station to the instrument and the same distance forward to the next staff station. Between BMs, total length of backsights and total length of foresights should not differ by more than 50 metres. This may be achieved by adjusting the sight length of the last few observations.

8.5.7.6 Refraction

To minimise refraction error ensure that no point in the line of sight is closer to the ground than half a metre.

8.5.7.7 Procedure

The staff should be read to 0.001 m by estimation. The use of one staff only and the leapfrog mode of progression requiring the backsight to be read first from each instrument station is permissible, but the use of two staves is highly desirable. To lessen errors due to soil rebound and changing refraction, the time interval between the reading of the backsight and foresight must be minimised. If a halt is necessary, read the foresight onto a temporary bench mark. This should be described as such in the level book, but it is not necessary to leave a space in the record unless the break is overnight.

Good change points such as spikes, metal ground plates, nails in bitumen, or small pegs are essential and should be marked in a temporary manner for relocation if necessary.

8.5.7.8 Recordings

(a) The entire record must be made in ink. A heading giving all the usual information as listed for second order work should be included. The staff readings should be recorded in the appropriate columns for reduction by the “Rise and Fall” method. See Annex 2 for an example.

(b) Bench marks must be fully described, with reference number, description and location. Distinguish between old marks found and used, and new marks established. The published values of all old bench marks must be listed and the mark chosen as datum clearly indicated. Leave space to record the reduced level which will be adopted for newly established bench marks.

(c) Change points should be briefly described. A general key diagram showing the relative location of bench marks and the position and direction of connecting flights of levels is valuable. This should be included in the level book for permanent record, despite the intention to duplicate it on a work sheet. Recording procedures may be modified if computer processing of reduction and adjustment is used.

8.5.7.9 Reduction

Reduction must be by the rise and fall method only. Arithmetic checks must be applied; on each page of the record, individually total the backsights, foresights and rise and fall columns and record the results.
The difference between the sum of the backsights and the sum of the foresights should be equal to the difference between the total rise and total fall and between the first and last reduced levels.

Apart from compensating errors, this gives a complete check on the arithmetic and must be documented at the bottom of each page.

Calculators should not be used to determine RLs from rises and falls as transcription errors may occur, leaving individual RLs unchecked. This practice should be observed unless a tape of the input and output is printed, and the transcription checked by a second person.

If the bench mark to bench mark or circuit closure error is within the allowable limit of 12 mm √k it must be adjusted by an appropriate method. This adjustment must be made to obtain the best possible values for the published levels. Except in special circumstances, these should be rounded off to the nearest 0.005 m.

8.5.8 Millimetre Levelling

In engineering work the degree of accuracy required and the physical constraints of the circumstances frequently call for modified techniques.

8.5.8.1 Application and Procedure

Where a large number of levels are needed to millimetre accuracy, and are confined to a relatively small area, it is not practical to change the instrument position after every staff reading. Hence intermediate sights must be accepted, but should be checked by a duplicate set of observations from a second set-up. To preserve overall accuracy, third order guidelines (Section 8.5.7) should be used. Restrict each change point to a bench mark, either existing, or especially established and levelled independently. Prove the datum and the connection to it by commencing on one BM and closing onto another.

8.5.8.2 Reduction of Millimetre Levelling

To maintain the required accuracy, the BM-to-BM misclosure should not exceed 0.002 m and this must be proportioned out to yield acceptable reduced levels for the intermediate sights.

Where these intermediate sights are numerous it is convenient to make the adjustment to the backsight or foresight readings which have been struck out and the amended value inserted. Reductions are then completed using the rise and fall method with full arithmetic checks. See Annexes 3 and 4.

8.5.9 Centimetre Levelling

For certain types of levelling the third order maximum error of 12 mm √k is unnecessary and a figure of 30 mm √k or better is more realistic. For this, the term Centimeter, or fifth order, is proposed. This would conform to the ICSM specification of 36 mm √k.

8.5.9.1 Procedure

In centimetre levelling, where intermediate sights are read to the nearest centimetre, reduced levels must not be quoted to three decimal points. Because the reduction of the observations must include full mathematical checks, a two decimal place recording procedure must be adopted. To preserve overall accuracy, temporary bench marks can be established using third order methods. Round off their adopted RLs to two decimal places before the reduction of the lower order work commences. The field work of general levelling follows the basic rules for higher order work, with some flexibility.

8.5.9.2 Setting up the Level

(a) Check the instrument regularly for adjustment and make sure the staff is in good order;
(b) Ensure stability of instrument set-up;
(c) Position the instrument so that the backsight and foresight will be the same length, and that the intermediate sights will, as far as practical, be equal to them. Level the instrument with the telescope directed alternatively to the back change point and in the direction of the proposed foresight.

8.5.9.3 Datum

Prove the stability of the datum BM, and the accuracy of the published reduced level, as transcribed into the level book, by checking onto a second bench mark. In certain circumstances, another published level, on a solid structure, may prove the datum to the required order of accuracy.

8.5.9.4 Change Points

Sight length will depend on conditions, but generally the change points should be positioned so that backsights and foresights do not exceed 100 metres. Change points which can be located in the future should be selected.

8.5.9.5 Recording

Observe and record staff readings to two decimal places only. Backsight and foresight readings may have + or - appended to indicate more or less. This gives an effective reading of 0.003 m on change points. See Annex 5.

Record the location of intermediate staff stations by means of an alpha-numeric code on a dimensioned sketch in the level book or in a separate field book.

8.5.9.6 Reduction

Generally the + and - will tend to cancel out in the reduction, and can be ignored. However, they may be used as a logical basis for adjusting out the BM-to-BM misclosure if it falls within the allowed limits of 30 mm √k with no more than three adjustments of 0.01 m in one kilometre of levelling. To make the adjustment in the level book, strike out the original reading (but leave it legible) and insert the adjusted reading.
Amend the total backsights and total foresights and check the BM-to-BM close before deducing the rises and falls. Then total and check the rises and falls before the reduced levels are derived. The use of this method ensures that the misclosure has been evenly distributed over all readings and that the best possible values of RLs have been obtained without separate documentation.

8.5.10 Staff Stations

Particular attention must be paid to the method of measuring and recording the exact location of the staff station. If correctly observed detail levels appear on a plan in the wrong position, due to mistakes either in field recording or plotting, the level error will equal the distance error multiplied by the tangent of the ground slope angle. The error will be minimal in flat terrain but may be considerable in steep country.

8.5.10.1 Control Framework

The framework of the survey must be preserved to enable the project, which will be designed to fit on the surface depicted by the levels, to be correctly positioned. Structures must be set out from the original survey base. Failure to do this may cause serious and expensive mistakes in the siting of works.

8.5.10.2 Chainage and Offset

A standard level book makes provision for a staff station position to be recorded by means of a chainage along and an offset left or right from a base traverse. With amended headings, these columns may be used to record the co-ordinates of each point (on either standard or local grid).

The grid axes or traverse lines should be positively marked on the ground using a method appropriate to the situation and having in mind the purpose of the survey.

The accuracy of the position of spot levels on the resulting plan depends on the methods used to obtain chainage and offset of these points.

The staff should not be used for horizontal measurement as it leads to undue wear and tear.

8.5.10.3 Random Spot Levels

Random spot levels can be fixed by the bearing and the distance measured from an adjacent theodolite station. Stadia distances will yield a lower order of accuracy which may be acceptable.

A staff station position may be identified on a dimensioned sketch in a field book by a cross, and an alpha-numeric code symbol which is repeated in the level book with a full description.

The choice of method lies with the surveyor; the method used and the order of accuracy must be clearly indicated in his notes.

8.5.11 Basic Booking Procedures

The recording of staff readings must conform to certain basic principles as detailed further on. As examples of procedures to be adopted when two or more pages of levels have to be booked and reduced are not included, a complete re-statement is necessary.

8.5.11.1 Backsight

After the level has been set up, the first reading taken is known as a backsight. The staff is held on a bench mark or a change point, the level of which is known or can be deduced. The reading is always booked in the first or “back” column. On the same line in the “remarks” column is recorded a full description of the bench mark, its published reduced level, reference to datum etc., or, if appropriate, brief details of the change point.

8.5.11.2 Foresight

The last reading before the instrument is moved is taken with the staff held on a change point or bench mark. This is known as a “foresight” and is booked in the third or “fore” column with the appropriate description in the “remarks” column.

8.5.11.3 Intermediate Sights

All other staff readings are known as “intermediate sights” and are usually, but not always, booked in the second or “inter” column.

8.5.11.4 Change Points

When a series of levels is taken, with two or more instrument set-ups, the foresight from the first set-up is taken with the staff on a firm and well defined “change point” which is described, on the same line, in the “remarks” column. From the next instrument set-up the first reading is to the staff held on the same “change point” and is booked on the same line in the “back” column.

8.5.11.5 Turning the Page

Every page must commence with an entry in the “back” column only and finish (except for totals) with an entry in the “fore” column only.

When a “foresight” reading to a “change point” is recorded on the last available line on the page, the backsight to the same change point, from the next instrument set-up, is recorded in the “back” column of the first line of the next page, and has the same description in the “remarks” column. When an intermediate sight is to be recorded on the last available line on the page, it is booked in the “fore” column, and the same reading is booked in the “back” column of the first line of the next page. It is conventional to enclose readings booked in this way in brackets to indicate they are not change points. The reduced level of the last staff station on one page is duplicated on the first line of the next page. An error here can be easily overlooked, but would be detected when dosing on the next bench mark.
8.5.12 Reduction of Levels

For second order levels the method of reduction is quite simple, and is outlined in Section 8.5.6.10. For the reduction of third and fourth order levelling surveying textbooks invariably give details of two methods. THE "RISE AND FALL" METHOD MUST BE USED EXCLUSIVELY AS IT IS DEFINITELY SUPERIOR TO THE "HEIGHT OF COLLIMATION" METHOD.

8.5.12.1 Rise and Fall Method

The "rise and fall" method of reduction compares each pair of successive staff readings. The relative rise or fall is determined and applied to the RL of the first staff station to obtain the RL of the second station. The full "rise and fall" procedure gives a simple but absolute check on the arithmetic, compensating errors excepted.

8.5.12.2 Height of Collimation Method

The "height of collimation" method requires the subtraction of each staff reading from the reduced level of the line of sight of telescope. An error in arithmetic is not carried forward and so may remain undetected. This method is inferior and must not be used for reduction. There is a checking method for height of collimation reduction but it is too complicated for practical use.

8.5.12.3 Reduction Procedure

A staff reading represents the distance the staff station lies below the horizontal line of sight of the instrument. Rises or falls are deduced by taking the algebraic difference between each staff reading and the one following it, if observed from the same instrument set-up. This will have been recorded on the next line, immediately below, or below and to the right.

The rise or fall from the first to the second staff station is recorded on the same line as the second staff reading taken from the same set-up. Note that the "back" and "fore" readings on a change point line are not compared with each other.

8.5.12.4 Arithmetic Checks

To check the arithmetic of the reduction and accuracy of the entry the following steps apply:

(a) The "back", "fore", "rise" and "fall" columns on each page are individually totalled.

(b) The difference between the total of backsights and total of foresights should equal the difference between the total of rises and total of falls.

(c) After this check, the reduced levels of all the staff stations are deduced by applying the rise or fall to the reduced level of the previous station. Work successively from top to bottom of the page.

(d) Finally, the difference between the first RL and the last RL should equal the already balanced differences between total backsights and total foresights and total rises and falls. If there are no compensating errors this gives a complete check on the reduction. The column total and check differences should be recorded at the bottom of each page. If the last staff station is a bench mark, this also gives a check.

8.5.13 Reciprocal Levelling

This is the method used to maintain accuracy in a situation where the normal technique of balancing back and foresights cannot be used, e.g. the transfer of levels across a wide river.

The basic method consists of twice deducing the difference in level between change points on opposite banks and taking the mean. The instrument is set up first on one bank and then on the other. It is useful for the two change points to be substantially at the same RL. Different methods are detailed in sections 8.5.13.1 to 8.5.13.3, to highlight the sources of error and the steps taken to eliminate them. However, the preferred method (Section 8.5.13.3), using two instruments and two staves should be used in most situations.

8.5.13.1 One Level and One Staff

The steps, when using this method, are as follow:

(a) The level is set up on the left bank and the staff is held on the adjacent change point and read;

(b) The instrument stays in place on the left bank. The staff is transferred to the change point on the right bank and is read;

(c) The difference in level of these two readings is deduced. The staff stays in place on the right bank;

(d) The instrument is transferred to the right bank, set up, and the staff reading taken;

(e) The instrument stays in place on the right bank. The staff is transferred back to the left bank and is read;

(f) The difference in level of these readings is deduced;

(g) The mean of the two differences in level is adopted.

The use of this method cancels out errors due to collimation adjustment and to the curvature of the earth. It would also cancel out refraction if it remained constant, but this is unlikely due to the time factor involved in crossing and re-crossing the river. Also, in that time, errors due to soil rebound and instrument movement could arise.

Generally, this is not a satisfactory method.

8.5.13.2 One Level and Two Staves

This is the equipment available to a normal precise levelling party. The steps, when using this method, are as follow:

(a) Staff A is set on the left bank change point and Staff B on the right bank;

(b) The instrument is set up on the left bank close to Staff A. Both staves are read with the minimum time interval possible between each reading and the difference in level deduced. If possible the instrument should be set up in the projection of
the line B-A. This would enable both staves to be read without turning the telescope and disturbing the compensator;

(a) The instrument is re-set at a different height and the staves re-observed to allow another deduction of the rise or fall;
(b) The procedure is continually repeated to produce a set of ten or so differences in level. The mean is calculated and designated "mean of the first set". The staff readings should be taken to the maximum number of decimal places that the length of sight will permit. In the case of the near staff this could be one or two places more than the far staff;
(c) The instrument is transferred to the right bank. The two staves are interchanged, Staff B going to the left bank and Staff A to the right to cancel out index and possibly graduation errors. The reading process is repeated to yield a "mean of the second set";
(d) The mean of the two sets is adopted as the difference in level.

A comparison of the individual mean differences in level which comprise one set could disclose a trend, which may be due to changing refraction. If this is not present, the difference between the mean difference in level derived from the first set and that derived from the second set is due to error in collimation adjustment, staff index and graduation error, curvature and constant refraction. This is cancelled out in the final mean.

If changing refraction is suspected or if precautions against it are warranted, the whole procedure must be repeated with the instrument set up on the right bank first and afterwards transferred to the left bank. The mean of the double set is then adopted. This would result in the cancellation of an error due to a uniform change in refraction occurring throughout the duration of the whole job but not that due to fluctuating refraction changes.

8.5.133 Two Levels and Two Staves

The steps, when using this method, are as follow:
(a) A staff and an instrument are set up on each bank and two simultaneous sets of observations are taken. The mean rise or fall derived from these will be free of error due to curvature and refraction, but will include collimation, staff index and graduation errors;
(b) Both levels and staves are interchanged and a further two sets of simultaneous observations taken and the mean deduced. The mean of the two means can be adopted as the best possible value for the rise or fall between the change points.

8.5.14 Special Techniques and Equipment

8.5.14.1 Instrument Check

The instrument should be checked for collimation adjustment using the "two peg" method. Two pegs should be placed about sixty metres apart on level ground to minimise staff graduation error and refraction effects. The instrument is set midway between the pegs so that the collimation errors in the staff readings will compensate, and the true difference in level will be obtained. A second difference in level, observed from a set-up close to one peg, will contain the error. If this exceeds 1 in 20,000, adjustment should be made by a person who is authorised to do so. However, the adjustment and condition of the instrument can deteriorate rapidly in adverse conditions, so methods which will ensure that errors will be disclosed or eliminated by compensation must be adopted.

An alternative is to set out 3 x 20 metre bays on level ground. Set up the level on one of the end points and read to a staff on the closer of the inner points followed by the further of the inner points. Transfer the level to the other end point and repeat the procedure.

The collimation error is given by:

\[ |1| - |2| + |3| = |4| \]

The observed value for reading |4| should agree with that derived from the above equation to 1 part in 20,000 (i.e. 3mm in 60m). This method may be inbuilt into digital levels with a predetermined observation order programmed into the instrument by the maker.

8.5.14.2 Negative Reduced Levels

Certain levels (e.g. inverters of sewers in coastal areas) may be below the datum surface. If there are few of these, the word "minus" must be written before the numerical value. In surveys where many levels are below the standard datum, an arbitrary datum 1000 metres lower should be adopted. The laws of algebra must be applied in the normal reduction procedure by the "rise and fall" method: a fall applied to a negative RL will increase its numerical value; conversely a rise will decrease it. For example, a point three metres below RL minus 105 will be at RL minus 108.

8.5.14.3 Inverted Staff

To obtain the reduced level of a point such as the under side of a bridge, the intrados of an arch, the soffit of a doorway or the obvert of a large pipe, the staff is inverted with the zero end on the point to be labelled. The reading, taken at the line of collimation, is negative in the algebraic sense when compared with a normal staff reading. It is recorded with the word "minus" as a prefix rather than the mathematical symbol.

In the reduction process the laws of algebra are applied to obtain the rise or fall. The normal procedure is then used to deduce the RL.

Sometimes bench marks and change points may have to be established on the roof or crown of a tunnel. Staff readings here will also be negative, while intermediate sights onto the floor with the staff held normally will be positive. If the tunnel is below sea level the RLS will also be negative unless an arbitrary datum is adopted. Clear thinking, aided by a sketch, is essential in such situations.

8.5.14.4 Setting Out

In engineering construction work, pegs and other marks have to be placed at designed RLS. The staff readings required to yield these results are deduced by subtracting the design RL from the RL of the line of collimation of the instrument. The pegs or marks placed should be checked by independent re-levelling from verified bench marks on the same datum used for the design survey. These check readings should be reduced by the "rise and fall" method. Alternatively, a self computing staff can be used. See Section 8.5.14.8.
In setting out a substantial structure, such as the ring base of a large water tank, priority is usually given to producing a level plane rather than an absolute RL. In these circumstances a bench mark must be established on the structure itself at the earliest opportunity, and, after verification, used for all subsequent work. This ensures maximum relative accuracy between the various parts of the structure, and eliminates error due to possible differential movement of the structure and the bench mark.

8.5.14.5 Vertical Measurement

Measurement in a vertical plane, such as the transfer of RL down a shaft, is best carried out using two instruments. A fully graduated steel tape is suspended with a weight on the lower end to supply tension. It is read simultaneously using two levels: one at the surface carrying the RL brought forward from a bench mark, and the other below for transfer onto the new mark. Without moving the instruments, the tape can be raised or lowered to allow more sets of readings to be taken and the mean adopted. Alternatively, the instruments may be moved to obtain additional readings onto a fixed tape.

The observed difference in height on the steel tape has to be corrected for temperature, standardization and the elongation of the tape under its own weight.

The following formulae may be used:

(a) Temperature and Standardization

\[ K_t = L \cdot a \cdot (t_m - t_c) \]

where \( K_t \) = correction
\( L \) = length of used section of tape
\( t_m \) = observed temperature, Celsius
\( t_c \) = temperature of standardization, Celsius
\( a \) = co-efficient of expansion per °C

where \( a = 11.2 \times 10^{-6} \) for steel
\( a = 6.3 \times 10^{-6} \) for invar

(b) Elongation

\[ K_e = \frac{L}{E \cdot A} \left( T + \frac{wL}{2} \right) \]

where \( K_e \) = elongation in metres
\( T \) = tension applied at the lower end in N
\( L \) = length in metres of that portion of the tape being used for measurement
\( w \) = mass of unit length of tape in kg/m
\( A \) = cross section area of tape in m²
\( E \) = modulus of elasticity of tape

where \( E = 21 \times 10^9 \) N/m² for steel
\( E = 15 \times 10^9 \) N/m² for invar

For convenience, the tension applied to the vertical tape should be made equal to the standard tension of the fully supported horizontal tape reduced by half the mass of that portion of the tape involved in the measurement. The elongation of the tape is not linearly uniform, so that it is invalid to take readings other than at points for which the standard tension or correction has been calculated.

8.5.14.6 Improvised Equipment

The surveyor should have the necessary knowledge and confidence so that, in an emergency, he can obtain satisfactory results using obsolete, poor quality or improvised equipment. In this category can be included the Cowley level, a water level made from clear plastic hose, and an “A” frame with a plumbob suspended from the apex.

8.5.14.7 Laser Equipment

With laser equipment a beam of light is projected whereas with a surveying telescope a ray of light is observed. The fundamental principles are similar although the details of their operation may differ. Rotating laser projectors can be set to define either horizontal or inclined planes.

A laser beam is subject to the same refractive phenomena as a normal light ray and in the situations where laser equipment is generally used, these effects may be greater and more frequently encountered. At the same time it is usually difficult to use compensating techniques similar to normal levelling to cancel out the errors due to these causes.

8.5.14.8 Self Computing Staff

This specialised equipment is useful where a large number of surface levels are required in an elevation range not exceeding three metres. The graduations run in the reverse to the normal direction. The face is on an endless band and is adjustable. It can be set and locked so that the reading at the cross hair is the decimal component of the reduced level of the bench mark. A direct reading can then be made of the decimal component and a deduction made of the RL of every staff station. No other reduction is necessary. The RL and its notation is the only record required.

The normal precautionary closing reading onto a second bench mark also serves as a check against possible incorrect setting or movement of the graduated face.

8.5.14.9 Mining Staff

This is a modified form of telescopic staff where the graduated face is on flexible metal and unfolds as it is extended. It can thus be used in pipes, etc., where the first leaf of a normal staff is insufficient and two leaves would not fit.

8.5.15 Errors in Differential Levelling

Misclosure in a circuit of levels or between bench marks can be due to one or more of a variety of reasons. Errors and mistakes tabulated in the following check list must be considered if the cause of a misclosure is to be isolated and rectified.
8.5.15.1 Errors Due to Environmental Factors
These errors include the following:
(a) Wind vibration of instrument;
(b) Wind deflection of staff;
(c) Abnormal refraction of line of sight due to:
   - grazing ray
   - refraction changing with distance (partly over bitumen, water, etc.)
   - refraction changing with time
   - shimmer;
(d) Soil movement affecting:
   - bench mark stability (long term)
   - instrument stability (sinking or soil rebound)
   - change point stability.

8.5.15.2 Errors Due to the Instrument
These errors include the following:
(a) Collimation maladjustment;
(b) Parallax maladjustment;
(c) Micrometer index error (run incorrect);
(d) Compensator error due to:
   - systematic compensation error
   - complete malfunction
   - intermittent malfunction
   - pea bubble centring error
   - loose optics in telescope
   - loose tripod shoes and/or leg pivot bolts
   - sun on tripod legs
   - sun on instrument head.

8.5.15.3 Errors Due to the Staff
These errors include the following:
(a) Graduation error;
(b) Index error;
(c) Failure of clip to catch;
(d) Loose clips;
(e) Loose base;
(f) Dirt adhering to base;
(g) Staff bubble error;
(h) Calibration error.

8.5.15.4 Errors Due to Data
These errors include the reduced level of bench mark being:
- incorrectly published
- incorrectly transcribed into book
- incorrect due to ground movement or disturbance of the mark
- incorrect due to misidentification of the BM
- incompatible with RL of the closing bench mark due to its derivation from a separate source.

8.5.15.5 Errors Due to the Observer
These errors include the following:
(a) Observation Errors due to:
   - incorrect reading of staff graduation
   - reading stadia hair by mistake;
(b) Instrument Moved due to:
   - accidental knock
   - surveyor’s weight on unstable ground;
(c) Recording Errors due to:
   - transposition of observed figures
   - incorrect figures recorded due to memory lapse
   - booking in wrong column
   - booking incorrect data (BM number, RL etc.)
   - staff station position incorrectly recorded;
(d) Reduction Errors due to:
   - misreading calculator
   - BM data transcription error
   - subtraction error in deducing rise and fall
   - rise or fall booked in wrong column
   - addition error in totalling columns
   - failure to total all columns
   - compensating errors
   - error in deducing reduced level
   - error in transferring RL to next page;
(e) Non Equality of backsights and foresights.

8.5.15.6 Errors Due to the Staff Holder
(a) Bench Mark misidentified because:
   - incorrect BM used
   - incorrect mark used as a BM
   - incorrect part of BM used as a bench, i.e. cover or dust cap;
(b) Change Point unstable;
(c) Change Point misidentified;
(d) Staff not vertical;
(e) Base of Staff coated with clay, bitumen etc.

While the effects of many of these errors can be avoided by carefully following the recommended procedures, none of them can be completely discounted when seeking the cause of misclosure. The task is much more difficult if more than one unknown error is involved.
8.5.16 Satellite Based Levelling

Satellite based technology such as GPS is coming into common usage for military and civil positioning of both people and vehicles.

In the survey practice GPS is more likely to be used for horizontal coordinate positioning and precise long distance determination than for determining height differences.

The GPS satellite constellation is intrinsically more suited to determining horizontal positioning rather than vertical poiting, due to the fact that satellite reception from below the horizon is not possible.

8.5.16.1 General Principles

Spherical height differences can be determined to high accuracy depending on the technique adopted. The height of the antenna electronic centre needs to be measured accurately, and the duration of the observation session needs to be tailored to the desired accuracy of the result. As an example, for lines of 500km 3 to 5 24 hour sessions will need to be observed to obtain the best possible results. This will also require the use of sophisticated reduction algorithms.

The further reduction of the spherical values to the geoid (to equate to AHD) requires the use of a high order geoid undulation model of which the best currently available is AUSGEOID 98.

For shorter distances on local projects observation session times can be shorter and local AHD Bench Marks can be employed to model the local geoid surface.

In this case observation sessions can be as short as 15 minutes but the factors of desired accuracy, available control, distance involved and satellite availability require an informed professional judgement.

In all cases better results can be obtained by spending more time collecting satellite data.

8.5.16 Trigonometric Levelling

The trigonometric method of levelling is generally less accurate than differential levelling, but it can be used in situations where time factors or difficulties involved make the results acceptable.

Trigonometric levelling is generally accurate to between 0.5 m and 1.5 m depending on the technique used. However, third order levelling accuracy can be obtained by observing simultaneous reciprocal vertical angles and EDM distances of less than 1 km.

8.5.16.1 General Principles

By measuring the distance between two points and observing the vertical angle or zenith distance at one point to the other, the difference in height between the two points can be found. In the case of single observations, corrections must be made for curvature and refraction. If simultaneous reciprocal observations are taken, the refraction correction can, theoretically, be eliminated.

8.5.16.2 Curvature

The curvature correction is directly related to the measured distance and radius of the Earth, and can be allowed for exactly. See Section 8.5.16.5.

8.5.16.3 Refraction

The chief source of error in trigonometric levelling is that arising from the uncertainty regarding the amount of atmospheric refraction.

It is important to realise that the coefficient of refraction k, (Section 8.5.3.4), varies with the atmospheric conditions at the time of observation and with the topography over which the line of sight runs. It will not necessarily be the same at either end of the line. Therefore, a value of k determined at a particular moment of time is unlikely to be valid for another line observed at a different time.

Refraction effects are least when the atmosphere is most stable; this is generally in the early afternoon, and is the optimum time for observations.

Section 8.5.3 deals with this topic in detail.

8.5.16.4 Methods of Trigonometric Levelling

There are two methods of trigonometric levelling in common usage.

8.5.16.5 Simultaneous Reciprocal Observations

If the zenith distances are observed simultaneously at either end of a line, the refraction effect is minimal. The difference in height is given by the formula:

\[ A h = h_b - h_a = S \tan \frac{1}{2}(Z_b - Z_a) + \frac{1}{2}(i_b + g_b - i_a - g_a) \]

where

- \( A \) = station of known height
- \( B \) = station of unknown height
- \( h_b - h_a \) = correction to be applied algebraically to RL of A to obtain RL of B
- \( S \) = mean sea level distance between A and B
- \( Z_{a} \) and \( Z_{b} \) = zenith distance observed at A and B
- \( i_{a} \) and \( g_{a} \) = heights of instrument and target respectively at A.
- \( i_{b} \) and \( g_{b} \) = heights of instrument and target respectively at B.

The above formula is suitable for most trigonometric levelling situations. If third order accuracy over short lines is required, the following formula should be used:

\[ h_b - h_a = S \tan \frac{1}{2}(Z_b - Z_a) \left(1 + \frac{h_b}{r}\right) \]
\( 1 + \frac{S}{12r} \) \\
+ \frac{1}{2} (i_\text{a} + g_\text{a} - i_\text{b} - g_\text{b} \)

where \( h_\text{m} \) = mean height between A and B
\( r \) = radius of the Earth

The above formula assumes that the refraction effect is equal at both ends of the line. This assumption will hardly ever be exactly true for long lines, but for short distances (up to one kilometre) the refraction effects, although not equal, will be small and insignificant.

### 8.5.16.6 Single Observations

In this method, the difference in height is derived from the zenith distance measured at one of the stations only. The value of the refraction coefficient is required.

The difference in height is given by the formula:

\[
h = h_\text{a} - h_\text{b} = S \cot Z_\text{a} + \frac{(1 - 2k)S^2}{2r} + i_\text{a} - g_\text{a}
\]

where \( k \) = coefficient of refraction. Adopting \( k = 0.07 \), the combined curvature and refraction correction \( \frac{(1 - 2k)S^2}{2r} \) amounts to 67 mm for one kilometre, and varies as the square of the distance.

Since the coefficient of refraction is variable, this method is inferior to that of simultaneous reciprocal observations.

### 8.5.16.7 Reduction to Station Mark

Instead of applying the heights of instrument and target as a linear correction to the computed height difference, a correction to the observed zenith distance can be computed.

\[
\text{Correction} = C'' = \frac{g_\text{b} - i_\text{a}}{S \, \text{arc } 1''}
\]

### 8.5.16.8 Determination of “k”

The coefficient of refraction is defined as the ratio between the refraction angle (which is the angle at the instrument between the ideal straight line of sight and the actual curved line of sight) and the angle which the two stations subtend at the centre of the earth. If reciprocal observations have been taken, the refraction coefficient can be derived from the following formula:

\[
k = \frac{(180' + \theta) \cdot (Z_\text{a} + Z_\text{b})}{20}
\]

where \( Z_\text{a}, Z_\text{b} \) = the observed zenith distance (corrected for the heights of instrument and target by the formula in Section 8.5.16.7), and \( \theta = \) the angle subtended at the centre of the earth \( \frac{S}{r \, \text{arc } 1''} \).

For ease of computation, the formula may be transformed to:

\[
k = \frac{S - (Z_\text{a} + Z_\text{b} \cdot 180') \, \text{r.arc } 1''}{2S}
\]

For this purpose, the value at Melbourne of r.arc 1" in metres is 30.83. The “k” in this context should not be confused with the meteorological factor “k” in some European formulae for the reduction of long range EDM observations.

Whenever reciprocal observations are made, the value of “k” should be computed for comparison with the empirical value \( k = 0.07 \). If during observations the time, weather conditions and terrain along the observed line are noted, data can be assembled to gain an appreciation of the variation of refraction effects for local areas, and as a possible value of \( k \) for single observations.

### 8.5.16.9 Equipment

(a) Theodolite. A one-second theodolite is needed for all work where an accuracy of greater than 1 in 10 000 is required.

The datum to which the full circle vertical angle is referred must be noted.

Depending on the type of instrument used, the following vertical circle datums can be encountered:

(i) Zenith Datum. This datum reference is used in the majority of instruments;
(ii) Nadir Datum. This datum reference is not common;

(b) Targets. Targets of some type are essential. For reciprocal observations over short lines the trunnion axis of the other theodolite can be used. For longer lines, lamps are needed, set over the ground marks on low tripods between the legs of the theodolite tripod;

(c) Communication. A method of communication is needed to achieve truly simultaneous reciprocal observations. Transceivers can be used for short to medium length lines, while for long lines use can be made of the voice communication of long range EDM.

8.5.16.10 Observing Procedure

The method and observing technique adopted will depend on the degree of accuracy specified for the particular job.

(a) Selection of Stations. Grazing rays, as shown below, should be avoided when selecting stations. The refraction effects at A would most likely differ markedly from those at B. A better choice of points would be C and D;

(b) Optimum Observing Time. The period from noon to 1500 hrs Local Mean Time is considered the most favourable observing time;

(c) Vertical Angles. To obtain satisfactory results, vertical angles must be carefully observed. It is desirable to make four face left and four face right readings for long lines, or to achieve third order levelling accuracy over distances up to one kilometre. The number of pointings may be reduced for jobs requiring less accuracy;

(d) Height of Instrument and Observed Target. Each observation must include a record of the height of instrument (axis) and observed point (target) above ground level or mark. Care must be taken in these measurements; for example, if third order levelling accuracy is sought, a 0.015 metre error in the height of a target for a distance of one kilometre would make the results unacceptable;

(e) Distance Measurement. The distance may be found by computation or scaling. Generally, it is measured with EDM equipment, especially if accurate results are required.

8.5.16.11 Applications

Trigonometric levelling techniques may be used for a wide range of field situations. The method chosen will depend on the accuracy required, terrain, time factor, and personnel and equipment available.

(a) Methods

(i) Simultaneous Reciprocal Observations. This requires two survey parties but yields the most precise results. Third order levelling accuracy can be attained by using closed loops of short lines;

(ii) Non-simultaneous Reciprocal Observations. If only one observing party is available, lesser accuracy must be accepted. However, the accuracy may be improved by choosing ideal conditions, shortening the lines and the length of the loops;

(iii) Single Observations. A lower order of accuracy will be attained. Correction for curvature and refraction has to be applied to each observation. Scaled distances may be adequate for reconnaissance work. Using EDM for distance measurement supersedes and greatly extends the range of stadia tacheometry for the production of a spot level network.

(b) Field Uses

(i) Control Network. Trigonometric levelling can be carried out in conjunction with or independent of a triangulation scheme or traverse framework. Take simultaneous reciprocal observations and where possible tie the trigonometric heighting traverse into points of known RL;

(ii) Levelling of Photo Control Points. Photo control points can be levelled quickly, and to the required accuracy, by using trigonometric levelling;

(iii) Large Areas of Difficult Terrain Requiring Levels. If extreme accuracy is not required, trigonometric levelling can be used over a large area containing high points. Single observations with scaled or EDM distances is usually sufficient, particularly if checks from a second instrument station can be obtained;

(iv) Preliminary Levels for Engineering Projects. With the advent of EDM, the slope distance can be measured accurately, and this has led
to trigonometric levelling being used as an alternative, in some cases, to differential levelling over distances up to 1 km.

To achieve results equivalent to those obtained by differential levelling, careful simultaneous reciprocal observations must be made, along with a measured slope distance. Using this technique, third order levelling accuracy can be obtained.

Third order levelling results obtained by trigonometric levelling are satisfactory for many engineering situations: to obtain the equivalent result by conventional levelling would be time consuming.

 Situations in which third order trigonometric levelling could be used include:

- Levels for preliminary planning
- Check levelling
- Levels for construction or design purposes (if the accuracy is suitable).

8.5.17 Barometric Levelling

The use of an aneroid barometer may be the most economical and convenient method of obtaining spot heights in some situations: for example, in reconnaissance work or in rough timbered country, where a low order of accuracy is acceptable. An accuracy of 2 to 5 metres can be obtained depending on the type of equipment used, and the methods adopted.

Due to the development and improvement of alternatives, the need for barometric levelling now arises less frequently, but a working knowledge of the subject is still essential.

8.5.17.1 General Principles

The surface of the earth is the bottom of an ocean of air.

The difference in elevation between two points on this surface is the difference between depths of each point below the top of the atmosphere.

These depths are determined by measuring the weight of the column of air above each point using a sensitive pressure gauge.

To derive height differences from the observed pressures the reduction formulae and procedures take into account variations in temperature, humidity and atmospheric wave phenomena.

Certain assumptions are made:

(a) The mean of the air temperature observed at the base of each of two air columns is taken to be the mean temperature of the air column rising above the lower point to the level of the upper point;

(b) No attempt is made to measure humidity and an average value is built into the formula;

(c) Variations in the readings of the base barometers are taken to apply equally and simultaneously at all field stations. These variations will be a combination of the diurnal effect and the changing meteorological conditions.

Because of the errors in these assumptions, turbulence within the air columns and the mechanical limitations of the aneroid barometer, the levels derived from observations have inaccuracies. Procedures have been laid down to limit these inaccuracies. If the limitations are clearly understood the method is invaluable in appropriate situations.

8.5.17.2 The Surveying Aneroid Barometer

The basic instrument is an aneroid barometer. This essentially consists of a metal container exhausted of air (the vacuum box), which moves in response to changes in air pressure. Small deflections are magnified some 200 to 300 times and shown on a scale, generally graduated to show both pressure and the indicated altitude derived by an appropriate formula.

8.5.17.3 Reading Systems

In the standard surveying aneroid, the connection between the vacuum box and the pointer is a complex and delicate system of pivoted levers and linkages. Friction, wear and small displacements of the moving parts due to jolting in the field can affect the readings. To minimise this, other designs of instrument have been developed:

- In the ‘Paulin’ type instrument there are very few moving parts which are only operated for reading purposes when a pointer is brought to a null position.

- Modern instruments are available with an electronic indicator and digital readout but they still rely on the movement of an aneroid capsule.

8.5.17.4 Reading Irregularities

- ‘Drift’ is a gradual long term creep in the indicated reading of an aneroid which follows a marked change in pressure. It is independent of minor variations in the atmosphere and takes place in the direction of pressure change after the pressure has become steady.

- ‘Lag’ is the effect which prevents an aneroid from registering its final reading for a short interval after a sudden change of altitude. The effect may be minimised by tapping the glass gently and waning five minutes before taking a reading.

- ‘Index Error’ is the difference in indicated altitude between two aneroids when standing side by side. This should remain constant, but due to drift and lag may be quite different at the end of the day.

The combined effect of drift, lag, and index error may be allowed for in the reduction procedure by taking readings from the traverse barometer at the base position before during and after the job, for comparison with the base barometer.

8.5.17.5 Temperature of Calibration

The altitude scales of surveying aneroids are graduated according to an adopted standard temperature, and the observed difference in height has to be cor-
To determine the calibration temperature of a particular aneroid, set zero on the altitude scale to 1030 mb on the pressure scale and read the indicated altitude opposite 990 mb. The temperature of calibration is then deduced from the following table:

<table>
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<tr>
<th>Indicated altitude (m)</th>
<th>318</th>
<th>323</th>
<th>329</th>
<th>335</th>
<th>341</th>
<th>347</th>
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<tbody>
<tr>
<td>Temperature of calibration</td>
<td>0°C</td>
<td>5°C</td>
<td>10°C</td>
<td>15°C</td>
<td>20°C</td>
<td>25°C</td>
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</tbody>
</table>

This indicates which formula should be used to derive the correction for temperature of the air column which has to be applied to the indicated difference in height.

8.5.17.6 Temperature Compensated

An aneroid barometer which is marked “temperature compensated” has allowance made for the effect of temperature on the instrument itself including the linkages between the vacuum capsule and the pointer. Consequently, when the temperature changes but the air pressure remains constant the reading does not alter.

However, a correction still has to be applied to the observed difference in height to allow for the difference between the mean temperature of the air column and the temperature of calibration of the altitude scale.

8.5.17.7 Field Procedures

The basic field procedure consists of taking readings on a mobile “traverse barometer” and comparing them with simultaneous readings on a stationary “base barometer”.

In practice, methods are used to improve the accuracy of the result or to overcome problems due to limited equipment or difficult topography. It is assumed that the following basic procedures and constraints have been noted and that mean indicated altitude readings have been adopted.

8.5.17.8 Basic Procedures and Constraints

(a) To maintain accuracy, restrict each day work to a limited area and vertical range;
(b) Make checks onto bench marks, where applicable, and by returning to the base station at least every four hours;
(c) In mountainous country, confine levelling to one main valley. Avoid crossing main ridges into adjacent valleys, which have independent atmospheric characteristics;
(d) Suspend field work at times of fluctuating barometric pressure e.g. during strong winds and at the approach of storms. Results are unreliable at air temperatures exceeding 35°C;
(e) Mount aneroid barometers in padded carrying boxes to safeguard them against shocks. When available they should be used in groups of three to minimise the effects of individual variation and isolate mal-function;
(f) Thermometers are required to record air temperatures at base and in the field and watches are needed to synchronize the readings.

8.5.17.9 Single Base Method

One barometer or group remains stationary at a base of known RL while one or more mobile traverse barometers are carried around to the field sites:

(a) The base is set up in a well shaded position, open to the air. The barometers are placed face up and level at waist height. The thermometer is hung at least a metre above the ground. Watches are synchronized;
(b) The field barometers are placed alongside the base instrument and allowed to settle for at least fifteen minutes before reading;
(c) Time, temperature and the indicated altitude of each barometer is recorded on the appropriate base and field forms;
(d) The base barometers continue to be read at ten minute intervals throughout the day;
(e) The mobile field barometers are carried to the required locations and held at waist height for reading. To eliminate lag effects they are first allowed to settle for five minutes;
(f) Wherever possible additional readings are taken at bench marks and at ground stations included in other networks;
(g) A check at base is made at least every four hours and at the end of the day. A settling period of fifteen minutes must be observed.

8.5.17.10 Double Base Method

This method was used by the Snowy Mountains Authority to improve accuracy. An upper and lower base is established at control bench marks and the field stations are kept as far as possible within the altitude range between the two:

(a) All instruments, base and traverse are read simultaneously at the most convenient base station;
(b) The other base station is then occupied and both base instruments are read simultaneously at regular 10-15 minute intervals throughout the day;
(c) The times of reading the field instruments are recorded and at the end of the day all instruments are brought back to the first base for simultaneous reading;
(d) For each field station observed differences in height above the lower base and below the upper base are deduced. Any discrepancy is adjusted out in direct proportion to the two observed differences.
8.5.17.11 Leap Frog Traverse Method

The leap frog traverse requires two mobile batteries of instruments and observers, designated A & B, to move alternatively so that simultaneous observations are made at each successive pair of stations.

(a) At the initial benchmark at station 1 “A” & “B” are read simultaneously. While “A” remains at station 1, “B” moves forward to station 2 and simultaneous readings are taken, perhaps synchronized by radio. “A” then joins “B” for further observations at station 2 which now becomes the base for a similar extension to station 3;

(b) Alternatively “A” may leap frog “B” and go directly from station 1 to station 3 for another pair of readings. “B” will then by-pass “A” to establish station 4 and so on. Ultimately A & B will come together for final simultaneous readings at the last station.

Radio contact to ensure simultaneous readings obviates the need to make a series of time and altitude notations at the base station. The difference in altitude between each pair of stations is derived separately followed by the adoption of reduced levels.

Variations may be introduced into the procedure to provide cross checks or a closed loop. Accuracy may be increased by keeping the distance between stations and the time interval between readings as short as possible. Temperatures must be recorded at the time of reading and allowances made.

8.5.17.12 Reduction of Observations

Basic Formulae. The fundamental formula for the reduction of barometric observations to determine height differences is:

\[
 h_2 - h_1 = K \log_{10} \left( \frac{p_1}{p_2} \right) (T_m + 273.2)
\]

where:

- \( h_1, h_2 \) = altitudes at lower and upper stations 1 and 2.
- \( p_1, p_2 \) = corresponding pressures in millibars.
- \( T_m \) = mean temperature in degrees Celsius of the air between 1 and 2.
- \( K \) = a constant depending on the composition of a test sample of air taken to represent average conditions.

For average composition of air with 66% humidity and tested at 273.2 degrees Kelvin,

\[
 h_2 - h_1 = 67.58 \log_{10} \left( \frac{p_1}{p_2} \right) (T_m + 273.2) \text{ metres}
\]

Note that as the pressures measured appear in the expression as a ratio, the unit in which they are measured is not significant (but should of course be in millibars as an SI unit).

Thus, although older English aneroids were graduated in “inches of mercury”, the above formula may be used with \( p_1 \) and \( p_2 \) as read on the instrument. The height difference will be in metres.

For conversion to alternative units the following data is useful:

Standard atmosphere -

\[
 760 \text{ mm Hg} = 1013.25 \text{ millibars} = 29.92 \text{ ins Hg}
\]

1 inch = 25.4 mm

0°C Celsius = 273° Kelvin = 32°Fahrenheit

100°C Celsius = 373° Kelvin = 212°Fahrenheit

### BAROMETRIC PRESSURE CONVERSIONS

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Annex 6 is a form produced for instructional purposes for the reduction of barometric heights. It is based on Airy’s formula.

8.5.18 Other Methods

Alternative techniques are available for the derivation of heights and contours. These require specialized equipment for the higher degrees of precision, and will not be treated in detail. Only a brief statement of performance characteristics is provided.

8.5.18.1 Photogrammetry

Aerial and terrestrial photographs can often provide the most economic means of obtaining general topographic information. Where contours only are required at small intervals and over relatively small areas, a careful cost analysis against conventional levelling is necessary.

The accuracy of photogrammetric work depends on the scale, quality and orientation of the photography, the density and placement of ground control, and is also influenced by the nature of the terrain and vegetation.

Contours to an interval of 0.5 metre can be fairly readily obtained by photogrametry in open terrain. Smaller intervals require very refined techniques, and reliability of the finished product checked by independent methods.
Generally 90% of the contours will be accurate to within half the contour interval, irrespective of the scale of the plan. Also, in many circumstances, the shape of the topography will be more truly displayed by photogrammetry than by interpolated field survey levelling.

8.5.18.2 Profile Heighting

(a) EchoSounding. An echo sounder, fitted in a boat, transmits an impulse to the sea bottom. The return impulse is picked up by the receiver, and from the recorded time interval between transmission and reception a depth can be derived.

The operating range is up to 100 metres in depth and an accuracy of ±0.1 metre can be expected from a surveying type echo sounder.

(b) Airborne Profile Recorder. An airborne profile recorder is used to determine a profile of the ground beneath the path of the aircraft in which it is carried. The method has its greatest application over inaccessible terrain. The earlier radar based equipment has been superseded by the more precise laser based equipment. With this system, the vertical separation between aircraft and ground can be measured to very small limits, but for absolute heights of terrain, the continuous determination of the height of the aircraft above sea level is most critical and will ultimately control the quality of the heights obtained. With laser based equipment, these can be obtained to precisions of the order of 1 metre in the best situation.

BIBLIOGRAPHY

8.5.19 Digital Levelling

8.5.19.1 Introduction

Digital levels are classified as automatic levels because they use compensators. The type of compensator varies with the manufacturer but it is generally damped magnetically or by air or liquid.

Digital Levels measure electronically to bar-coded staves, but can also be used to measure optically to conventional staves.

The measurement principles of the digital level are generally the same in both optical and electronic measurement modes.

The advantages of the digital levelling system are its ease of use, the fact that the information is digitally recorded, thus eliminating reading and writing errors, and the ability to obtain heights and distances automatically during field observations.

There are a number of digital levels on the market, and instruments are available for all types of levelling including precision levelling.

8.5.19.2 Digital Levels

The digital level can be utilised as a normal optical level, i.e. by looking through the telescope, focusing the reticle on a conventionally graduated staff and reading the horizontal hair intercept.

In digital mode it is still necessary to focus onto the bar-coded staff to enable the reading to be taken; then, depending upon the particular manufacturer’s mode of operation, the instrument can be used to measure with or without recording.

8.5.19.3 Levelling practices.

8.5.19.3.1 Observing near the extremities of the staff

Observations read to near the top or the bottom of the staff should be sufficiently far from the end to ensure that the staff fills the field of view as seen through the instrument.

Digital levels will generally give an audible warning if there is insufficient coded staff within the field of view to resolve an observation.

8.5.19.3.2 Sight Distances

As in optical levelling, the distance to the staff should not exceed 25 to 30m. Other rules that apply to conventional optical levelling also apply to digital levelling. Observations to bar-coded staves can be affected by dappled light and shade patterns falling on its face which can impede the operation of the electronic sensor.

8.5.19.4 Calibration

8.5.19.4.1 Instrument

Testing and adjusting of a digital level should be performed in the same manner as is used for an optical level, i.e. The “two-peg” test and bull’s eye level check.

Should problems still be evident after these tests and adjustments have been performed, it is recommended that the digital level be returned to the agent for maintenance.

8.5.19.4.2 Staff

Digital levelling utilises bar coded staves which come in a variety of configurations, sizes and materials.

In general, invar bar-coded staves are offered with and without certification from the factory, depending on the vendor.

Manufacturers do not offer certification on the standard bar-coded staff.

Should certification be sought for bar-coded staves, enquiries can be made of Land Victoria.
## ANNEX 1

**SURVEY PRACTICE HANDBOOK - VICTORIA**

**PART 2 2000**

**REF-S145**

**STANDARD LEVELS: MMB7013 to MMB7015**

**EAST MITCHELLTOWN**

**MAP REF: YARRA 2:5/175912**

**COMMENCED 4-4-79, COMPLETED 5-4-79**

**SURVEYOR: R. SMITH**

**PARTY: G. JONES R. BROWN**

**INSTRUMENT: N. I. 2, NO. 87451 WITH PARALLEL PLATE**

**STAVES: INVAR "P" AND "G"**

**WEATHER: FINE, CLOUDY, SOUTH BREEZE, TEMP 14°C**

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**CROSS SECTIONS**

- **Left**
- **Right**

**Mean Fall**

\[
\text{Mean Fall MMB 7013} = \frac{2.1508 + 2.1499}{2} = 2.1503 \text{ metres}
\]

**SURVEYOR**

R. SMITH

R. SMITH

April 79
### Survey Practice Handbook - Victoria

**Part 2 2000**

**Map Ref:** YARRA 2-5/175912

**Weather:** Calm, cloudy, temp 18°

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<th>Inter</th>
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<th>Rise</th>
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**Statements**

- MMB 7013: Fall 2.143
- MMB 7015: Fall 2.150

**Surveyor:** R. Smith

**Date:** 3-4-79
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R. Smith
R. Smith

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MAP REF: YARRA 2.5/17912
WEATHER: CALM. CLOUDY. TEMP 18°
Ref: T153

AS CONSTRUCTED LEVELS

BOOKER HILL TANK, BAKERSVILLE.

SURVEYOR: R. SMITH
PARTY: G. JONES
INSTRUMENT: ZEISS N12 NO.7142 STAFF "K"
DATE: COMMENCED 4/7/79, COMPLETED 4/7/79
WEATHER: CLOUDY, COOL BREEZE, TEMP 13°

MAP REF: CORIO 87/328712

SURVEY PRACTICE HANDBOOK - VICTORIA
PART 2 2000

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CROSS SECTIONS

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(40.155 PUBLISHED) MMB 4.177 plaque in concrete S.W. corner. A, top of concrete.
B, top of concrete.
C, top of concrete.
D, top of concrete.
E, top of concrete.
F, top of concrete.
G, top of concrete.
H, top of concrete.
I, top of concrete.

(40.650 PUBLISHED) T.B.M. rivet in ringwall.
T.B.M. rivet in ringwall.
T.B.M. rivet in ringwall.
T.B.M. rivet in ringwall.

R. Smith
R. SMITH
4-7-79
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**Remarks**

**SURVEY PRACTICE HANDBOOK - VICTORIA**

**PART 2 2000**

**ANNEX 5**

**SURVEYOR:** R. SMITH  **PARTY:** G. JONES

**INSTRUMENT:** Ni2 17321 STAFF "K"  **WEATHER:** CLOUDY. STIFF BREEZE. COLD. TEMP 11°
### Annex 6

<table>
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<tr>
<th>Station Number</th>
<th>Description</th>
<th>Time</th>
<th>Field Barometer mb</th>
<th>Field Temperature Celsius</th>
<th>Base Barometer mb</th>
<th>Base Temperature Celsius</th>
<th>Corrected Field Barometer mb</th>
<th>Interpolated Drift and Index Correction</th>
<th>Closure Adjustment</th>
<th>R.L. of Field Station (Metres)</th>
<th>Height Above Base (Metres)</th>
<th>Adjusted Reduced Level</th>
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<td>BM A RL = 402.6 m</td>
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