Close range Photogrammetry for the Structural Monitoring of the Star Ferry Colonnade

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Abstract

Close range photogrammetry was used to monitor the colonnade of the Hong Kong Cultural and Arts Centre adjacent to the Star Ferry pier in Tsim Sha Tsui, Hong Kong. A consumer-level digital camera was used to capture convergent imagery for four epochs of measurement. At the same time conventional intersection surveys were made of the structure. Photogrammetric measurement of 57 targeted points was made with Elcovision 10 software. Statistical analysis of the photogrammetrically derived coordinates showed, at the 95% level of confidence, that the structure was stable. Comparison with the intersection survey solution showed that the photogrammetric solution produced coordinates of similar accuracy and was up to 50% cheaper to implement.

1 Introduction

Each year Hong Kong is the target for numerous tropical storms and typhoons. These events deposit large volumes of water over the whole of the territory in a short period of time. In 2001 a peak rainfall intensity of 330mm per hour was recorded. During the year a total of 3,100mm of rain was recorded (average of 2,200mm), 85% (2,620mm) of which fell during the months between June and September. Large areas of steep hills surrounded by flat coastal margins dominate the topography of Hong Kong. When it rains, a high proportion of that falling on the hills quickly finds its way to the flat areas causing flooding in many areas of the territory. A policy of land reclamation from the sea has extended the flat coastal area that has exacerbated drainage problems in, amongst others, the Kowloon area. The Drainage Services Department (DSD) of the Hong Kong Special Administrative Region (HKSAR) Government has been charged with devising a solution to the flooding and drainage problems of the territory. Details of the scheme can be found on the Internet at http://www.info.gov.hk/dsd/flood/problem.htm.

To combat the urban flooding problem in West Kowloon, DSD has a plan to carry out an extensive drainage upgrading works, including a special transfer scheme (Kai Tak Transfer Scheme and Tai Hang Tung Storage Scheme). Stage 1, Phase 2 of the West Kowloon Drainage Improvement Project is to relieve the problem in the West Kowloon area between Kowloon Tong and Tsim Sha Tsui. Just one small part of this project is the installation of a 2.5 m diameter pipe adjacent to the colonnade of the Hong Kong Arts and Cultural Centre near the Star Ferry pier in Tsim Sha Tsui. The pipes will discharge storm water into Victoria Harbour.

The colonnade is a semi-circular structure 8 metres tall and 9 metres in diameter. The pipes are to be laid in an open 4 metre wide, 6.5 metre deep trench that approaches to within 1.5 metres of the colonnade. During excavation and pipe laying the walls of the trench are to be shored with sheet piling. The colonnade and trench prior to excavation is shown in Figure 1. Project engineers were concerned with the stability of the colonnade as it came under the influence of ground settlement due to dewatering caused by the excavation as shown in Figure 2. Due to the public nature of the colonnade, the consulting engineers required that the structure be monitored for settlement during the process of the pipe laying work. This paper describes the photogrammetric work done to fulfil the dual aims of a) monitoring the structure and b) evaluating its competitiveness on the basis of accuracy and cost efficiency with respect to conventional survey techniques.

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2 Photogrammetric design

Figure 1. The Colonnade at Star Ferry with the trench in the foreground

The specified accuracy for the deformation survey was each point coordinate (x, y and z) should be within 2mm of its actual position. To make photogrammetry a viable technology option to undertake the monitoring, it was necessary to use a digital camera (Fujifilm MX2700) as the project was to be undertaken "in-house" where no precision film scanner was available. This camera has a 7.6mm focal length lens and a 7.2mm x 4.8mm, 1,800 x 1,200 pixel CCD giving a pixel size of 4 µm. Assuming a manual measurement accuracy of 0.5 pixel, this represents an angular measurement accuracy of 57 seconds of arc. An object distance of 10.5m allowed the height of the colonnade to fit the small dimension of the image giving an image scale of 1:1,400. Turning the camera 90 degrees to increase the imaging scale was not an option as the maximum horizontal extent of the colonnade was greater than its height. Figures 3a and 3b show the two sides of the colonnade.

2.1 Targetting

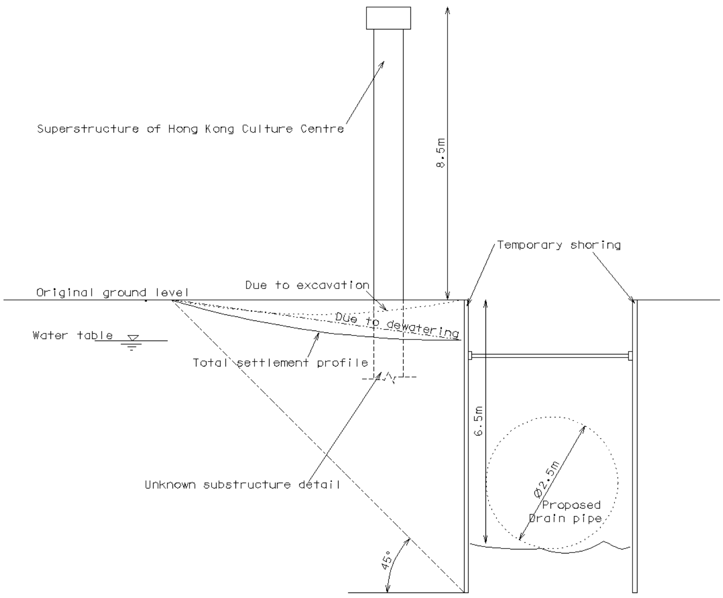

Figure 2. Stability effects due to excavation and dewatering

Two types of targets were used – monitoring point targets and control point targets. These were made from 3M 7610 retro-reflective sheeting and Leica retro-reflective targets respectively. A target size of 30mm produced an image of just over 5 pixels and was considered satisfactory for the processing software. Each 7610 target was over-printed with a black cross hair mask. The retro-reflective targets proved to be very durable and also "invisible" to the public. Over the period of the project no targets were lost but from time to time some targets were obscured by plant and equipment. A total of 50 monitoring points and 20 control points were placed on the object as shown in Figure 4. Seven of the 20 control target points were used as check-points for the accuracy assessment.

Figure 3. The colonnade. (a). Left side and (b). Right side of colonnade

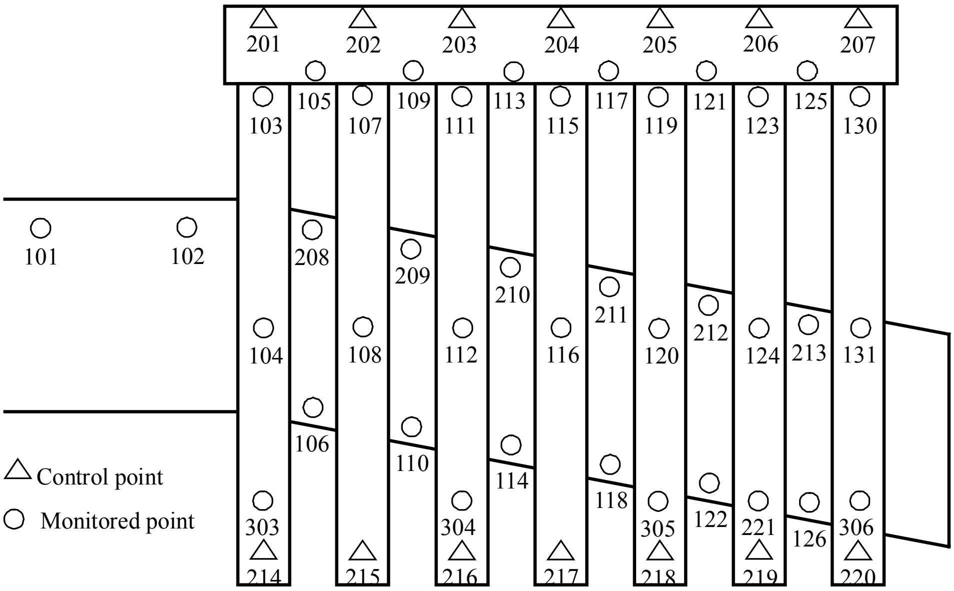

Figure 4. Monitoring and control point locations

2.2 Control network

The control network provided two functions in this project. Firstly it provided control point coordinates for the photogrammetry and secondly an independent check on the selected monitoring points. Physical constrains at the site required careful planning to ensure that a network of suitable configuration was established. In total 9 ground stations were established as shown in Figure 5.

All observations of the control network were made with a Leica TC1800 total station. The instrument was calibrated for horizontal and vertical collimation so that, in conjunction with the dual axis compensator, single face observations could be made. The EDME component of the instrument was also calibrated and zero, scale and cyclic error applied at the time of measurement. Three arcs of observations were made to each point in the network with distances also being measured between all network points. All observations were reduced using STARNET software with all 3-D coordinates passing the chi square test at the 95% level of confidence. The R.M.S. error of all point coordinates was 1 millimetre after the least squares adjustment.

2.3 Imaging geometry

The semi-circular shape of the colonnade dictated that a ring-based method of image acquisition be adopted. Such a configuration implies convergent imaging geometry however care had to be taken in considering the micro imaging geometry. Initial thoughts were to use a two level approach - a lower level taken with the camera close to the ground and a second level taken with the camera elevated by standing on a 3 metre ladder. This approach had to be abandoned shortly after the initial design was made as a hording (visible in Figure 3b.) was placed around the structure.

An initial estimate of the achievable precision was made using Equation (1) (Fraser, 1996):

$$\bar{\sigma}_c = \frac{dq\sigma_a}{\sqrt{k}} \tag{1}$$

with $\mathbf{d}$ = 10.5 m, $\mathbf{q}$ = 0.7, $\mathbf{k}$ = 1 and σ_a = 57", $\bar{\sigma}_c$ = 0.002 mm, thus satisfying the engineering requirements. Using $\mathbf{k}$ = 3 gives $\bar{\sigma}_c$ = 0.001 mm.

Based on a second set of criteria ($\mathbf{k}$ = 3), a network of 11 camera stations was designed and is illustrated in Figure 6. The purpose of the single station at the rear of the colonnade was to close the loop and reinforce the connection between ends of the structure. At each camera station three images were taken, one normal to the structure and one at angles ±30° to the normal. The result was a set of 33 images with 80% overlap. A trial of this configuration was undertaken with poor results. It was apparent that the camera station at the rear could not sufficiently tie the two sides of the structure together. As there was no room to place additional camera stations at the rear, additional stations were placed in front of the colonnade as shown in Figure 7. With the greater density of camera stations only one image instead of three at each station was taken.

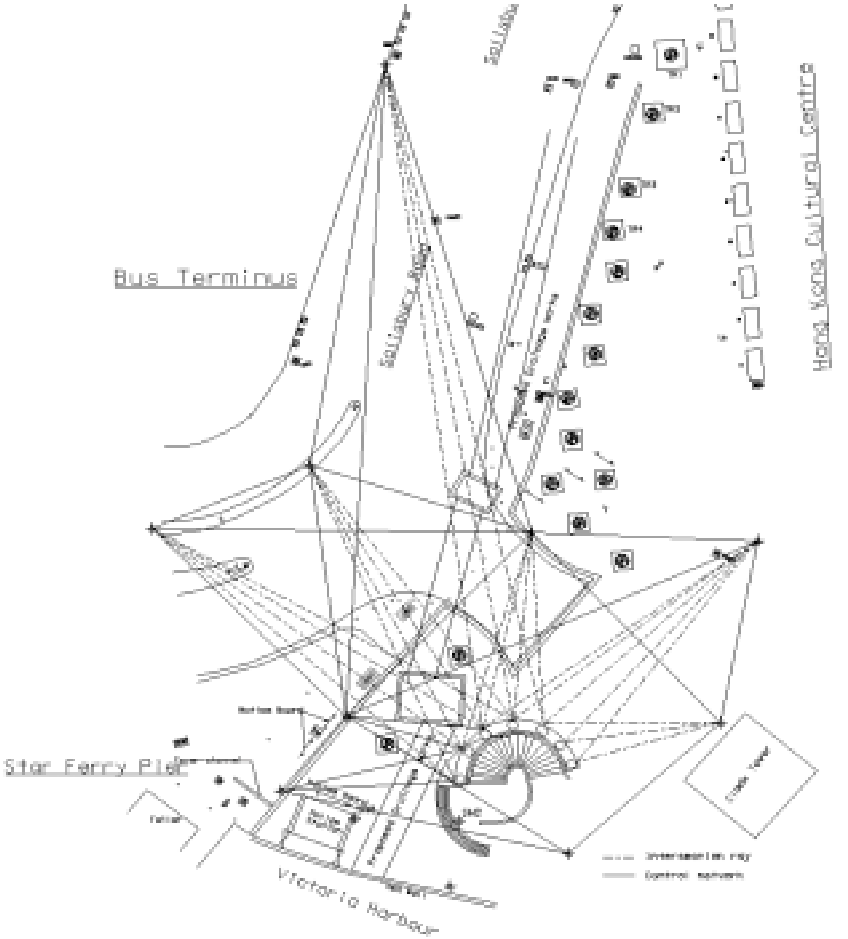

Figure 5. Control network configuration

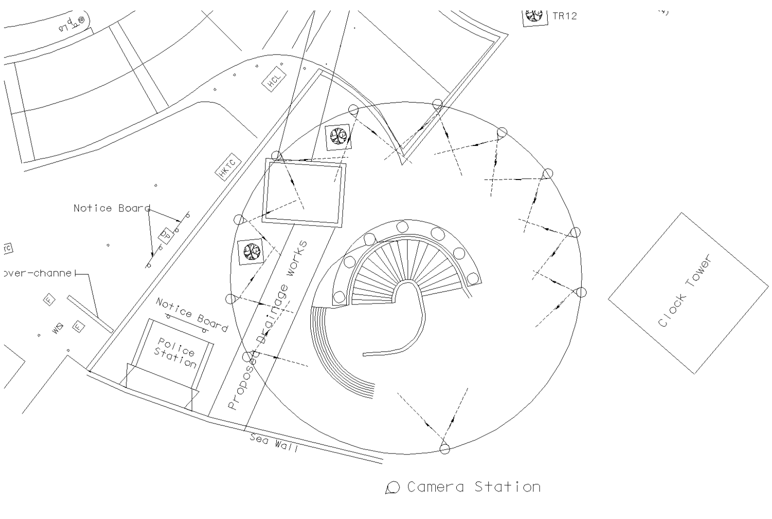

Figure 6. Initial photogrammetric network design

3 Data reduction

A total of 4 epochs of photogrammetric measurements were completed on 14/08/01, 20/08/01, 31/08/01 and 12/10/01. All images were measured using Elcovision 10 software. Targets were manually measured; there is no automated sub-pixel measurement function in the Elcovision 10 software. The reduction procedure was to first performing relative orientation between pairs of images and then an absolute orientation by bundle adjustment of all image pairs. Due to variations in the working conditions at the site, the imaging geometries were not identical for each epoch. A summary of the photogrammetric measurement results for the four epochs is given in Table 1.

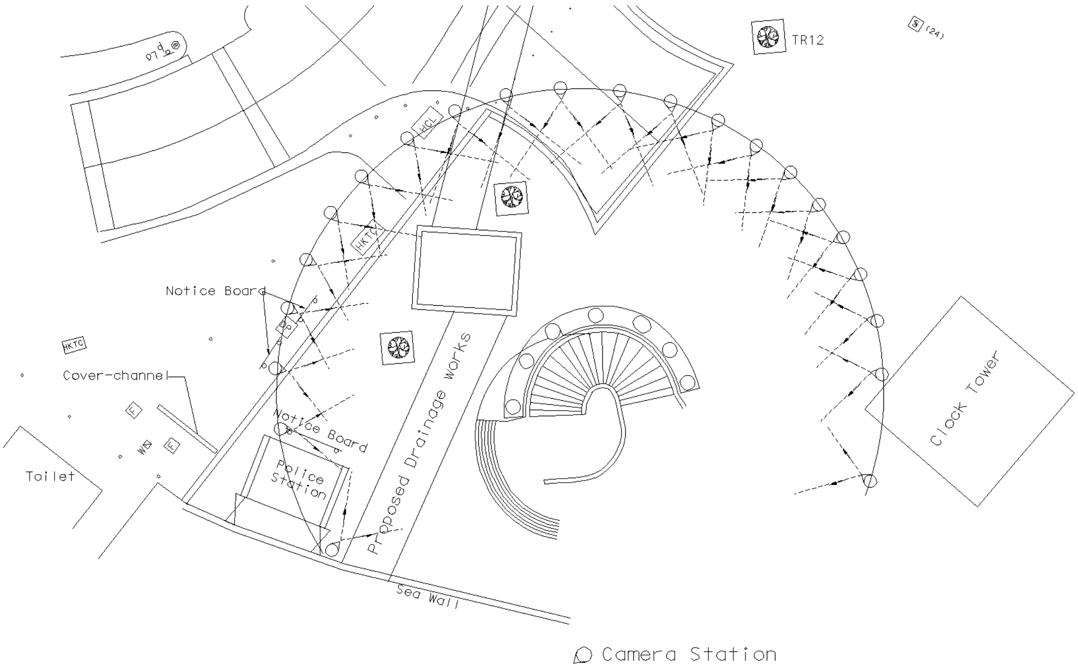

Figure 7. Final photogrammetric network design

Table 1. Summary of the photogrammetric results.

	Epoch I	Epoch II	Epoch III	Epoch IV
Number of Photographs	17	19	21	26
Number of targets for measurement	56	51	56	57
Average number of images per point	4	4	4	5
Degrees of freedom	299	481	493	541
Number of iterations for solution	7	5	6	6
RMS image measurement (μm)	1.0	1.0	1.1	1.3
RMS control x (mm)	0.13	0.16	0.22	0.34
RMS control y (mm)	0.17	0.22	0.31	0.34
RMS control z (mm)	0.14	0.19	0.23	0.44
RMS control position (mm)	0.26	0.33	0.44	0.65
Mean standard deviation x (mm)	1.67	1.71	1.43	1.70
Mean standard deviation y (mm)	1.62	1.85	1.53	1.60
Mean standard deviation z (mm)	0.86	0.81	0.78	0.76
Mean standard deviation pos (mm)	2.53	2.68	2.35	2.55

The significant increase in the number of images used in Epoch IV was due to the unfavourable re-location of a piece of machinery necessitating the extra images to tie one side of the structure to the other. Despite the increased redundancy the extra images provided, there was no compatible increase in the accuracy or precision of the measured points. Indeed, it can be seen that the image measurement accuracy declined by 30% compared to that of the previous epochs. Inspection of the image coordinate residuals did not reveal any outliers in the digitising of image points but rather an overall lowering of digitising precision which has been put down to "familiarity" with using the photogrammetric system. That is, the operation of manual measurement had become tedious rather than challenging.

3.1 Internal accuracy

Internal accuracy was assessed by comparison of the RMS errors of the control points. It indicates that the photogrammetric network can be constrained through the use of redundant control to produce control point positional accuracy for Epochs I to III of less than 0.5mm in each coordinate direction. The effects of the less favourable imaging geometry for Epoch IV are again obvious through its larger RMS error. The precision of each point's coordinates was consistent between each Epoch. The mean standard deviation of position statistic shows that the estimated precision for k = 1 (2mm) was overoptimistic by about 25%. However, as the engineering specification called for each coordinate being within 2mm of its true position, the internal results were deemed to be satisfactory.

3.2 External accuracy

In Hong Kong, intersection survey is widely used as the preferred measuring system for deformation monitoring as it is strongly believed that it can yield higher accuracy in relative positioning of monitored points than other methods. A simple check was carried out to test the applicability of this assertion against the photogrammetric method before performing deformation analysis. A comparison of the coordinates produced by the intersection and photogrammetric measuring systems was made for this purpose. Monitoring points 208, 209, 212 and 213 were chosen as check points for the comparison. Table 2 summarises the coordinates difference of each point at Epochs I, II, III and IV.

Table 2. Coordinates differences for check points.

Monitoring point no.	Coordinate differences (mm) (photogrammetry – intersection)		
	ΔE	ΔN	ΔZ
Epoch I			
208	+2	-1	-2
209	+1	-6	+1
212	+4	-5	-2
213	+3	-1	-3
Epoch II			
208	+3	-5	-1
209	+2	-5	+1
212	+6	-3	-4
213	+4	+4	-4
Epoch III			
208	-5	+4	0
209	+3	-2	+1
212	+3	0	-3
213	+4	0	+4
Epoch IV			
208	-2	-4	-4
209	+1	-4	-4
212	+3	-1	-4
213	+5	-1	-4

Initial perusal of these results was not very encouraging. However it was realised that the average precision of the surveyed points was 2.1mm and so a total difference of 4mm (2mm for the survey coordinate and 2mm for the photogrammetric coordinate) was deemed satisfactory. These results showed that 39 out of the 48 coordinates differences were within the range. As the engineer demanded an accuracy of 2mm for each of the X, Y, Z coordinates of the monitoring points, the measurements produced by photogrammetry system were deemed to have fulfilled the accuracy requirement. For those points with a coordinate difference greater than 4mm, it was found that in all cases it was due to the manual target measurement process of the Elcovision software. If digital photogrammetry is to be applied to similar projects, it is recommended that software systems providing good quality, sub-pixel target centroiding be employed.

4 Deformation analysis

Two levels of statistical testing were done to assess the stability of the colonnade - global congruency testing and single point movement. The purpose of the former was to test for overall movement of the structure and that of the latter to test if points had moved in isolation. The aim of the deformation monitoring is to determine whether the monitored object has deformed or not. Therefore, the deformation analysis is based on statistical testing to ensure the deformation occurrence is not the result of random or systematic errors in the monitoring network. Details of the development of deformation analysis as carried out in this project are described in Biacs (1989).

4.1 Global congruency testing

The adopted procedure of statistical testing was:

a) Estimation of the coefficients of determination models and their covariance using all available information.
b) Perform the calculation of the test statistic:

$$\mathbf{T} = \frac{\mathbf{q_\Delta} / \mathbf{f_\Delta}}{\mathbf{s_0}^2} \approx \mathbf{F(f_\Delta, f, \alpha)} \tag{2}$$

where $\mathbf{\Delta}$ = Coordinates difference between two epochs; $\mathbf{q_\Delta}$ = the quadratic form of residuals $\mathbf{\Delta^T (Q_\Delta)^{-1} \Delta}$; $\mathbf{f_\Delta}$ = number of coordinate values in $\mathbf{\Delta}$; $\mathbf{Q_\Delta}$ = Sum of the coordinate variance-covariance matrices of the two epochs; $\mathbf{s_0}^2$ = common variance factor; $\mathbf{f}$ = total degrees of freedom of both epochs; α = level of significance; and $\mathbf{F(f_\Delta, f, \alpha)}$ = Fisher-distribution.

c) Compare the **T** and **F** values to test the global movement from the F-distribution.

If **T** > **F**, deformation is significant at the chosen level of confidence otherwise the object being monitored is considered to be stable (**T** ≤ **F**.).

4.2 Single point movement

Even if the global congruency testing indicates that, overall, the object is stable, it is still necessary to check individual points for possible movement. The simplest method to test for significant localized point deformation uses the displacement vector as follows:

$$\begin{aligned} \mathbf{T} &= \frac{\mathbf{ds}}{\mathbf{s_0}\sqrt{\mathbf{R^T Q_d R}}} \approx \mathbf{F(1, f, \alpha)} \\ \mathbf{R^T} &= \left[\mathbf{d_x}/\mathbf{d_s}, \mathbf{d_y}/\mathbf{d_s}, \mathbf{d_z}/\mathbf{d_s} \right] \\ \mathbf{ds} &= \sqrt{\mathbf{d_x}^2 + \mathbf{d_y}^2 + \mathbf{d_z}^2} \end{aligned} \tag{3}$$

where $\mathbf{Q_d}$ = submatrix of point coordinate difference variance-covariance matrix; **f** = total degrees of freedom; α = level of significance; $\mathbf{s_0}^2$ = common variance factor; $\mathbf{F(f_\Delta, f, \alpha)}$ = Fisher-distribution.

If for a point **T** > **F**, the null hypothesis (the point hasn't moved between epochs) is rejected which means the movement of the individual point is significant.

4.3 Results

Unfortunately, the results of this analysis could not be seen as being completely reliable owing to the fact the only the diagonal elements of the variance-covariance matrix are available from the Elcovision software requiring covariances to be set to zero. The following tests were made:

a) Epoch II versus Epoch I
b) Epoch III versus Epoch I
c) Epoch III versus Epoch II
d) Epoch IV versus Epoch III
e) Epoch IV versus Epoch I

Results of the global congruency test for these five combinations showed that each passed at the 95% confidence level indicating that no significant global movement occurred between these epochs.

For single point movement (points 101-126, 130, 131, 208-213, 221 and 303-306) it was found that a displacement of more than 13mm was necessary before failure of the test at the 95% confidence level occurred. For all testing the only point to be recorded as having failed the test was point 101 between Epochs I and II. This point was closely checked in subsequent Epochs and was found to be stable. Investigation showed that in Epoch II point 101 was only intersected from three locations and one of them was slightly obscured by a lighting fixture, thus it was concluded that the coordinates of point 101, Epoch II, were not reliable.

5 Cost comparison

Of particular interest of this project was the cost comparison between photogrammetry and conventional survey. The manpower and capital costs were calculated for each of the two survey methods and are presented in Tables 3 and 4 respectively.

Table 3. Manpower costs per epoch -HK$

Items	Close range photogrammetry	Conventional survey
Time spent on field work	0.5 hour for photography 3 hours for control survey	16 hours for the field survey
Staff for field work	1 surveyor and 1 chainman	1 surveyor and 1 chainman
Time spent on office work	6 hours	2 hours
Staff for office work	1 surveyor	1 surveyor
Surveyor, per hour	$150/hr. (Avg. salary)x 10hr.	$150/hr. (Avg. salary) x 18hr.
Chainman, per hour	$48/hr. (Avg. salary) x 4hr.	$48/hr. (Avg. salary) x 16hr.
Total cost of survey per epoch	**$1,692/epoch**	**$3,468/epoch**

Although the office work for photogrammetric measurement is slightly higher, the total manpower cost of conventional survey is 2 times that of close range photogrammetry. If it is necessary to obtain one epoch of measurement per week the total cost saving for manpower for close range photogrammetry is approximately $7,100 per month, over 50% less than the cost of using conventional surveying.

The capital costs include both hardware and software costs. The hardware costs are the computer system, total station and associated accessories and the digital camera. The software costs are the close range photogrammetry package and the network adjustment software. This project used Elcovision 10 software but alternative software such as EOS Systems Inc.'s PhotoModeler is could equally be used. PhotoModeler is a significantly cheaper product and includes tools for sub-pixel target centroiding, camera calibration and the creation of fully rendered 3D models.

A one-off comparison of capital costs is seldom fair and so an evaluation of a multi-project scenario including manpower was made. Table 5 shows the costs involved if photogrammetry was adopted as a routine service being involved in 5 simultaneous projects over a 6 month time period.

Table 4. Capital costs -HK$.

Items	Close Range Photogrammetry	Conventional Survey
Computer system	$15,000	$15,000
Photogrammetry package Elcovision 10 / PhotoModeler	$150,000 / $10,000	NIL
Network adjustment package	$8,000	$8,000
Total station and accessories	$200,000	$200,000
Digital camera	$4,000	NIL
Total cost of instrumentation	**$377,000 / $237,000**	**$223,000**

Table 5. Capital costs for 5 simultaneous projects - HK$.

Items	Close Range Photogrammetry	Conventional Survey
Computer system	$15,000	$15,000
Photogrammetry package Elcovision 10 / PhotoModeler	$150,000 / $10,000	NIL
Network adjustment package	$8,000	$8,000
Total station and accessories	$200,000	$200,000
Digital camera	$4,000	NIL
Capital costs	$377,000 / $237,000	$223,000
Manpower costs for 5 projects over 26 weeks	$1692 x 5 x 26 weeks = $220,000.	$3468 x 5 x 26 weeks = $453,000.
Total cost of surveys	**$597,000 / $457,000**	**$676,000**

Under such a scenario a savings of $79,000 can be made for the Elcovision 10 option allowing its cost to be recovered in 12 months. In the case of using alternative software such as PhotoModeler, the cost recovery can be achieved in 2 weeks. It is recognised that this may not be a realistic scenario, but it does show there are significant savings to be made once photogrammetry is assimilated into a company's work practice. Using Elcovision 10, as was done in this project, a cost savings of 12% can be achieved. Using PhotoModeler, a savings of 32% is possible.

6 Conclusion

In this project, both photogrammetry and intersection survey were used to perform a non-contact deformation survey of the Star Ferry colonnade. Four epochs of measurement were used to assess the performance of photogrammetry in terms of accuracy and cost. Using the intersection survey results as the basis of comparison, the photogrammetric survey passed the engineering requirement that each coordinate should be within 2mm of their true location. Given that the precision of the intersection survey was ±2mm, the performance of photogrammetric survey was considered to be acceptable. On the basis of cost, without considering the capital cost, it was found that photogrammetry offered a savings of 50%; when using a system such as Elcovision 10 in a full-time mode, the capital costs could be recovered within 12 months and that for a system such as PhotoModeler within 2 months. These represent savings of 12% and 32% respectively.

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