Monitoring the Three Dimensional Movement of a Large Stone Structure

M.A.R.Cooper
N.E.Lindsey
D.M.Stirling
Department of Civil Engineering, The City University
London
United Kingdom
Commission V Working Group V/4

Abstract. The paper considers the problem of measuring the movements of a stone railway bridge, which is in constant use. Previous methods of monitoring movement are briefly described and the photogrammetric method is discussed. A comparison is made of two methods of providing the geodetic control. The restitution of the photography was by independent model phototriangulation, and sets of coordinates for targetted points obtained. Methods of presenting the results for the bridge engineer are considered as are the problems associated with selection of suitable datum points.

INTRODUCTION

Skelton Bridge carries the East Coast Main Line of British Rail over the River Ouse 3 miles northwest of York. It is a stone bridge more than 100 years old with three main spans and a cattle creep and has two piers in the river. The bridge is 100 m long and shows marked deformation in the parapet wall over the southern pier on the upstream side. This deformation apparently has existed for some years but it is uncertain whether the movement is continuing or what the total area is that has been affected. With the introduction into service of Inter City 125 trains with their increased impact loading British Rail bridge engineers initiated a monitoring project. Initially British Rail employed precise levelling to determine settlement and measured offsets from a reference laser for deflections. This involved working on the actual bridge and entailed closing the line. Photogrammetry was then tried in the hope the deflections and settlement could be determined without closing the line and disrupting very busy rail traffic. This proved feasible and photogrammetric surveys were carried out four times over a four year period. Stone structures subjected to dynamic loads and influenced by water table levels, soil conditions and atmospheric conditons are liable to constant internal movements often of such a magnitude that movements actually caused by settlement are hidden. Photogrammetric surveillance surveys because of their depth of cover and resolution now make it possible to monitor the normal movements within the structure and provide an opportunity to detect trends in movements likely to be caused by the settlement. Unlike conventional survey methods in which unique points are observed, the photogrammetric survey permits any number of points to be observed.

CONTROL SURVEY

The Bridge at Skelton is constructed on the flood plain of the River Ouse and is probably founded in alluvial deposits which are different on either side of the river. The whole area floods on a regular basis and so the control established on the bridge was referred to a stable station on high ground over 200 m from the structure. A network of eight survey stations was used to transfer the datum and reference direction to the bridge. The stations themselves were hand augered concrete piles 150 mm in diameter and 2-3 m long debonded for the upper 1.5 m and anchored at the lower end. These stations were positioned by British Rail engineers ready for observation in April 1978. The type of pile and its method of placing was recommended by British Rail Research Department, Derby as it was considered that they would maintain their plan positions even if subjected to vertical movement because of flooding.

The network was observed for each survey. All possible angles and distances in the network were measured and the stations were precisely levelled. For the first three visits 1" theodolites and Tellurometer MA-100 infrared EDM were used. A Wild TC-1 electronic tacheometer was used on the fourth visit. A further five survey stations were placed on an adjacent bridge to the main bridge and these were resected by bearing and distance from the eight main stations. Twenty five targetted points were then coordinated by theodolite intersection on the main bridge from these five stations. The network was adjusted using a three-dimensional variation of coordinates program.

In all four surveys the standard errors of the targetted bridge control points were $\pm$ 2.5 mm. In the first three visits around ten of the 247 measured values were rejected from the final adjustment for having a residual of greater than three times the Sigma zero of the adjustments. In the fourth survey only three observations were rejected. This better consistency using the TC-1 is probably due to the elimination of centering errors between the angle and the distance measurements. Although automatic recording was not employed the TC-1 improved the efficiency of the survey by over 50%

PHOTOGRAPHY

A strip of seventeen photographs, covering the upstream face of the bridge was taken on each visit using a Zeiss UMK camera fitted with a 100 mm focal length lens. The photography was taken from the adjacent Slow Line bridge affording a taking distance of 12 m glass plates 180 mm x 130 mm were used and the camera only needed approximate alignment and levelling for each exposure.

The first set of glass plates obtained were used in a Zeiss Stereometrograph to produce a 1:50 scale plot of the bridge for inspection and record purposes. No further plotting was carried out using subsequent sets of photography as the expected movements would be too small to be significant at the plotted scale.

All the photography was measured on a Zeiss Stecometer as a series of stereopairs, sixteen per visit, with the observations being recorded on paper tape. An independent model block adjustment routine (Cooper,1982) was used to obtain the coordinates of 250 targetted points on the bridge face. The internal precision of the adjustment produced a root mean square error of $\pm$1.02 mm. The root mean square fit onto the surveyed control was $\pm$2.21 mm. This would appear to confirm that modern photogrammetric equipment and techniques often produce a higher internal precision than the geodetic methods used for the control surveys.

PRESENTATION OF RESULTS

The original requirement was to produce a list of coordinates of the targetted points on the bridge. By the time the third set of coordinates was available it was proving very tedious to compare lists of coordinates Therefore a graphical method of illustrating movement of the structure was tried. Displacements between consecutive sets of photography were plotted as vectors in the XY,XZ and YZ planes at full size (the mean vector was 5 mm) as an overlay to the 1:50 plot. Immediately a pattern of movement for the bridge became apparant (Fig.2). The rising and falling of the bridge coincides with periods of flooding of the river at the time of the 1979 survey. This was not regarded by the bridge engineers as significant but the twisting of the bridge (Fig.2.) was, and a fourth survey was requested to see if a cyclic pattern was discernable. Also produced along with the plots were lists of the changes in coordinates that occurred between consecutive surveys (Table.1).

SELECTION OF DATUM STATIONS

The control survey was related to the remote Station A (Fig.1.) and orientated to a reference direction sighted onto Station F. All the coordinates for control stations, re-sected camera stations and targetted points on the bridge face were established within that set of rectangular axes. The resultant displacements as depicted on Figs 2 and 3 derived from the four sets of sequential photography between 1979 and 1981 were then all based on the assumption that Station A remained stable. This was acceptable since the indicated movements were close to the normal range of mobility exhibited by large stone structures.

When the monitoring programme had finished it was decided to investigate the effects should A have moved. This was achieved by selecting another station as fixed datum, the new station was one established on the railway embankment at the South end of the bridge; Station D.It was assumed that Station D probably could move but, the fact that it was on the embankment suggested that the results with respect to this station,as datum, might be more useful than the 'absolute' movements related to Station A.

Tables I and II show a sample range of derived displacements for the period 80-81 Table I based on Station A and Table II based on the new datum Station D. The values shown in Table II are noticeably smaller than those in Table I.

The whole question of datum stations is important and requires proper discussion with the structural engineer at the planning stage of any monitoring project. The time and expense involved in establishing an absolute datum at a stable point may not always be necessary. The absolute movement of a structure with respect to some stable remote point may not be as important as relative movement within the structure itself.

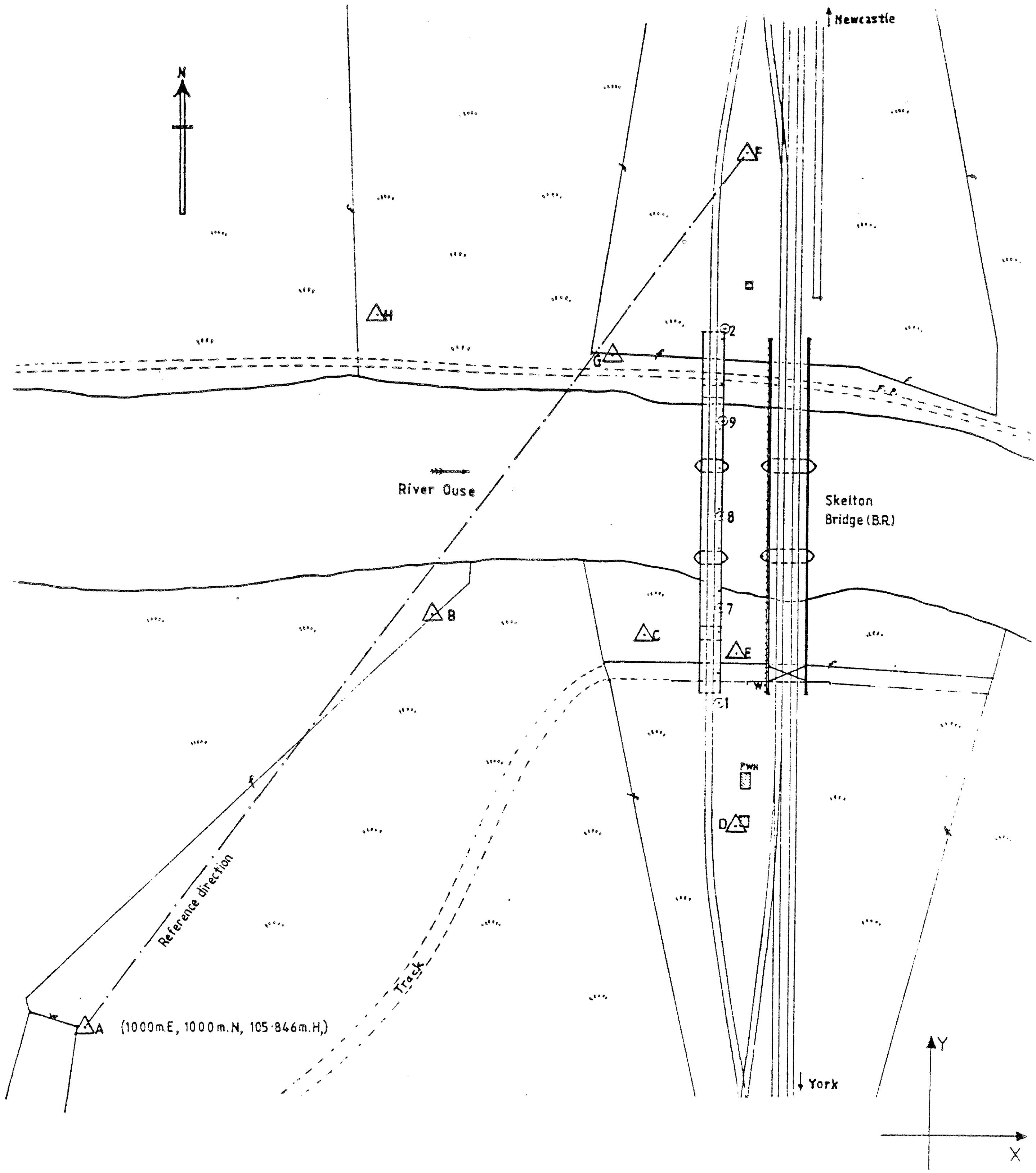
A subject presently under examination at The City University is that of so called "free" adjustment with no explicitly fixed datum. However there is some resistance among a structural engineers to this concept.

CONCLUSION

The photogrammetric monitoring method proved to be acceptable. The movements detected were regarded as insignificant and did not justify further annual monitoring, every five years will be suitable.

The information which came from the survey confirmed the widely held theory that structures particularly stone structures are continually adjusting position or "breathing" as conditions around them fluctuate. It is necessary to look for extended movements as trends over a period of time to be able to recognise new or significant changes brought about by material or structural failure.

One major advantage which became apparant to the engineer during the survey programme was the permanent photographic record which could be re-examined and measured at will.



LAYOUT OF GROUND-CONTROL STATIONS

Fig 1.

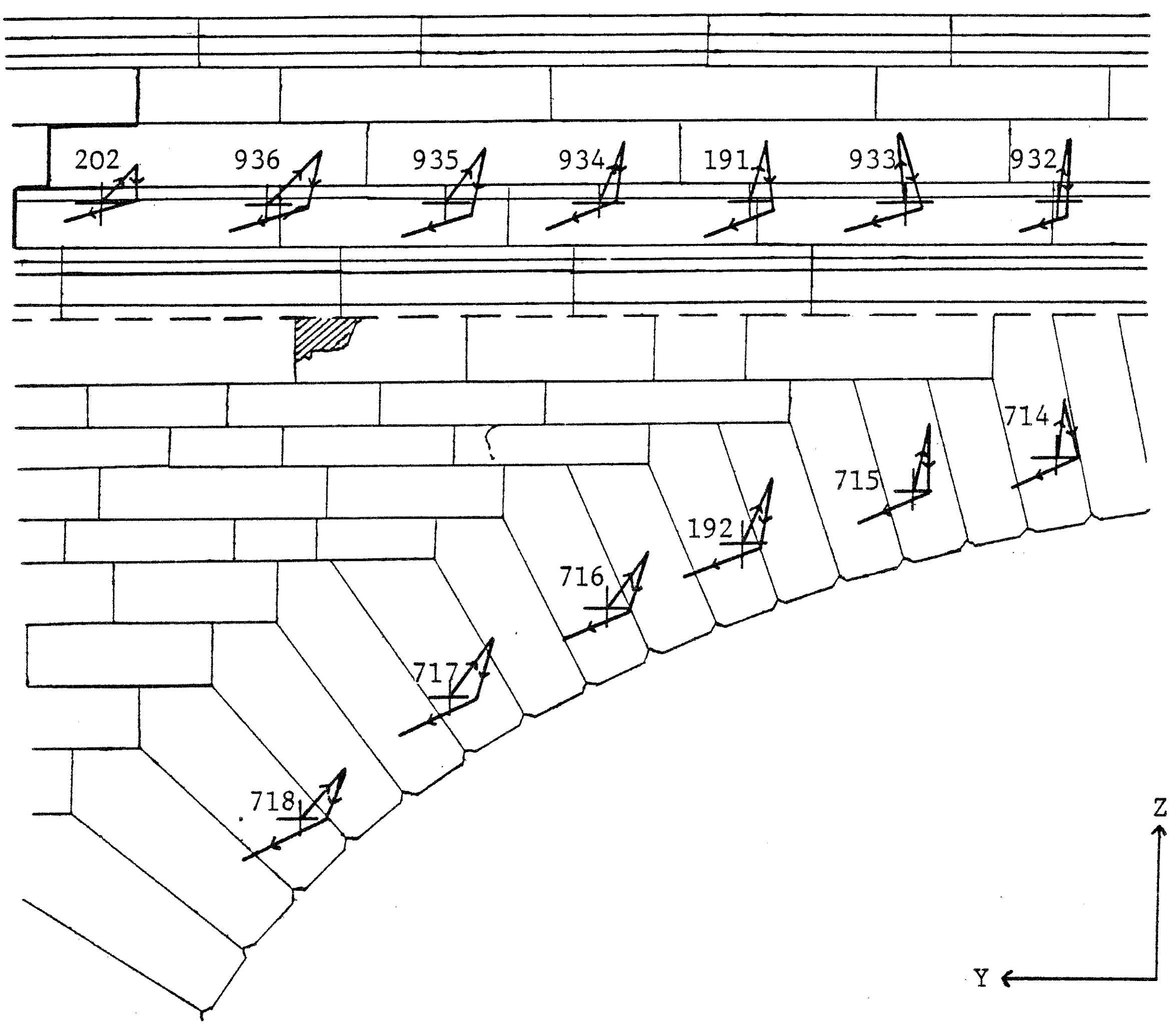


Figure 2. Y-Z displacement vectors overlay on 1:50 detail plot.
(Displacement vectors at full size)

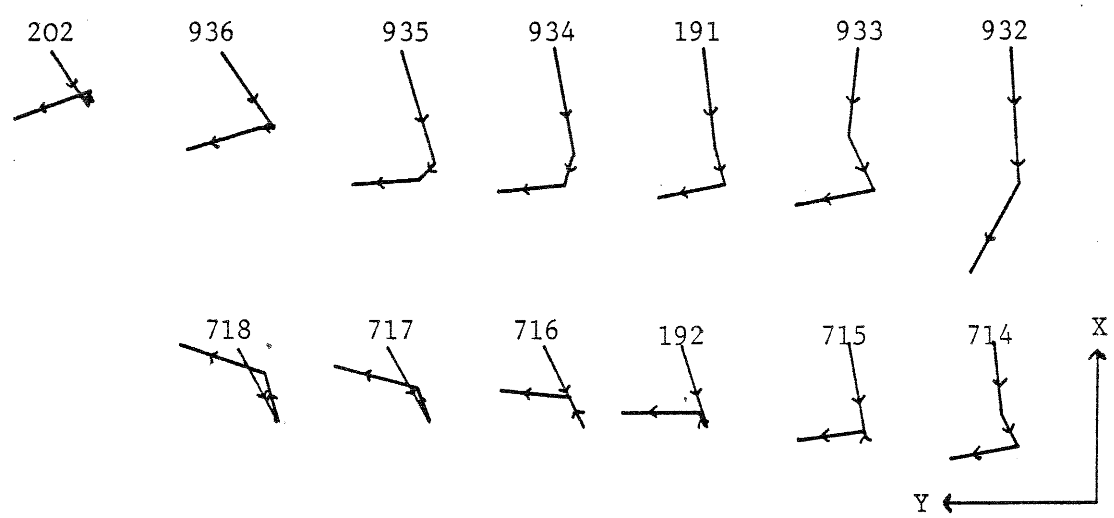


Figure 3. X-Y displacement vectors. Vectors at full size.

Point	Δ X	Δ Y	Δ Z
202	-0.004	0.010	-0.003
936	-0.003	0.010	-0.003
935	-0.001	0.009	-0.003
934	-0.001	0.009	-0.004
191	-0.002	0.009	-0.004
933	-0.002	0.010	-0.003
932	-0.012	0.006	-0.002
714	-0.002	0.009	-0.004
715	-0.001	0.009	-0.004
192	0.000	0.010	-0.004
716	0.001	0.009	-0.004
717	0.003	0.010	-0.005
718	0.004	0.011	-0.006

Table 1. 1980-81 coordinate differences
Original datum held fixed
Units are metres.

Point	Δ X	Δ Y	Δ Z
202	-0.003	0.006	0.001
936	-0.002	0.006	0.001
935	0.000	0.005	0.001
934	0.000	0.005	0.000
191	-0.001	0.005	0.000
933	-0.001	0.006	0.001
932	-0.011	0.002	0.002
714	-0.001	0.005	0.000
715	0.000	0.005	0.000
192	0.001	0.006	0.000
716	0.002	0.005	0.000
717	0.004	0.006	-0.001
718	0.005	0.007	-0.002

Table 2 1980-81 coordinate differences.
Relative datum held fixed
Units are metres.

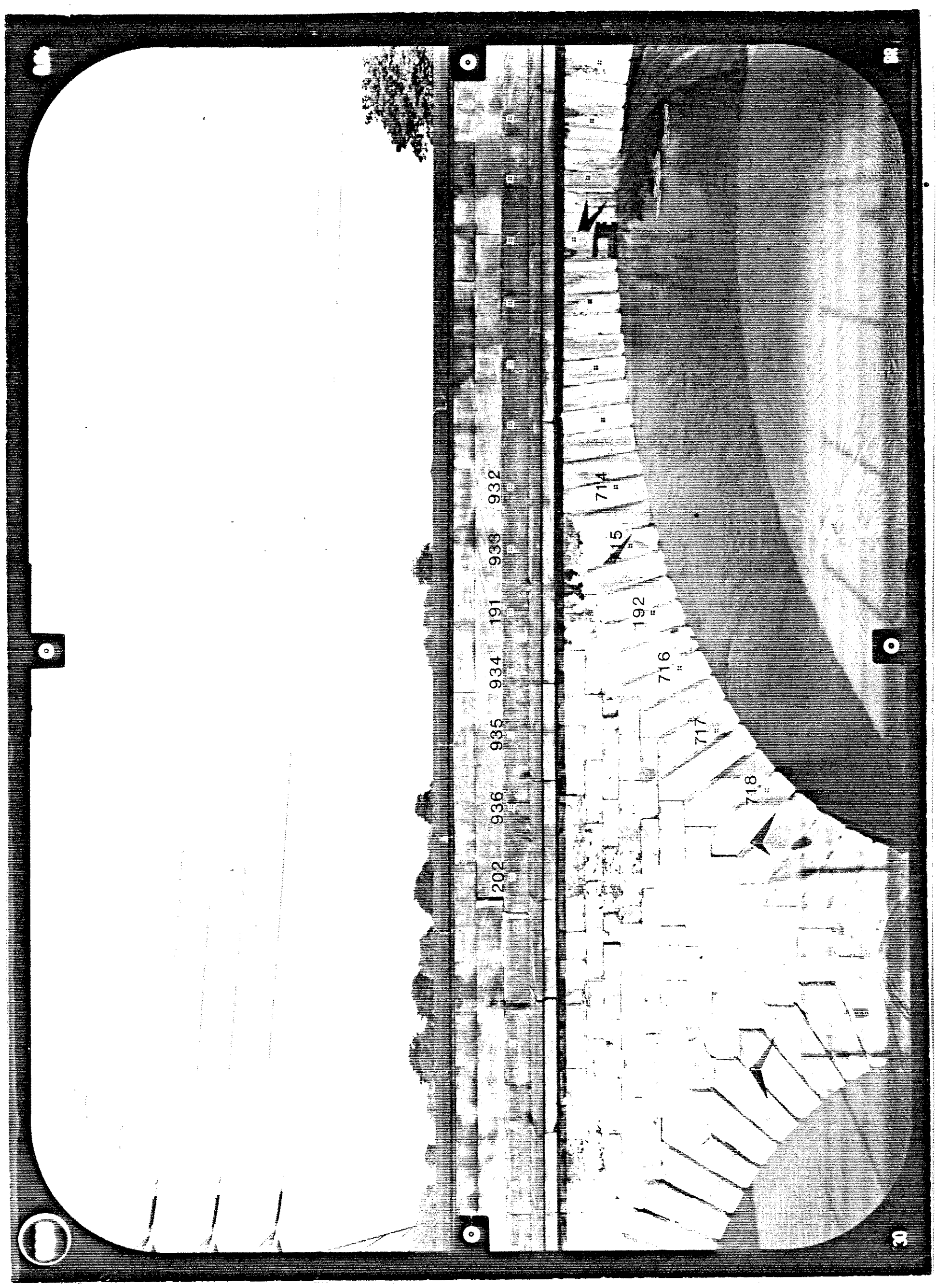


Plate 1

References

Cooper,M.A.R..,1982. The Computation of Ordnance Survey Bute Aerial Triangulation using XYZBLC. The Photogrammetric Record,Vol X No 59 pp 547-558