ALIGNMENT OF CRANE RAILS USING A SURVEY NETWORK

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Abstract

Alignment surveys have been carried out by various means and for a variety of applications for decades. Although optical offset and photogrammetric methods are successfully used in many cases, triangulation using theodolites or total stations is preferred for most alignment surveys carried out by surveyors. This paper describes a novel alignment survey which required 300m long crane rails to be measured to an accuracy of one millimetre using a combination of angular triangulation and distance measurement.

Introduction

In late 1992 the Department of Geomatics was approached by a major Australian manufacturer with a somewhat unusual problem. The problem concerned the excessive and uneven wearing of the wheels of an overhead gantry crane in a main storage and dispatch warehouse. The replacement of the overhead crane wheels is both an expensive and time consuming process, which of course also results in a significant loss of production time.

It was suggested that two primary factors were responsible for the excessive and uneven wear of the crane wheels. First, departures from the standard rail gauge (Figure 1a) would result in wear due to excessive lateral force. Second, height differentials between the rails (Figure 1b) would result in wear due to uneven distribution of weight.

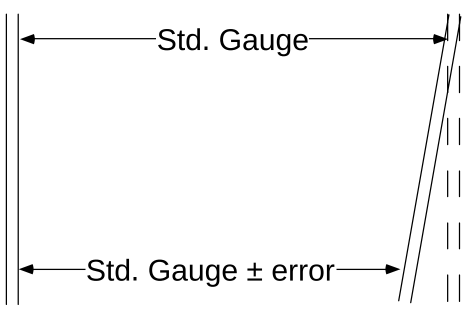

Figure 1a. Incorrect or inconsistent rail gauge.

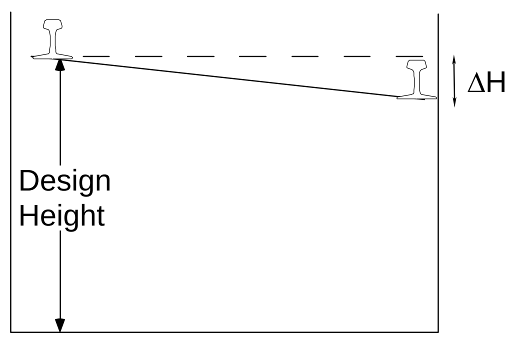

Figure 1b. Height differential between rails.

The survey task here was to measure the straightness and parallelism of the rail centre lines to millimetre accuracy in both the lateral and vertical directions. These measurements were to be taken at approximate 4.5 metre intervals along the length of the rails within a non-production period. The coordinate information gathered was to be represented graphically with particular reference to the lateral and vertical deviations between corresponding rail points. It was envisaged that this information could then be used by engineers on site to evaluate and correct any irregularities present at each of the rail positions.

Site Inspection

To formulate a suitable measurement strategy the first step was to inspect the working environment. The crane rails are approximately 300m long and are mounted about 7m above the ground on steel girders. Supporting uprights are spaced at 9.1m intervals and the separation between the rails is approximately 40m, slightly less than the width of the warehouse.

From a site inspection it is possible to identify and minimise any potential problems, both large and small, during the survey. In this instance it was readily apparent that the floor area was far too cluttered with manufactured product to permit the establishment of instrument stations with unobstructed lines of sight. To counteract this problem it was negotiated that a five to ten metre buffer zone be established along the length of the warehouse prior to commencing the job. In ideal circumstances the warehouse would have been substantially cleared of product, but this was simply not commercially feasible for an area of 1.2 hectares.

Access to the crane rails was to be via two 8 metre ladders to be provided by the client. Although this was likely to be awkward given the amount of product on the warehouse floor, there was no immediate alternative. Possible sites for permanent reference targets were identified. Some were accessible from floor level or using the ladders, but a number of high targets would have to be mounted to ensure clear lines of sight. It was identified on site that some targets would likely require use of the crane platform for a short time prior to the day of the survey.

Also during the inspection it was agreed that the two crane platforms, which would not be operating during the survey, would be parked at the ends of the warehouse to minimise line of sight obstructions. A consequence of this would be that the extreme ends of the rails would be inaccessible, reducing the overall length to 270 metres.

Some preliminary ideas for the design of the survey were discussed on site but no immediate decisions were made. Site plans were supplied by the company to facilitate a more detailed consideration of the design of the survey.

Methodology

Having inspected the site it was necessary to establish a measurement methodology capable of satisfying the accuracy and organisational requirements of the task. The rail position and length obviously raised a number geometric problems that needed to be addressed.

Initially the use of an on-line triangulation system, comprising three total stations and PC based software, was considered. The principles of such systems are very well known (Allan, 1988) and they have gained widespread acceptance for industrial measurement applications (Roberts and Moffitt, 1987; Woodward, 1987). Despite of the elongated geometry, such a system could quite easily satisfy the accuracy requirements and coordinate determination could be provided in real time. The advantage of the online system is that it provides a quality measure of every intersection measurement as it is taken, and measurements can be repeated as needed.

However the online system was deemed unsuitable because it could not satisfy many of the other job requirements. The amount of time needed to establish the system was the primary concern. The establishment of a three instrument system set up requires approximately forty-five minutes and it was estimated that approximately twelve set ups would be necessary to adequately cover the length of the rails. In addition, without additional measurements at each set up the potential for propagation of error from the transfer of a local coordinates between set ups was an issue. The cost associated with the increased time at the site made the use of this technique uncompetitive.

A variation on the online triangulation system was considered as a means of bypassing some of the problems associated with the method. The proposed method would still be instrument based, however each instrument station would be established pseudo-independently. This variation was based primarily on the establishment of multiple instrument stations through resection to a number of common points, followed by intersection observations to the rails. Eight resection targets strategically distributed throughout the warehouse were considered sufficient to ensure that every instrument station would have at least six visible reference points (see Figure 2). Information gathered at each instrument would then be stored in a data recorder. This information could be downloaded and manipulated in a survey network adjustment to obtain station coordinates for each point. Similar approaches have been successfully applied to industrial measurement tasks elsewhere (Shortis, 1992; Stirling et al, 1994) and the technique has been in general use by surveyors and engineers for industrial applications during the last 50 years (Hume, 1970).

The advantage of this approach is that the amount of time required on site would be reduced significantly. The time to observe six to eight radiations to resect each of approximately eighteen stations would be the order of five minutes, as opposed to forty-five minutes for each of twelve online triangulation set ups. The remaining major time components included the initial organisation and the actual measurement of the estimated 120 rail positions. Including other ancillary tasks, the entire project was expected to require ten hours in the absence of any major problems.

The obvious disadvantage of the proposed method is that gross errors in the measurements would not be detected until after the actual survey. The only redress for this problem is to incorporate more measurements into the survey to improve the reliability, and thereby also avoid a situation where a rail position would be determined by a minimum number of measurements. The only feasible approach to adding measurements in this case was to include distances into the survey. Inclusion of distances is not possible with the current revision of the online triangulation system, most certainly because coordinates are much more precisely determined by angles, as compared to distances, over short ranges. However, the inclusion of distances would improve the reliability of the proposed survey network both as a whole and for individual rail locations. The technique adopted is effectively an over-determined survey network composed of horizontal angles, vertical angles and slope distances.

Photogrammetric methods were also considered as a means of satisfying the measurement objectives, as they are widely used for appropriate industrial and engineering applications (Shortis and Fraser, 1991). However, the rail length and separation made the use of any particular photogrammetric method logistically unacceptable, both in terms of the associated cost and the achievable accuracy. Optical alignment techniques were ruled out for similar reasons. The post-processed survey network technique was adopted because it was the most cost-competitive.

Simulation

Having established a methodology it was necessary to verify that the accuracy requirements could be met. This was accomplished through use of a survey network simulation (Cooper, 1987) to design and analyse the survey network. Utilising approximate station positions, resection target locations and rail point coordinates based on the geometry of the survey and the site plan, plus the expected measurements to be taken and the expected precisions of measurements, it is possible to estimate the likely precisions of all coordinates. No actual measurements are required at this stage, the results of the simulation are purely based on the geometry of the network and the types and precisions of the measurements.

In this instance angular and distance information would be gathered using Geodimeter 460 and 4000 total stations. These instruments are servo driven and have an angular resolution of ±1", the best available to the Department. Distance accuracy as specified by the manufacturer is ±(5mm + 5ppm), but experience over short ranges indicates that these values are slightly pessimistic (Stirling et al, 1994). Sokkia SDR33 data recorders were to be utilised to store angular and distance information collected from each station.

The coordinate precisions obtained from the simulation are a very useful diagnostic tool for the design, and re-design, of the survey. Evaluation of the coordinate precisions and diagrams of error ellipses enables the identification of weak points within the network and therefore facilitates the testing of variable geometric strategies prior to implementation (Allman and Hoar, 1973). An iterative re-design process can be used to optimise the number of stations and measurements required to meet the accuracy criteria. In addition, the information can be used to formulate some type of approximate costing through a prior knowledge of the number of stations and measurements needed to complete the task.

Based on a number of simulations using the TDVC survey adjustment software (Shortis and Seager, 1994), the proposed survey network comprised eighteen instrument stations established using eight resection points as shown in Figure 2. The stations were slightly staggered along the centre-line, in order to avoid the possible ambiguity of horizontal angles to the datum points or other stations approaching 180 degrees. The following measurement strategy was proposed :

- three instruments recording horizontal and vertical angles to each rail target point
- instruments would be separated by approximately 15 metres
- each instrument station would be established through resection to eight targets strategically distributed throughout the warehouse
- the inclusion of distances in the measurement process would strengthen the adjustment
- station heights would be “fixed” through precise levelling to strengthen all heights throughout the survey

- a calibrated scale bar would be introduced to strengthen the overall scale of the network

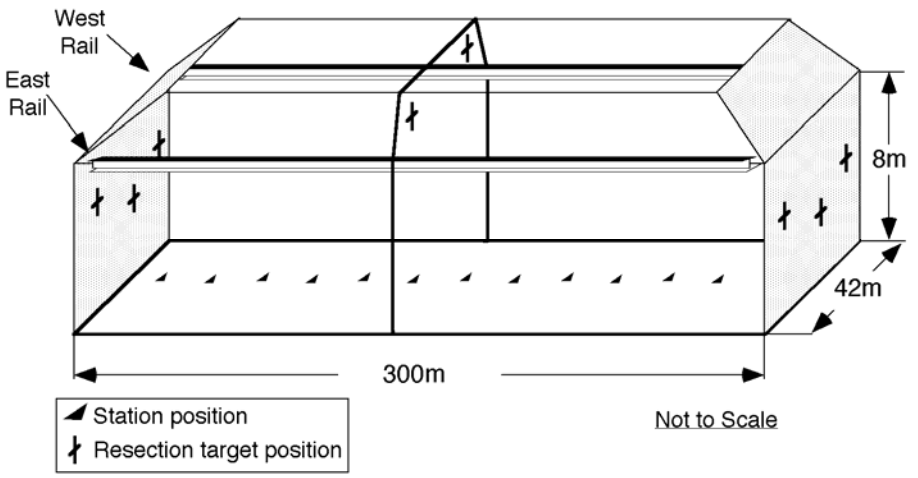

Figure 2. Approximate locations of the instrument stations, resection targets and rails within the warehouse.

From the outset it was apparent that the rail position above ground level and the physical length of the rails would create a number of geometric problems. Figure 3 shows a small section of the network design and it is clear from the shape and orientation of the error ellipses that there is an unavoidable bias due to the elongated nature of the survey network. There is no effective and cost-competitive solution to this problem.

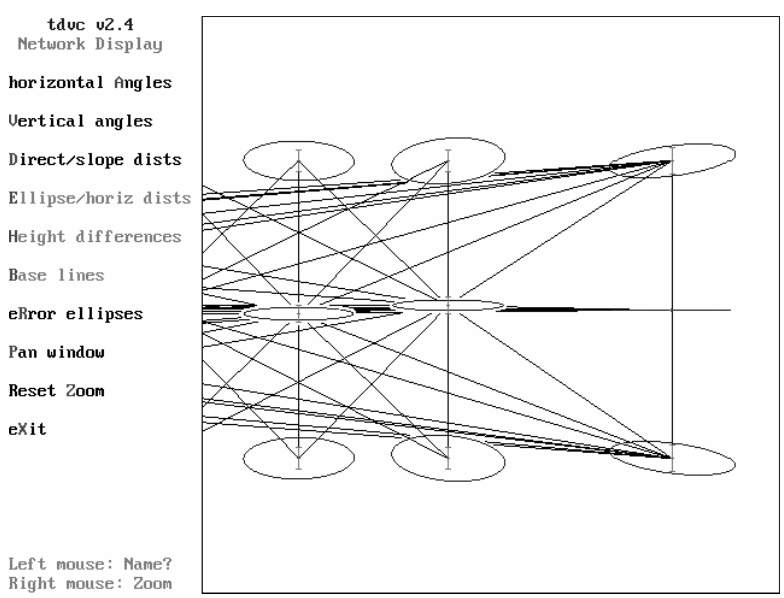

Figure 3. Network simulation diagram of a small section of the survey (edited plan view, error ellipses indicate X-Y precision, vertical bars indicate Z precision).

However, using an angular precision of ±2" and the specified distance precision from the manufacturer, the lateral and vertical coordinate precisions of the rail locations were ±0.5 and ±0.3 millimetres respectively. The longitudinal coordinate precisions of the rail locations were as large as ±1.5 millimetres, but this coordinate was not critical. As could be expected, altering the distance precision to ±3 millimetres improved the overall reliability and, in particular, the non-critical longitudinal coordinate precision improved to ±1.1 millimetres.

More importantly, reducing some of the rail locations to only two radiations substantially degraded the overall reliability and reduced the coordinate precisions to ±(1.8 to 2.3), ±(0.7 to 1.2), ±(0.3 to 0.5) millimetres in the longitudinal, lateral and vertical directions respectively. The variations are due to the different geometries generated by the relative positions of the rail locations and the instrument stations (see Figure 3). Although the results for the critical coordinates are still acceptable with respect to the nominal ±1 millimetre specification, it is clear that three radiations to each rail location would be very desirable to guarantee the required precision.

Targeting

In order to complete measurement to the rail centre line it was necessary to design and build two different types of targets. The most critical of these were the two identical rail targets, whilst the central resection targets (see Figure 2) were of somewhat lesser importance. Both of these types of targets required a purpose built EDM reflector and telescope alignment cross-hair combination so that a single, rapid, precise pointing would be possible from close range. Standard prism reflector targets were used for the resection targets near the ends of the warehouse (see Figure 2) as they would be sighted over longer ranges and from the same general direction.

There were two primary constraints which governed the design of the rail targets, namely the height of the rails above the instrument stations and the need to accurately define the rail centre-line. In order to be seen from each of the instrument stations it was necessary to design a target with an offset so that it could be seen clearly above the rail and the rail support structure. As measurements were to be taken to the rail centre-line it was very important to ensure that this line was accurately defined.

Symmetrical targets were designed (Figure 4) to precisely straddle the rails at a constant vertical offset above the rail centre-line. Metal shims were inserted at the base of the rail to force the target centring at each measurement location. The target design incorporated an engraved, colour contrasting cross for theodolite pointing and a circle of retro-reflective tape to return the EDM signal.

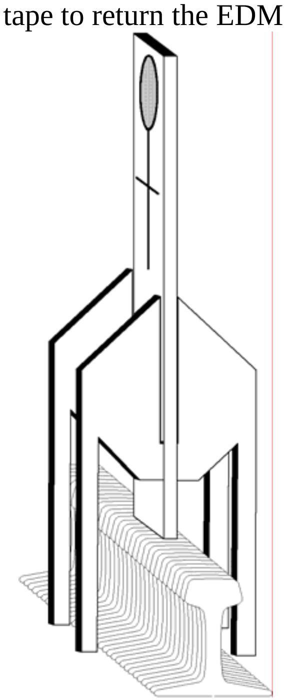

Figure 4. Rail target design.

The cross and circle centre separation distance was precisely matched against the separation distance between the telescope and EDM of the Geodimeter total stations (Figure 5). As the target offset distance was identical and constant for each of the rail targets all of the points could be reduced to a common datum. Longitudinal position was not critical and as such could be approximately established through reference to the crane rail supports.

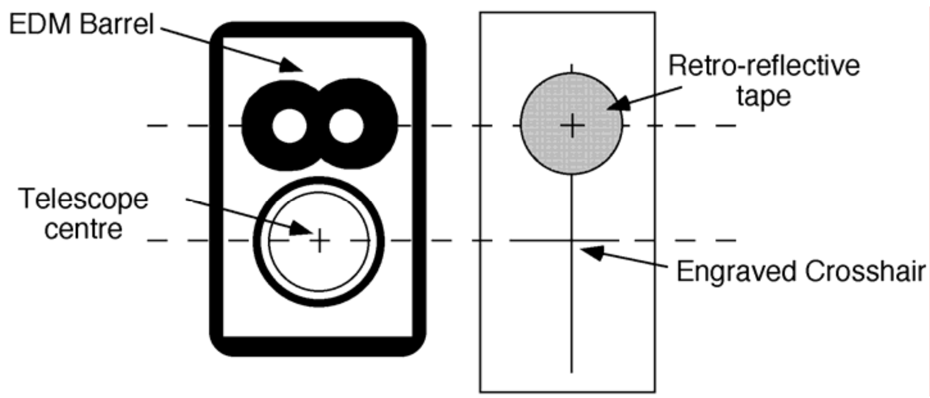

Figure 5. Resection target design.

The central resection targets would be sighted from opposite directions and therefore had to be double sided. These targets would be located on the overhead beams so that they would be visible from all instrument stations. Thin sheets of aluminium were drilled at the correct separation and the alignment cross and circle of reflector tape on each side centred on the drill holes. It was foreseen that the instrument stations near the centre of the warehouse would have a very oblique view of these targets because the faces would be directed to the ends of the warehouse. However again there was no obvious alternative considering that the targets, once clamped to the overhead beams using the crane platform to access the locations, would be inaccessible during the survey.

Retro-reflective tape is an effective substitute for the return of EDM signals over short ranges, however there can be a degradation in distance precision and potential for variation in the reflector constant with distance (Rüeger, 1990). Due to the possible degradation it was decided to adopt the specified distance precision from the manufacturer of the total stations. Empirical testing showed that the variation in the constant was negligible over the expected range of distances.

Measurement

There were a number of preliminary tasks that needed to be completed prior to the commencement of the main survey measurement task :

- establish datum points at each end of the warehouse and an approximate centre-line axis
- establish the instrument station locations as determined from the simulation but with regard to line of sight obstructions
- locate the resection targets around the warehouse
- determine instrument station heights through precise levelling

The precise levelling observations were reduced on site and the 600 metre run closed to 1.2 millimetres.

The main survey task was then begun with the initial three instrument stations. The routine shown in Figure 6 was executed to complete the measurement process. Approximately six positions on each rail were observed from each set up of three instruments. Inevitably some rail locations could only be observed from two of the three stations due to foreground obstructions. A one metre precise scale bar was measured from the initial three stations and the last three stations, in order to enhance the overall scale determination for the network.

Two serious problems emerged once the survey was commenced. The most threatening of these was the unexpected difficulty of accurately sighting to resection targets over distances greater than approximately 200 metres. The problems were primarily caused by poor lighting conditions, a heat haze due to high ambient temperatures, temperature variations caused by air conditioning units and various line of sight obstructions. This resulted in a progressive change in resection targets used as the survey station occupations moved along the centre-line, with the most distant resection targets often being unusable. The loss of data certainly weakened the overall network, but in all cases at least five resection targets were available to independently fix each instrument, as well as the influence of the common measurements to rail positions.

The second problem was that the time taken to move targets from point to point far exceeded the initial estimates. It was envisaged that two 8 metre ladders would used to reach the rails and locate the targets. However, the point to point measurement time of this routine was in excess of ten minutes because of the positioning and manipulation of the ladders. It became apparent that the time constraints for the measurement could not be met without speeding up the target relocation process. To this end two members of the team elected to sit on the rails and slide the targets between points. This effectively reduced the point to point measurement time to a function of the operator pointing speed, but was literally a pain in the posterior for the rail sitters.

Overall, the time taken to complete the collection of all 120 rail points was approximately twelve hours, of which one hour could be assigned to the resolution of on-site problems. The time taken was not significantly greater than that anticipated. This was not by chance but through good initial planning and on-site improvisation.

Measurement Processing and Analysis

The first phase of the measurement processing was the downloading of the field data from the SDR33 recorders. The field data was then converted to TDVC compatible format using a utility program known as SDR2TDVC (Shortis, 1994). This program computes means of different face pointings for all observed horizontal directions for an instrument station and then reduces the mean directions based on the mean direction to the reference object adopted for each instrument station. The program also computes means of different face pointings for vertical angles and mean distances where multiple observations have been taken.

The second phase was a merge of all the instrument station files and the estimation of approximate locations for all stations, resection marks and rail positions. The levels, or vertical coordinates, of the instrument stations were already known from the precise levelling. Using the site plan and the known geometry of the survey, all other coordinates were visually estimated to sufficient accuracy for TDVC to compute the adjustment of the network.

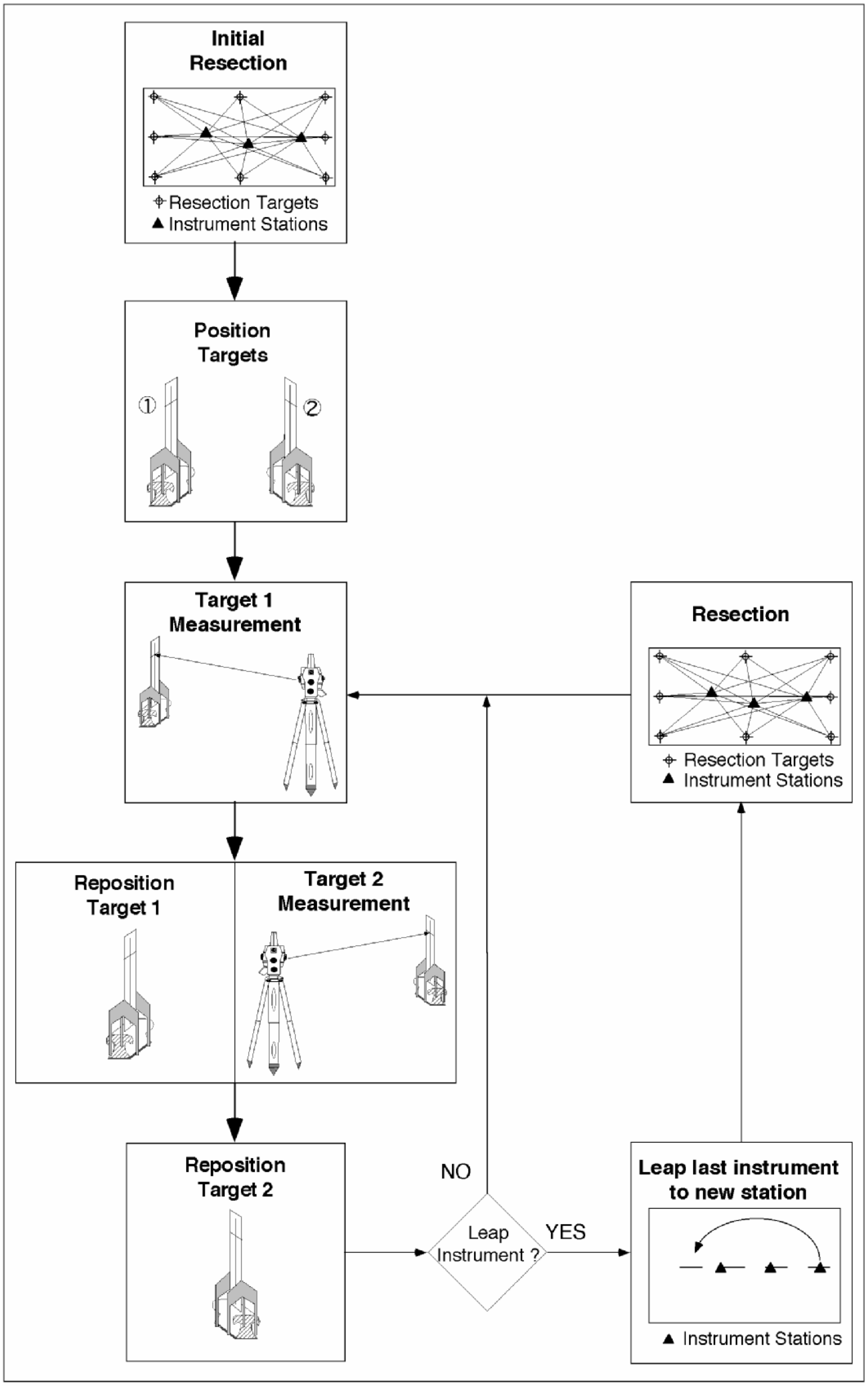

Figure 6. Flow chart of the main survey task

The third phase of the measurement processing should have been the survey network adjustment using TDVC. However this proved to be both unwieldy and cumbersome because of the amount of data involved. In the full adjustment of the network there were 150 positions determined using almost 1400 measurements. Although most of the difficulties could have been overcome, the process of gross error removal became the critical issue which forced this approach to be abandoned.

As a consequence, the survey network was broken down into two groups of data. The first group contained only resection measurements, resection targets, instrument stations and scale bar measurements. This data set comprised only 30 stations and 400 measurements. The network adjustment was processed with few problems or rejections of data.

The second group of data comprised all stations, all resection targets, all rail locations and all measurements. However, to reduce the complexity of the task, all the instrument station and resection target locations were held fixed in the adjustment. Considerably more processing was required to reach a satisfactory network adjustment. Some 45 observations were deleted from the measurement set due to suspected gross errors, as quantified by a Student-T test statistic at a 95% confidence limit (Shortis, 1994).

In the majority of cases a full radiation, composed of two angles and a distance, was deleted, indicating that something was amiss with the entire observation. Most of these gross errors could be attributed to poor atmospherics along the line of sight, grazing lines of sight across foreground obstructions or high angles of incidence to the targets. The remainder of the deleted measurements were primarily distances, and it could be assumed that poor signal from long lines of sight or high incidence angles, combined with anomalous refractive indices caused by unexpected variations in the atmospherics, were to blame.

The final step in the measurement processing was to compute the full network with the refined set of measurements and free instrument stations. No further data rejections were necessary. The average precision of the rail locations was ±(2.2,0.5,0.3) in the longitudinal, lateral and vertical coordinates respectively, which is commensurate with the simulated design. The evident worsening of the precisions of the longitudinal coordinates is due to the decrease in the number of longer distance measurements to resection targets. As predicted by the simulations, the poorest results occurred for rail locations with only two radiations from instrument stations, where the precision of the lateral coordinate was as poor as ±0.8 millimetres. However, all results were within the specified nominal precision of ±1 millimetre for the critical coordinates.

Contrary to previous experience over short ranges, the average correction to the distance measurements was in accord with the specified precision stated by the manufacturer. This result can be explained by the combination of a mixture of ranges in the distances, the adverse atmospheric conditions and perhaps a degradation due to the retro-reflective targets used. There were many horizontal and vertical angles which were adjusted by more than ten seconds of arc within the network. The vast majority of these large corrections were associated with very short lines of sight where the pointing error was the dominant factor, or with the longest lines of sight where poor atmospheric conditions were an influence.

Data Analysis and Presentation

Further processing of the coordinate data was necessary to convert the results into an easily interpreted format. Essentially, displacements from the standard gauge and differences in level between the two rails had to be presented in both numerical and graphical form.

The first step in this process was to reformat the output coordinate file from TDVC into a plain text file. This file was then imported into the PC version of the Microsoft Excel spreadsheet program.

Spreadsheet functions were then used to perform the following preliminary calculations:

- compute the mean horizontal alignment of each rail
- realign the coordinate system to the mean horizontal alignment of the two rails
- compute the mean height of all rail positions

The following values were then computed at each pair of rail location positions :

- the lateral deviation of each rail with respect to the mean position of the rail
- the deviation from the standard gauge
- the vertical deviation of each rail from the mean rail height
- the difference in height between the two rails
- plot sheet coordinates of rail deviations

The largest deviations from the standard gauge were +21 and -14 millimetres whilst the largest height differences were +12 and -10 millimetres. The average gauge was some 6 millimetres greater than standard. Deviation values greater than 5 millimetres were deemed to be worthy of remedial action, which would have affected more than 50% of the rail supports.

The computed plot sheet data was then exported from the spreadsheet as plain text files. A small PC program was written, tested and applied to convert the positions and deviations into a Microstation CAD user command file which generated line strings corresponding to the deviations. After suitable additions to the design files to clarify the purpose of the line strings, a number of CAD plots were produced to clearly indicate the rail deviations. The horizontal alignment deviations are shown in Figure 7 and the vertical deviations are shown in Figure 8.

A number of other CAD plots with greater amounts of detail were also produced, including one sheet showing numerical values of deviations overlaid on the graphical representation. Numerical data from the spreadsheets was also transmitted to the client to provide an unambiguous representation of the deviations.

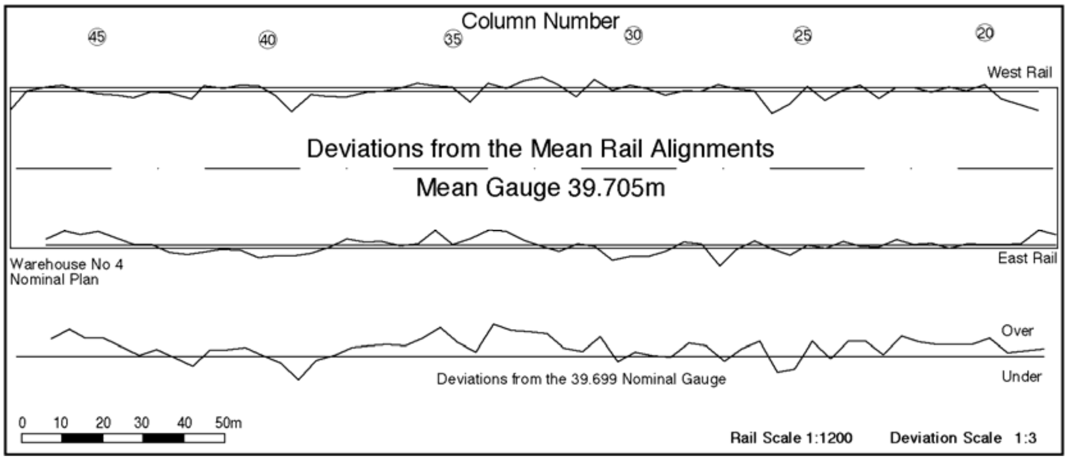

Figure 7. Gauge deviation diagram.

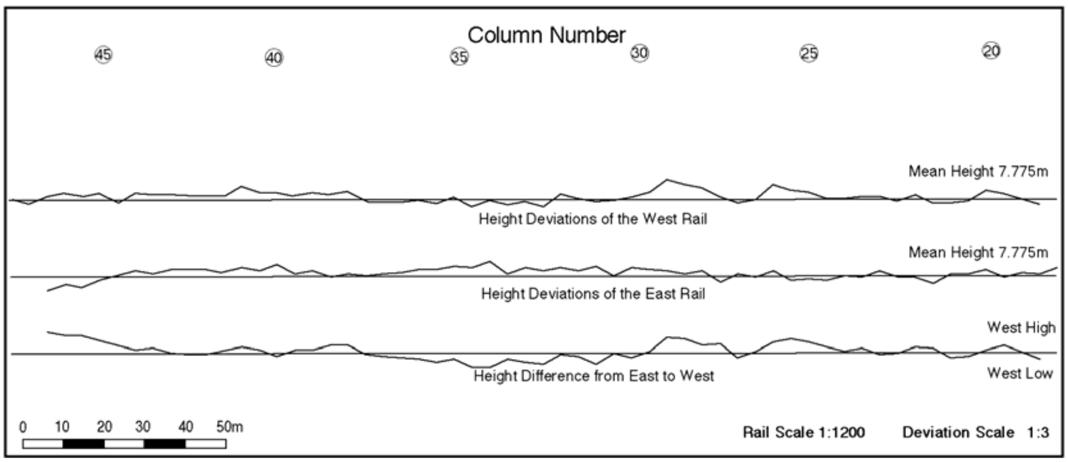

Figure 8. Height deviation diagram.

Conclusion

An over-determined survey network proved to be a cost-competitive and sufficiently accurate method of measuring the alignment of the crane rails. A number of major and minor problems were overcome on-site, but in general the process and results of the survey were as expected from a considerable amount of planning and network simulation.

The results of the survey clearly show significant deviations in gauge, relative height of the rails and straightness of the rails, which were at least partly responsible for the wear of the crane platform wheels. The results of the survey were used to adjust the rails, using shims, at the support points with the largest deviations from the standard gauge and straightness of the rails. Other remedial action was also taken, such as different wheel tread profiles, introduction of long travel drive bias to control slewing of the cranes and the addition of constant lubrication to the wheel flanges. As a consequence of this combination of actions, in 1994 the rate of wheel wear has been reduced to an insignificant level.

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