The 'Broadway' Project, Kelowna, BC

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ABSTRACT What happens when a site design for a 14 story building is selected based on consolidation tests that lead to a settlement prediction of 300 mm, but the full scale preload actually settles 1200 mm?

This surprise led us to the following conclusions.

a) Sampling can be accidentally selective.
b) It takes a good underpinning contractor, but adjacent buildings can be supported and lifted as necessary.
c) Horizontal movements accompany large vertical settlements (think Poissons ratio).
d) Chance anchors don't buckle, even when overloaded and off-vertical.
e) The hyperbolic method of plotting and foretelling settlements gave remarkable results for this site.
f) The owner's insurer does not care how much the owner saved during construction (approximately $2 to $3 million) if there is a lawsuit.

Introduction

This project became a major focus and an exciting challenge for our office during design, construction, and subsequent litigation. While the design was ultimately successful despite some alarming surprises, our satisfaction with the project was dampened by litigation from neighbors regarding the performance of their building.

Project

The project was originally to be constructed as a 12 story building with one or more levels of underground parking. However, given soil conditions and the high groundwater prevalent in the area, the concept of an underground parking level was soon abandoned and a more conventional building at grade was proposed, now to be 14 stories.

Options for foundation support included piles driven to a strong layer at roughly 40 m below grade, or a raft slab. The raft slab would need the site to be preloaded prior to construction, and would need to have the surface 9 m densified to reduce the risk of liquefaction as well as to improve the surface soil characteristics.

The use of a raft slab, preload, and densification had previously been used very successfully in the same general area for The Grand Hotel, and densification and a preload had been used for a smaller municipal library building nearby.

The contractor indicated preliminary pricing on the order of $5 to $6 million for a piling scheme, with roughly $2 to 3 million for the raft slab, preload and densification, including support of the adjacent building, and including $0.5 million proposed as a contingency. The only real disadvantages were the time to preload, which the owner could accept, as well as the difficultly of supporting a building on the adjacent property line. Some damage to municipal services, sidewalks, and a small second building were also likely.

With the assumption that the property line issues could be dealt with satisfactorily we considered that the raft slab option was a feasible one for the owner.

Soil Profile and Project Commencement

The requirements and calculations for assessing the design had been based on the original soils information and laboratory testing. These were initially based on auger holes, dynamic cone penetration tests, and a series of four piston-tube samples with laboratory consolidation tests. Extensive moisture contents, sieves and limits were also carried out.

The site layout, and the center soil profile is shown on Figures 1 and 2.

A settlement estimate was prepared for the building (with a basement), and an estimate of 150 to 200 mm of settlement was made. When the project lost the basement, deeper site information was obtained using CPT to confirm the soil profile to greater depth. The settlement estimate was revised to 300 mm, based on the original consolidation test results and the new loads expected for the building. Both estimates were sufficient to confirm that either piles or a preloaded raft slab would be required, as it was a certainty that settlements would be in excess of 100 mm.

A local underpinning contactor was selected to underpin and raise the building to the north (building #1) as necessary. The building to the east (building #2) was not expected to be significantly at risk, although it was poorly constructed, was very old, and was expected to be demolished shortly for redevelopment. The project owners also had a part

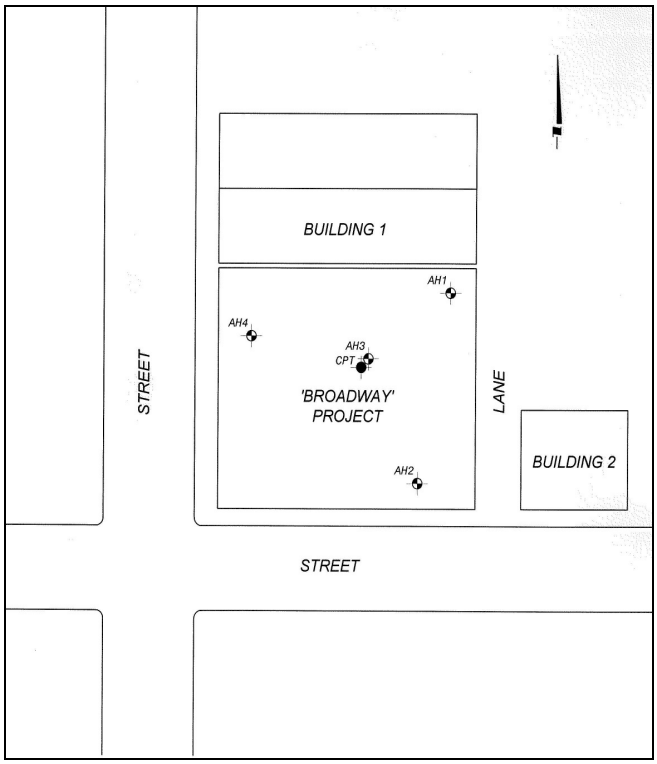

Figure 1: Site Plan

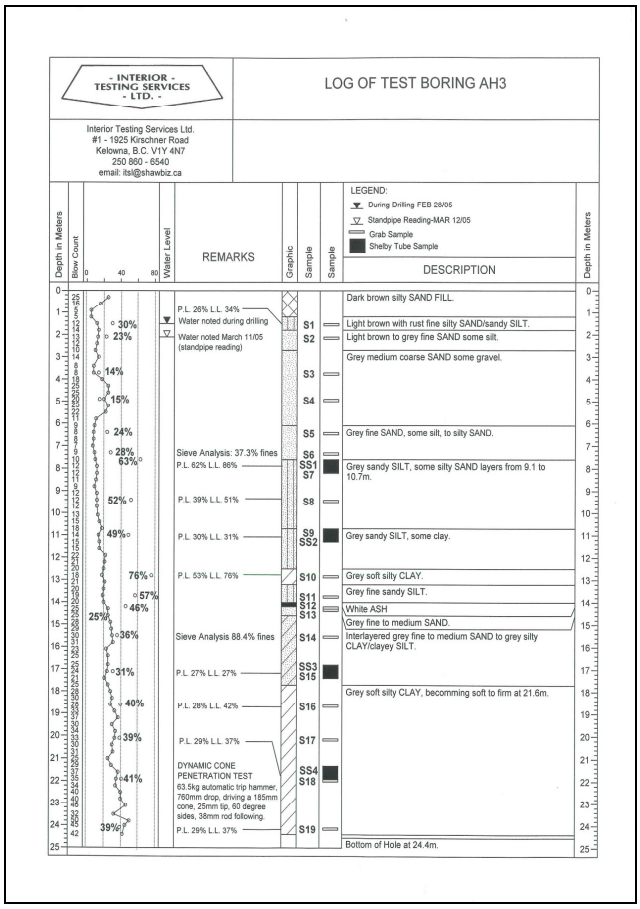

Figure 2: Central Borehole Log

interest in building #2, so that we were led to believe that its long term performance was not particularly important.

The local sidewalks, roadway, and potentially services could also be affected by the preload, and the municipality required the developer to repair these as necessary on completion.

Project Surprises

Preload Settlement

To roughly match the building loads across the site, the preload heights varied from roughly 6 m high on the property line at building #1 (restrained by a Lock Block wall), to 9.5 m high in the center of the site. Once placed, the total preload weight was on the order of 20% to 40% heavier than the total building weight, so the preload was substantial.

On placing the preload, we were unpleasantly surprised. The site began to settle very rapidly, and by day 11, one of the gauges showed settlement of 262 mm. As the other gauge was loaded (starting day 10) it had also settled 315 mm by day 21. It looked like our estimate of 300 mm of settlement was grossly in error.

The rapid settlement led us to re-visit our calculations for settlement and examine what might be leading to such variations.

We also had two additional estimates proposed by others. As part of an engineering review required by the City of Kelowna, another local geotechnical consultant #1 prepared a thoughtful and useful review of our report. Their estimate of settlement, based on the CPT results, was for 800 mm of settlement. As the site results 2 weeks earlier than this report showed 1139 mm of settlement, even this estimate fell short of what had already occurred. We asked another independent engineer #2 for a quick assessment, and on understanding our surprise, he gave a very quick estimate of 1600 mm using the worst virgin compression characteristics and thick layers of compressible soil. The actual result, some time later was roughly 1300 mm, so this turned out to be close, despite the relatively simplistic approach. At an even later date, for a prospective purchaser, another consultant #3 also reviewed the site results and prepared a short report, so there was no shortage of opinions or reviews of the site and soils conditions.

What had happened to our test results?

The explanation appears to lie in examining our sampling as it related to the CPT results. The CPT results had been obtained subsequent to the original design, primarily for checking the site to the full depth of stress. We had used the near-surface auger hole drilling, sampling, and laboratory tests in preference

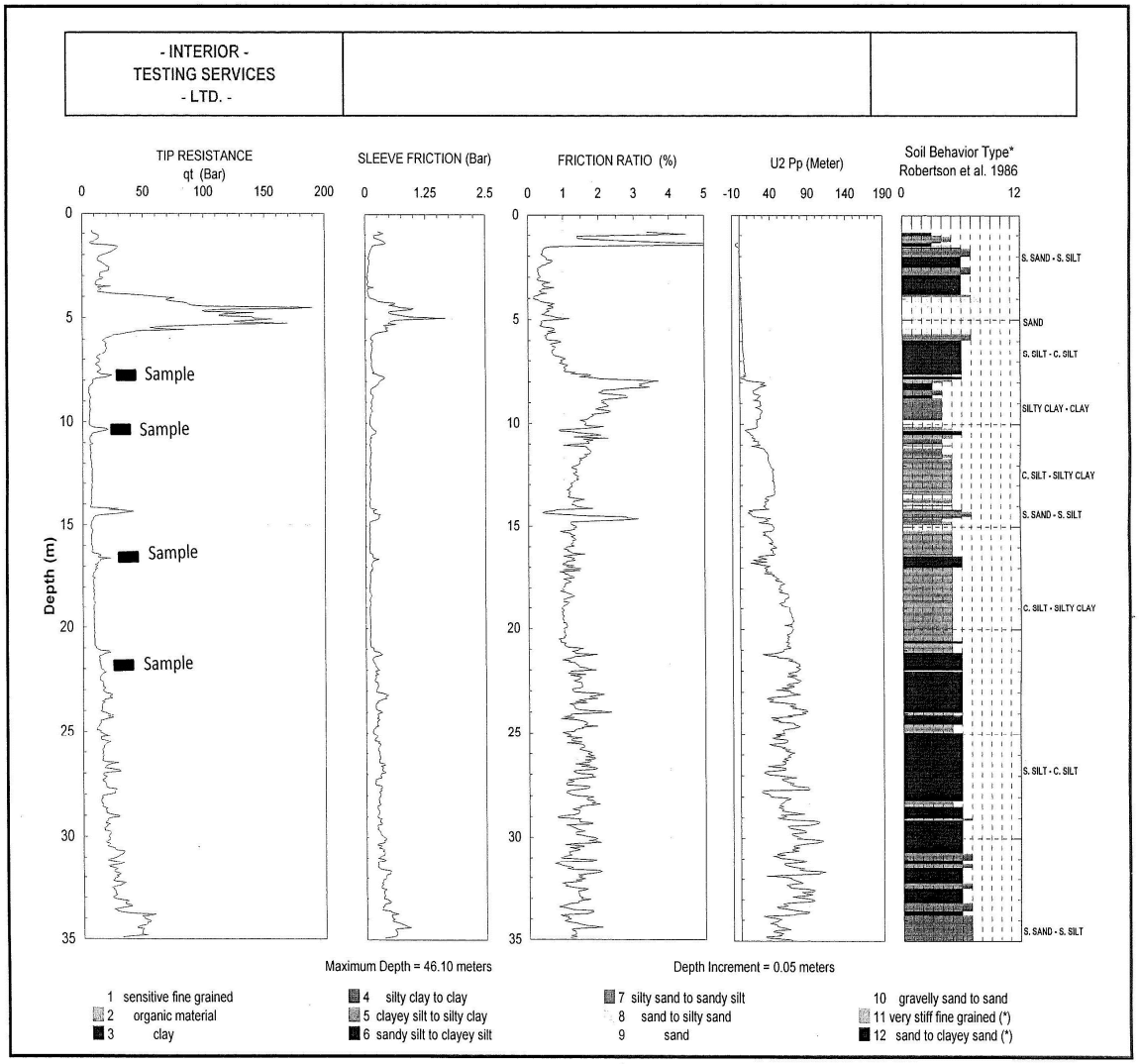

Figure 3: CPT log showing consolidation test sample locations

to CPT results for the purpose of settlement estimation, with the CPT results used more to confirm the depth to stronger possible piling layers.

On reviewing the CPT results versus our laboratory tests, we became suspicious that our sampling had produced an unintentional bias to stronger zones and this is indicated on Figure 3.

Somehow, our sampling had occurred almost precisely at each of the higher strength zones indicated by the CPT results. On further consideration, we recognized that although we had considered the recovered samples to be representative of significant thicknesses of the soft layers, they appeared to represent significantly stronger layers.

This explanation was also consistent with one other aspect we had not fully appreciated at the time. Our technician had actually attempted on several additional occasions to recover piston-tube samples at other depths, but these attempts had been unsuccessful, as is occasionally the case when sampling.

However, this now suggested that our samples were different, in that they had been recoverable. This suggested we had inadvertently recovered only the stronger samples. While the moisture contents, limits, and visual observations of the soils had not suggested any substantial difference in the materials, and the recovered samples appeared reasonable as representative samples, we now believe they were inadvertently selective.

Other explanations are still possible, but we think this is the most likely.

The magnitude of the settlement subsequently led to other surprises.

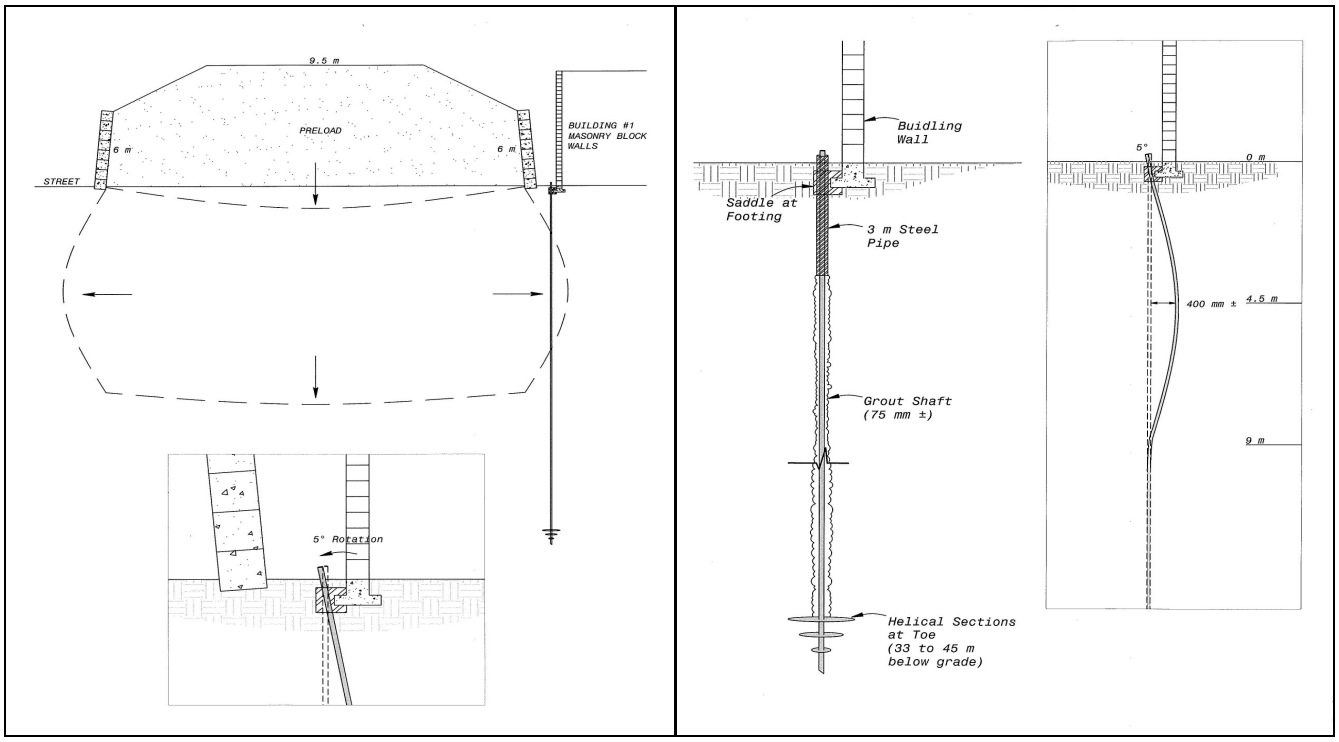

Figure 4: Schematic anchor section at building and anchor head rotation with estimated lateral shift

Underpinning Scheme, Building #1

Along the north property line, to support building #1, the underpinning contractor installed a series of Chance anchors to depths of 33 to 45 m below grade before finding sufficient resistance to believe they could lift the building as required. A schematic drawing of the scheme is shown on Figure 4.

The anchors were followed by a grout column as they were advanced. As they were installed, a 3 m (10') length of steel pipe was used to encapsulate the near surface section and connect it to the footing saddles.

The anchors were only intended to lift roughly 150 kN each (300 kN nominal ultimate load), and were linked by a series of hydraulic jacks so that the building could be lifted fairly evenly. Surveying was provided on a weekly basis to decide when lifts would occur.

The rapid settlement of the preload led to an early need to engage this system, and a further series of surprises were encountered.

i. How much does the building weigh, why won't it go up?

On initial jacking to the intended loads, the building would not move. It was subsequently found that the concrete blocks had been infilled, which was part of the answer. However, as it wouldn't lift until loads closer to 250 kN were applied, it ultimately appeared that some work was also needed to break the suction and that

the slab was also connected to the walls (adding weight and suction). This was getting uncomfortably close to the proposed ultimate capacity of the anchors.

ii. How come the anchors are rotating towards the preload at the saddle? Is it a problem? Are they buckling under the higher loads?

The potential for buckling the anchors became a worry when it became obvious that the Chance anchor heads had rotated on the order of 5° at the saddle.

When we considered 300 mm of settlement, we had not worried at all about lateral movements. However, as the vertical settlements began to approach 1 m, and the anchor head rotation became more obvious, we recognized that the anchors were almost certainly being pushed laterally, and we had not previously expected or considered this possibility.

The ground deformation, assuming elastic deformation at a Poissons ratio of 0.5, implied that the anchors were being

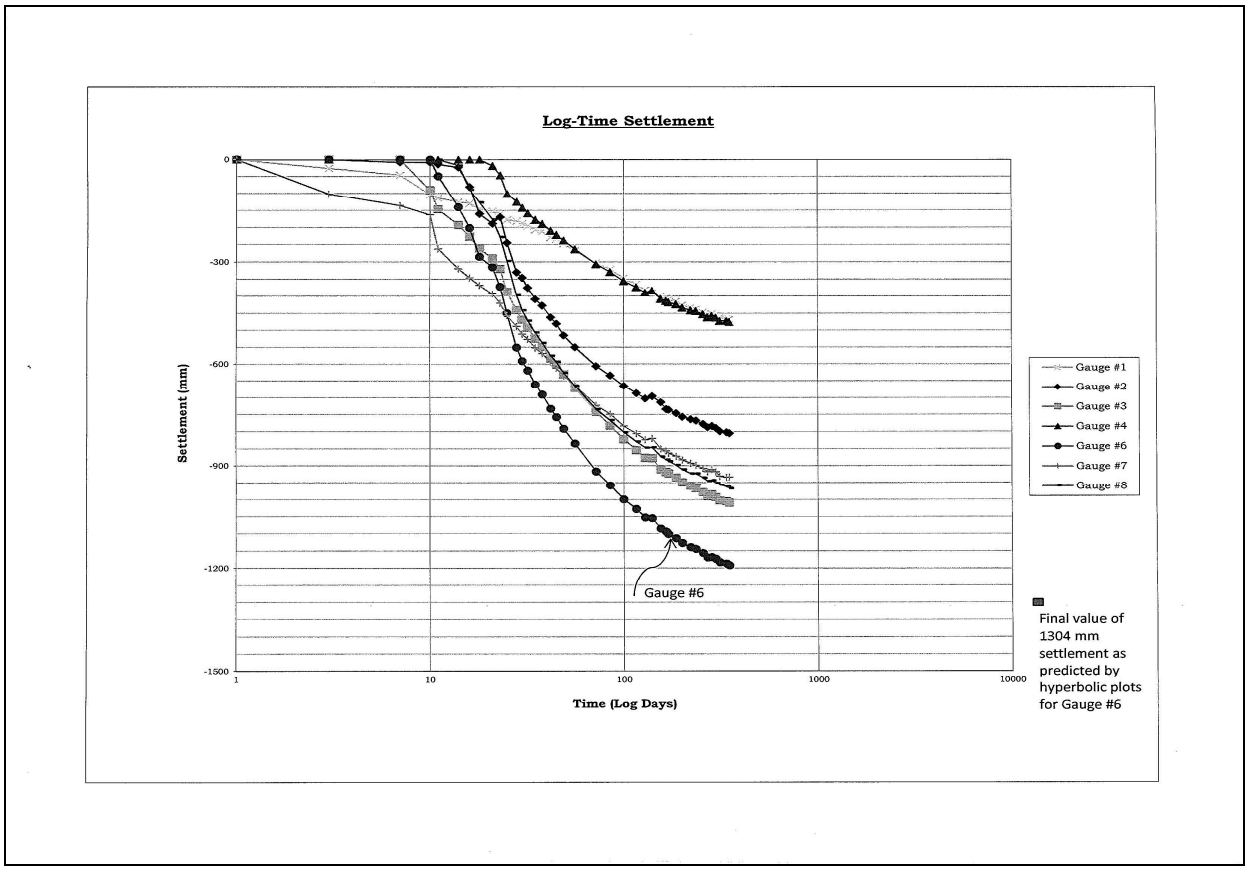

Figure 5: Log-time settlement plot to end of preload

pushed over on the order of 500 mm. We guessed this to be occurring between the surface and roughly 9 m below grade. Assuming a bowed anchor between 0 and 9 m below grade, the anchor head rotation of 5° implies roughly 400 mm of off-set at 4.5 m below grade, so perhaps Poissons ratio was only 0.4.

We recognized that the anchors now had significant curvatures in them, and this was a concern for the structural engineer, who worried about buckling of the anchors.

We discussed this with the contractor, and jointly were convinced that buckling due to axial loads were not likely the cause of the anchor deflections, as it was almost certain there was significant lateral movement of the soil and this was the real cause of the anchor head rotation. We were confident the anchors had simply been pushed into a curved shape within the soil, but they would still be well supported laterally, and buckling would not be an issue. The contractor subsequently put this to the test, and successfully applied loads on the order of 300 kN to the piles without difficulty. In this respect, we believe the upper 3 m length of pipe was sufficient to handle the surface conditions where buckling could have been a real possibility.

However, our enthusiasm for excess loads was tempered by the structural engineer, who advised that his connections to the wall were only designed for 225 kN.

Underpinning Scheme, Building #2

As the excess settlement became apparent, the adjacent building #2 became of some concern.

Some new cracks in the building were occurring, suggesting the settlements were beginning to pull this building down also, and it soon became apparent that it would also need to be supported.

To our surprise, despite the partial cross- ownership, there was apparently no similar agreement to support or repair building #2, as had been obtained by our client with the owner of building #1. Nor could this apparently be arranged now. Permission was given to lift the building to some degree, but complete access was not granted. It also became apparent that the building had numerous existing problems that meant it could not practically be supported and lifted as had occurred with building #1. Combined with the expectation that it was soon to be demolished, the lack of agreement, and the apparent eagerness of the owners to press their advantage, it appeared likely that there was a conflict brewing.

Table 1: Hyperbolic projections and survey results

Gauge	Sept. 21/07	Hyperbolic Predictions			Actual Results		Updated Final (Hyperbolic)
	34 Weeks	40 Weeks	50 Weeks	Final	40.6 Weeks	50 Weeks	
1	439	456	472	552	452 (-4)	468 (-4)	548
2	768	785	802	879	784 (-1)	805 (+3)	882
3	967	996	1022	1139	905 (-11)	1010 (-12)	1126
4	444	469	489	589	458 (-11)	476 (-13)	570
6	1144	1171	1195	1304	1168 (-3)	1192 (-3)	1298
7	899	914	932	1010	912 (-2)	933 (+1)	1013
8	923	947	968	1062	943 (-4)	967 (-1)	1053

We had identified that the adjacent buildings could be damaged by work on the site, but had not really expected much to occur with only 300 mm of settlement under the main preload area, and building #2 more than 6 m from the property line.

However, it was a surprise to find that there had been no discussions or agreements at all. It appears likely that the agreements on building #1 were solely because we made it very clear that significant damage could be done to the adjacent building #1, particularly during the preload phase. The City had also required that the owners commit to any repairs necessary to the sidewalks, roads or services in order to protect the interest of the municipality. Only the potential impact to building #2 had been neglected, and this may have been partly due to the difficulty created by the partial cross-ownership.

Preload Monitoring

Meanwhile, the preload continued to settle. Our normal evaluation method has usually been to use the log time versus settlement plots (as shown on Figure 5), and estimate the remaining settlement based on a straight-line projection. This is generally satisfactory but often has drawbacks when estimating the future and long term conditions. As part of their reports, Consultants #1 and #3 had both attempted to estimate future settlements from these plots, with conflicting views on remaining settlement, when the preload could be removed, potential secondary settlement, and so on, as legitimate differences in interpretation when reviewing such plots. As an option, we considered using the hyperbolic plot of settlement versus time over settlement, which derives from a paper by Tan, 1972, and had been applied to a bridge in Merritt, BC, by Wightman and Broomhead in an earlier VGS Symposium.

These plots soon caught our attention, as the results at each gauge settled into a straight line with completely predictable results after the first one or two months. The results were so robust that we used them to predict the surveyor's results, and were pleasantly surprised to see they consistently agreed, as shown on Table 1.

The hyperbolic plot is believed to represent both primary and secondary settlements, and the total amount of settlement is predicted by the inverse of the slope. A typical result is shown on Figure 6, showing the initial variations, then the development of the straight line. At each gauge, the hyperbolic projections were straight lines, but the slope (and therefore the final predicted total settlement) varied with the intensity of the applied preload.

We were able to project the total settlement for each location under the local preload condition, and compare the estimate to the settlement to-date, which gave a direct estimate of the settlement remaining. We could then project the time at which all predicted settlements would be within roughly 50 mm of each other. As the final building foundation was to be a raft slab, we knew 50 mm was within an acceptable range (typical allowable for a raft slab is on the order of 50 to 100 mm). This is indicated by Table 2.

We were impressed that the hyperbolic method gave such consistent results, and could be used to predict the survey results. The comparison between predicted, then actual results is shown on Figure 7.

Table 2: Projections for preload (final settlement minus projected settlement)

Gauge	Projected Settlement Remaining (40 Weeks)	Projected Settlement Remaining (50 Weeks)	Projected Settlement Remaining (60 Weeks)
1	96 mm	80 mm	68 mm
2	94 mm	77 mm	65 mm
3	143 mm	117 mm	100 mm
4	120 mm	100 mm	86 mm
6	132 mm	108 mm	92 mm
7	96 mm	78 mm	66 mm
8	115 mm	94 mm	79 mm
Differential	49 mm	40 mm	35 mm

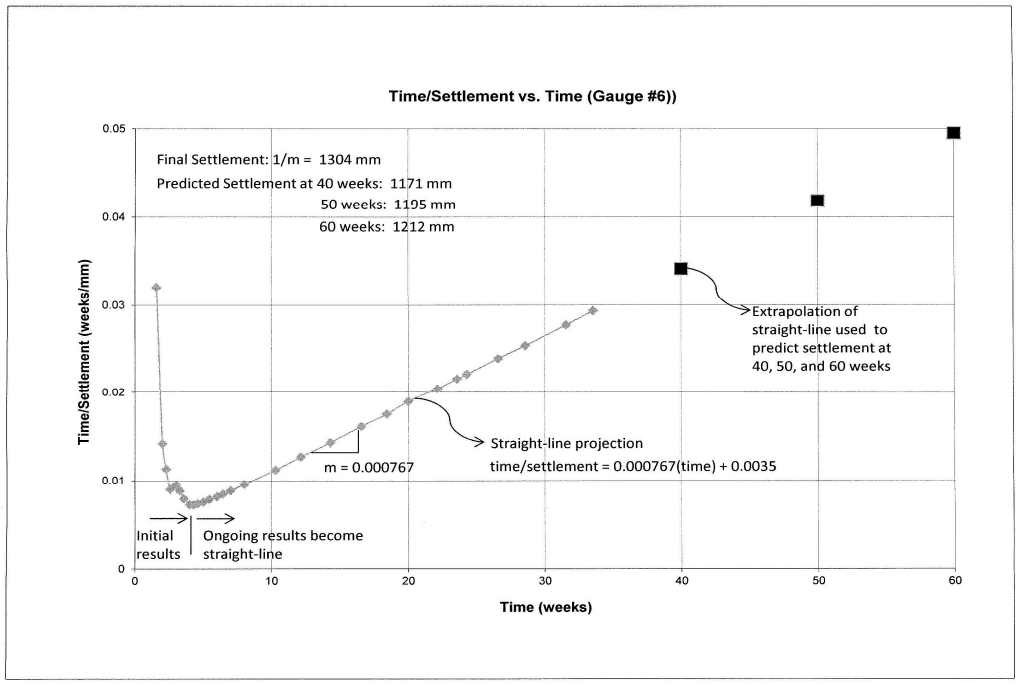

Figure 6: Settlement review and projections using hyperbolic method

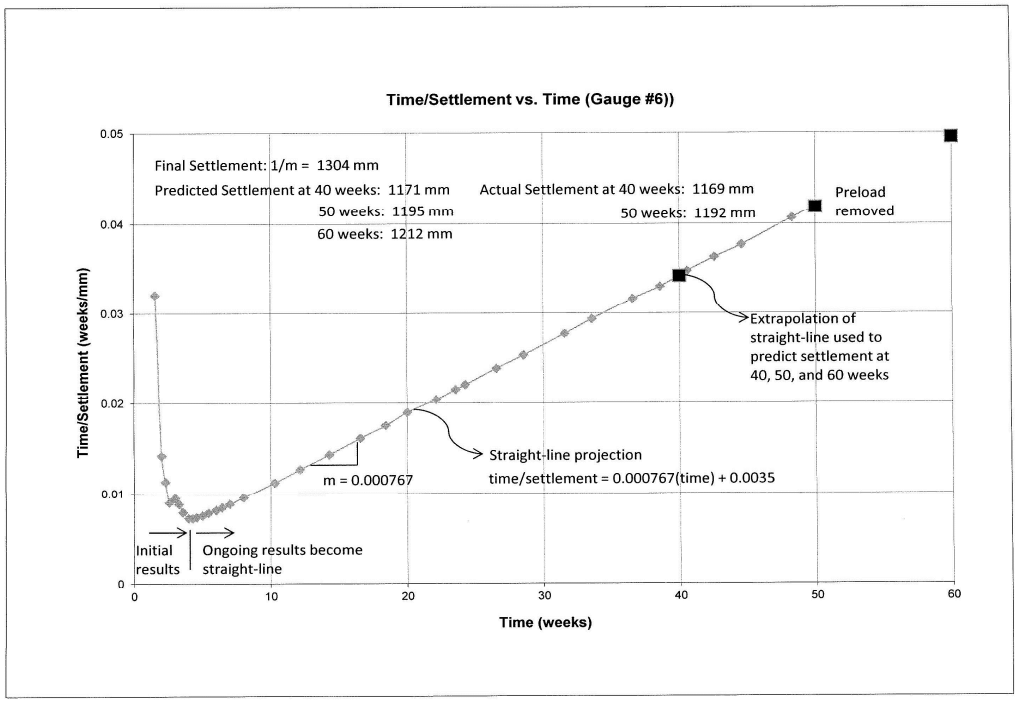

Figure 7: Settlement results and projections using hyperbolic method

Table 3: Settlement at varying depths

Gauge	Location	32 Days	Layer Consolidation	140 Days	Layer Consolidation	350 Days	Final Projection
6	Surface	304 mm		740 mm		1192 mm	1298 mm
			30 mm		28 mm	-	-
5A	-7.5m (25')	274 mm		712 mm		-	-
			55 mm		211 mm	-	-
5B	-13.7m (45')	219 mm		501 mm		-	-
5C	-19.8m (65')	177 mm		-		-	-
5D	-25.9m (85')	146 mm		-		-	-
5E	-32.0m (105')	177 mm (?)		-		-	-

We have since used the hyperbolic method on other sites, with similar satisfaction. On other sites, we have found that results diverged from the straight line, but usually with a projection of reduced amounts of settlement.

This method appears well suited to a site where large settlements occur, and are being carefully monitored over a period of more than 3 months.

As an aside, we also had a borehole with a spider system for monitoring settlements at depth. Unfortunately, this was vandalized fairly early on, so that the deepest results could not be obtained past 32 days. Regardless the results were interesting, suggesting the upper 7.6 m layer was relatively uncompressible (only 30 mm of settlement).

Results at 32 days and 140 days were as shown within Table 3. While these were of interest, they ultimately became secondary to simply using the hyperbolic plots to predict the final surface settlements.

Densification

After all the above, vibroreplacement densification was a breeze, and proceeded without incident once the preload had been removed.

Building Performance

Settlement monitoring of the completed raft slab was carried out, and only 10 to 25 mm was noted as the building was constructed to completion.

Litigation

Unfortunately, the postscript to the project was litigation, and a rather mysterious affair it was. The client had some type of insurance, but it probably wasn't quite right, and the cross-ownership likely complicated this further.

The building #2 group sued the owner (the project definitely caused at least some part of the building damage) and us as the geotechnical engineers (presumably we hadn't stopped them), the building contractor (not involved at all!), and the Municipality.

The building #2 owners had received a variety of structural opinions ("don't worry, it's fine;" to "I have some concerns but they are fixable;" and finally the favorite –"you must evacuate now!").

The claim was initially framed for roughly $1 million. The building contractor found yet another experienced structural engineer who suggested less than $50,000 to fix.

The claim was ultimately settled for $200,000 of which the plaintiff apparently spent nearly $100,000 on legal fees. Our insurer paid out a substantial proportion of the total settlement to help make it go away, and our contribution, 50% of our deductible, was sufficient to almost completely remove any profit we made on the work.

We are unclear how we could have protected ourselves from this type of problem, except to have the owner indemnify us against third party claims. As it was the owner's insurer footing the settlement bill, we know they did not care that the owner had saved roughly $3 million on construction costs by comparison to a piled solution. Obviously we were disappointed that the owner did not simply deal with the suit directly, but it is not a real surprise that it didn't happen.

Conclusion

The project was ultimately a success from a technical and engineering perspective, with a very real cost saving to the owner. However, our appetite for saving the owner money without being insulated from the risks he is running has been sorely tested.

Real problems could have occurred at the building to the north without the competence and know how exhibited by the underpinning contractor,

and this could have become an issue that might have reflected badly on our underestimate of settlement.

Otherwise, it appears that a preload and raft slab (with densification where needed) is a relatively bullet-proof method of design, even when there are a series of surprises.

Finally, we are impressed with the hyperbolic method of monitoring and predicting preload results. As has been speculated upon by others, it does appear that this approach is in some way fundamentally correct.

References

Tan Swan Beng, “An Empirical Method for Estimating Secondary and Total Settlement”, Proceedings of the Fourth Asia Regional Conference on Soil Mechanics and Foundation Engineering, Volume 2, p.147 to p.157, July 1972, Bangkok, Thailand