Paper prepared for ORVSS XXX (Ohio River Valley Soil Seminar) October, 1999

VALUE ENGINEERING - A GREAT CONCEPT WHY ISN'T IT USED MORE FREQUENTLY?

Jorj Osterberg[1]

ABSTRACT

Value Engineering is a great idea. When it has been used as intended, both economical and better designs have resulted. But for every successful case, there are many where it has not functioned well and far many other cases where it should have been used and was not used. This paper discusses instances where value engineering did not work and where many opportunities were missed for great savings and better designs. When value engineering is specified in a contract (mostly for government work), and the agency which prepared the design and specifications is the same agency which makes the decision on the value engineering report, it often looks upon the recommendations as an intrusion, and may think it reflects unfavorably on its original design So why should it accept someone else's ideas? In other cases, where value engineering is not an issue in the contract, why should the designer accept changes proposed by others which he thinks might reflect poorly on his engineering ability and would only cause more headaches and costs for him? There are other cases where the owner is not even aware of the possibility of a more economical and sometimes better design. If the owner was aware of the possibility, he probably would insist on a value engineering study. To rectify this situation, the agency responsible for design and construction should not be the same agency which judges the value engineering. An impartial third party should be the judge. For private work in which value engineering is not specified, there should be incentives for the design firm in the form of a percentage of the money saved and the additional cost incurred. The paper gives examples to illustrate the above.

INTRODUCTION

Value engineering is a great concept capable of potentially saving billions of dollars (yes, I mean *billions*) in construction costs and in many cases improved and safer designs. It is not likely that this potential will be reached in a very long time, if ever. However, it is within our power as engineers and contractors to save tens if not hundreds of millions a year if we get together and act. The

[1] Professor of Civil Engineering Emeritus, Northwestern University
Consulting Geotechnical Engineer, Aurora, Colorado

reasons for the present situation are many: the tendency of designers and particularly those in government to "play it safe" by being overly conservative; the reluctance to deviate from "accepted practice" for fear of later being sued; the antiquated and inflexible building codes; the attitude that "it's not my money, so why would I care"; do it the way it was always done; the ignorance or indifference of the owner about potential savings. If we engineers are honest, we should look at each project with the thought in mind - how would I do it if I were paying for it. Certainly safety and obeying the law is the primary and utmost consideration. Then the most economical, functional and practical solution should be sought, followed by a study of how costs can be cut. But don't forget, sometimes contractors have good ideas too! Often, an informal peer review within your own organization can be of great help. Then imagine you are the owner while you are reviewing your design for possible savings.

CASE HISTORIES

Following are case histories in which I have been directly involved in which substantial savings were possible but few in which the savings were actually realized.

Case 1 - Value Engineering Involved. The Corps of Engineers designed a new bridge to replace an old one which was built around the turn of the century. The new bridge was built close to the old one and the foundations were to be on drilled piers extending through a rather thick layer of very hard glacial till with boulders to rock. The foundations of the old bridge, which was a heavy stone-faced concrete bridge were on top of the glacial till and there was no evidence of settlement. The contractor, who realized the difficulties of drilling through the hard till and boulders, wondered why the new bridge, which weighed less than the old bridge, could not rest on drilled shafts resting on or a short distance into the hard till. He contacted a consulting engineering firm which had long experience designing bridges, which in turn consulted me for advice. I could see no reason why the drilled shafts needed to go to rock. There was no evidence of scour at the foundations of the old bridge. Furthermore there would be a substantial amount of overburden from the river bottom to the glacial till. A request was made to make a value engineering study but the Corps said absolutely no and would not even permit a meeting to discuss the matter. The bridge was built as the Corps had designed it. I did not make an estimate of what the savings might have been, but certainly it amounted to hundreds of thousands of dollars. Remember, this was your money and my money! This is a case of the owner who was also the designer ruling out value engineering out of hand.

Case 2 - Water Treatment Plant. A water treatment plant was designed to be next to a large river for a large mid-western city. The design firm for the city engaged a well-known geotechnical engineering firm to make a soil investigation and make a report giving recommendations for the foundations. After a comprehensive investigation, the firm recommended that a surcharge fill be placed over the areas which would be subjected to heavy loads. The surcharge fill was almost complete when the design firm went to the city engineer and said the structures must go on piles. The city engineer who was a civil engineer with some foundation experience wondered why the design firm changed its mind. When he couldn't get an acceptable answer, he contacted me. After studying the soil borings and report, I could see no reason why the structures needed to be on piles. The surcharge would consolidate the ground to loads larger than the structures would exert and the time-settlement curves would give a record to tell when and if the settlement at any time was sufficient. My recommendation was rejected and I was asked to come to the city and meet with the designers and the city engineer. The designers said, "If piles are used there will be no settlement." I refuted that statement and said there will be settlement, but if the piles were properly designed and installed, the settlement would be small and tolerable. Then I was asked, "Will you guarantee that if the piles were eliminated, there will be no cracks in the concrete?" I said I could not guarantee there would be no cracks but could give assurance that there would not be any detrimental settlement provided the surcharge was allowed to remain a sufficient time for the soil to consolidate to a load greater than to be applied. The designers then said the small additional cost of about $20,000 for piles would give additional assurances that there would be no settlement. I said, "What? I estimate that the additional cost of the piles would be about $200,000." The answer was, "But it would only cost the city $20,000 because it was an EPA project." and Uncle Sam would pay the rest. I said, "Who the h---'s money do you think this is?" When the city engineer was convinced that the surcharge method would work, he ordered the designers to eliminate the piles and use shallow spread foundations. I requested to have the settlement monitored as the project was under construction and when all the tanks were filled and to send me the results. The settlement did not exceed ¼ inch. Why the design firm changed its mind I will never know.

In most similar circumstances the owner would not have accepted my recommendation because the design firm would have refused to guarantee the work. Do you think this consultant wondered what he would do if it were his money that would be used to build the project?

Case 3 - Spread Footings. I was hired by Allstate Insurance Company to review a report making recommendations for the foundations and earthwork for a medium size office building in a large city on the West Coast. The report had sufficient borings and soils information to determine the allowable bearing pressure of footings. The consulting firm, a well-known one with a good reputation, recommended using 2,000 lbs/sq ft maximum bearing pressure. Soils information indicated to me that 5,000 lbs/sq ft could be used. I phoned the engineer who wrote the report and learned later that he was a new and inexperienced employee. I asked if there was anything unusual about the soil that made him recommend the low bearing pressure. He replied that there was nothing unusual. "Then, why did you recommend such a low bearing value?" He answered that using a higher bearing value would not save much. I pointed out that the savings would be in the tens of thousands of dollars. He then said he could not change the recommendation unless he went to higher authority. Well, I went to higher authority and talked to someone I knew and felt was well qualified. He agreed that a higher bearing value could be used but questioned my 5,000 lbs/sq ft recommendation. We finally agreed on 4,000 lbs/sq ft.

Even though very large sums of money were not involved, I give this example because I have experienced the very same thing a great many times. I am sure that every day many reports are made recommending bearing values which are much too low. When you multiply this by the number of footings which are constructed in a year, pretty soon it runs into a lot of money. Again, "It's not my money, so why would I care." However, high bearing values should only be recommended when the borings and soil tests clearly indicate that the bearing value you recommend can be justified. Even though this example does not justify a full value engineering study, there should be some mechanism whereby these wasteful practices do not occur.

Case 4 - Belled Piers. This is not a case of value engineering, but I can't resist telling it, because it defies all common sense. Actually, you might call it reverse value engineering. I wrote a report for Allstate recommending an office building be placed on belled piers and recommended the allowable bearing value. After the concrete frame and floors were completed, the structural engineer discovered that the shafts were drilled with a machine that made the angle to the vertical of the bells 45 degrees contrary to the specs which called for 60 degree bells. He stated to the owner that this was unacceptable and he could not approve it. The contractor was quite embarrassed since he had not read the specs carefully enough and missed the 60 degree requirement. I then looked at the concrete strengths from the test cylinders and found they were all about 5,500 lbs/sq in, whereas the specs called for 4,000 lb concrete. I presented this to the structural engineer but he said that made no difference. Then the contractor drilled a new shaft just outside the building and demonstrated that the angle of the bell was 53 degrees. This still made no impression on the structural engineer. Then a careful level survey of the entire building was made to determine if there was any differential settlement and there was none. Yet, the structural engineer could not be budged and said he could give no guarantee of the structural integrity of the building. When I asked him what the contractor should do to the building so he could give a guarantee, he had no answer. Since I had done a lot of previous work for Allstate and had good relations with their construction department, and since I convinced Allstate that the guarantee doesn't mean anything anyway, they took my advice to simply ignore the structural engineer. After the building was completely finished and in operation for more than a year, there was no evidence of differential settlement. Perhaps you might call this a case where value engineering would say, "Make no changes." I have had a number of cases in which similar incidences occurred, but in those cases the client would not accept my recommendation because the architect or design engineer would not make any guarantee if my recommendation were followed.

A great many cases have been encountered using the Osterberg method of load testing drilled shafts, in which if value engineering had been applied, many million of dollars could have been saved. In all these cases there was no mechanism I am aware of which could have initiated a value engineering review. To understand these cases a brief description of how the Osterberg Load Test is performed follows: Fig. 1 shows a hydraulic jack-like device placed on the bottom of a drilled shaft. After the concrete is poured and cured, pressure is applied to the device which exerts an equal upward and downward force on the shaft. The downward end bearing force is resisted at all times by the side shear (skin friction) and therefore no overhead load frame or dead weight reaction is needed. The upward movements of the bottom of the shaft, of the top of the shaft and the downward movement of the bottom are measured by telltales and are recorded on a data logger from which the movements can be plotted and/or shown directly on a monitor screen. Movements will continue until either the ultimate in side shear or the ultimate in end bearing is reached or the capacity of the device is reached, whichever occurs first. When this occurs, the test is completed. A method of constructing the equivalent top down curve (Ref.1) has been shown to agree with a top down test using a kentledge load made nearby.

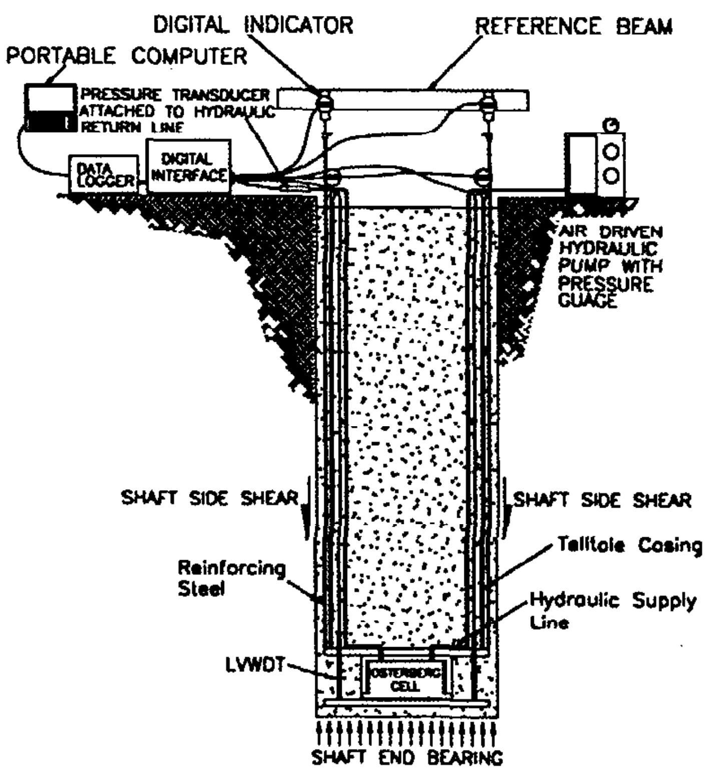

Fig 1 - Osterberg Load Test Method

Approximately 400 O-Cell tests have been performed. Load tests of up to 15,000 tons have been made on shafts up to 9 ft. in diameter and up to 200 feet deep. In the great majority of the tests, it has been found that the side shear (skin friction) is considerably larger than most designers assume and larger than estimated from compression tests. For rock sockets it has been found that the side shear is very much larger than that generally assumed and that compressive strengths of rock cores is a poor indicator of the sides shear. The cases discussed below illustrate these findings and indicate that a value engineering analysis could have saved very substantial sums of money. One case is given which shows how results on a test shaft used wisely and prudently for designing the working shafts is itself a form of value engineering.

LOAD TESTS ON DEEP SHAFTS USING THE OSTERBERG LOAD CELL

Case 5 - Test Shaft for drilled piers for a Bridge over the Ohio River at Owensboro, Kentucky.

Fig. 2 shows the soil and rock profile and the load- deflection curves for a test shaft at the site. Because of possible deep scour in the future, only the load capacity of the 19 ft of shale below the sand was to be considered in the design.

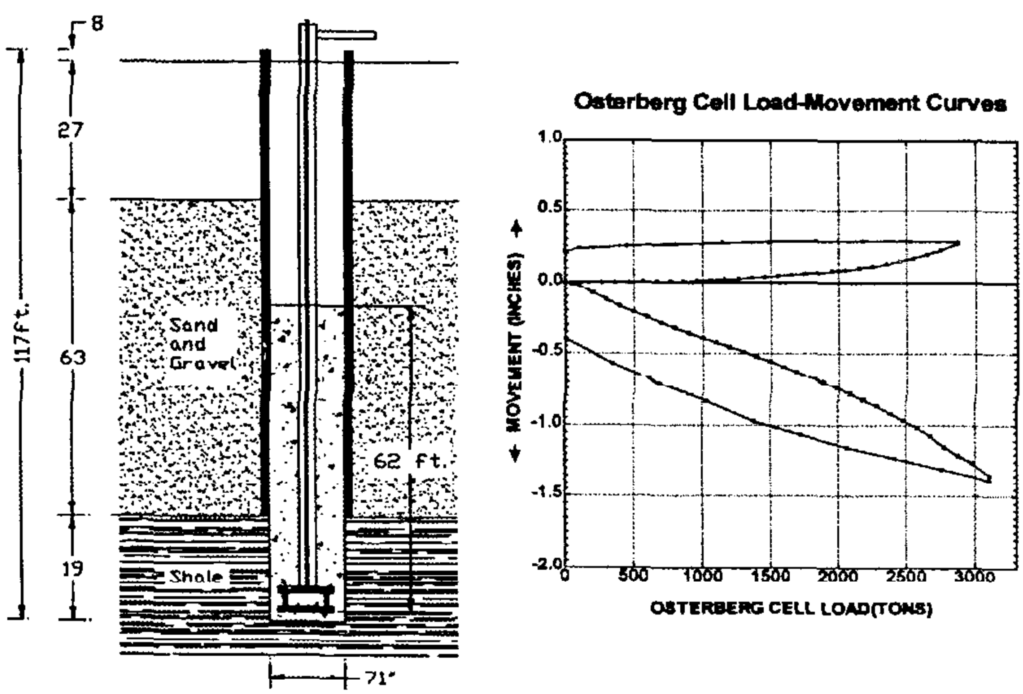

Fig 2 - Test Shaft for Bridge in Owensboro, KY

The rock consists of weathered shale (compressive strength varied from 350 to 500 lbs/sq in) with coal seams and sandstone seams. As shown in the Fig. 2, concrete was placed to some distance above the top of the shale. However, strain gage readings in the concrete above the shale showed that the load taken above the top of the shale was negligible. The test was designed to go to three times the design load.

However, the test went far beyond that and the ultimate load was not reached in either side shear or end bearing when the capacity of the device (3,000 tons up and 3,000 tons down) was reached as shown in the figure. At the design load of 1,000 tons, the total deflection was only 0.2 inch. The test showed that considerable savings (estimated to be in the millions) could be made if the working shafts would have been redesigned to take advantage of the actual measured strengths and deflections from the test results, but this was not done. For political and practical reasons, completion of the bridge on schedule was of the utmost importance and it was assumed that the time for redesign and renegotiating of the contract would delay completion. Though the redesign and renegotiating would have taken some time, it could easily have been made up by the reduction in construction time, but this was not even estimated. Value engineering as far as I know, was not even discussed. This is another case of using the load test results to demonstrate that the design was adequate but not to take advantage of what savings might result.

Case 6 - Test Shaft for a Tall Building in Hong Kong.

The drilled shaft, 3.3 ft. in diameter, went through 56 feet of overburden and 10 ft. into partially weathered granite. The hole was stabilized with bentonite slurry. The O-Cell was placed on the bottom and the shaft was filled with tremie concrete only to the top of the socket. Therefore no value was given to the side shear of the overburden. The test went only to twice the design load of the shaft even though the test could have gone to almost double that to reach the maximum allowable load of the O-Cell. The building code in Hong Kong does not allow the use of end bearing in shafts stabilized with bentonite slurry. At twice the design load in side shear, the upward deflection of the top of the shaft was only 0.07 inch. But since with the O-Cell, the upward load is equal to the downward load, the shaft was actually tested to four times the design load and at that load the downward movement of the bottom of the shaft was only 0.22 inch. Therefore at four times the design load the total deflection, if the load is applied to the top of the shaft, would be about 0.30 inch. Both the upward and downward load-deflection curves were linear. A conservative estimate of the lowest possible ultimate load indicated that, at the design load, the factor of safety would be at least 7. Wouldn't this be a great case for value engineering? Well, it wasn't. The designers were very happy that their design was safe. I wrote a critique which pointed out that considerable savings could materialize if the depth of penetration of the sockets into the rock would be cut in half. The reply was that it would be difficult to convince the building officials to change the design and to allow end bearing and also that time is money and the cost of delay would probably be more than the savings. What's the point in making a *load test on a test shaft and not even considering any changes in design?*

Case 7 - Test on a Working Shaft for a Bridge in the Midwest.

The shaft went through the overburden and was socketed 37 feet(!) into a shale formation. The drilling contractor and the geotechnical consultant told the state that the socket did not need to be so deep but it was drilled to that depth anyway. The design load was 500 tons and the test load was made to 3,000 tons up and 3000 tons down. Fig. 3 shows the load-upward movement of the shaft and the load-downward movement in end bearing. Fig. 4 shows the equivalent top-down load-deflection curve. It is seen that the upward movement and downward movement lines are linear and the equivalent curve is linear. At the design load the settlement is 0.01 inch and at 11 times the design load it is 0.10 inch. It is obvious that the great majority of the movement was elastic. The owners were very happy with the results and completed the bridge piers with the original design! This is the most extreme example of an over designed shaft I have ever encountered. Just think what could have been saved if the penetration into the rock could have been two diameters instead of twelve diameters.

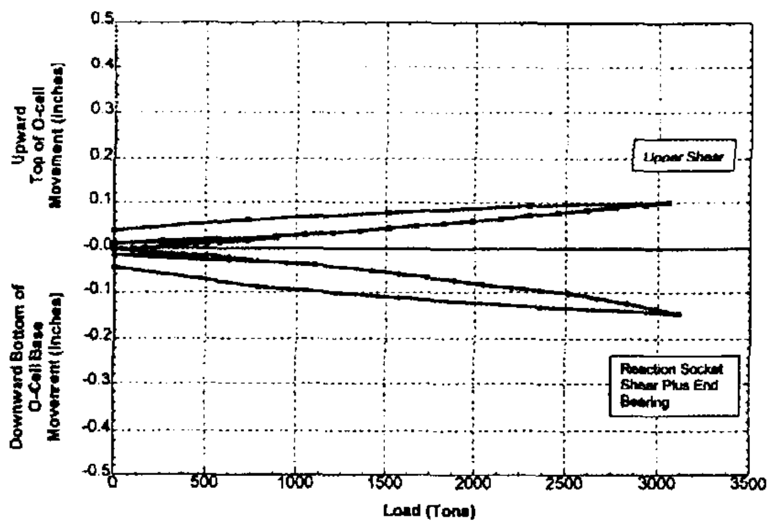

Fig 3 – Osterberg Cell Test Results – Load Movement Curves

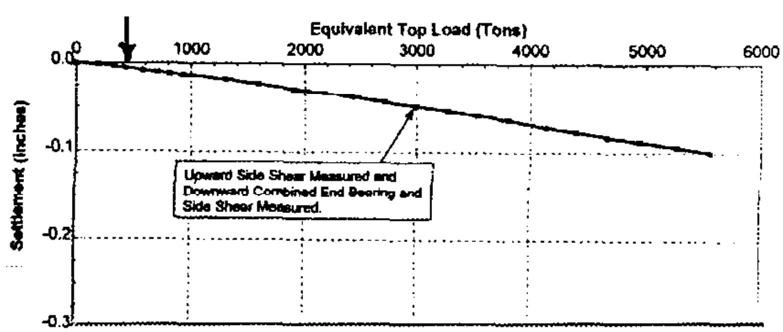

Fig 4 - Equivalent Top-Load Settlement Curve

Case 8 - Test Shaft for the Foundations of a Bridge in Massachusetts.

The soil profile consists of overburden underlain by shale. The purpose of the test shaft was to determine the ultimate side shear and end bearing of the rock socket and use these values for designing the working shafts. The O-Cell was placed on the bottom and the shaft grouted to the top of the socket. Figs 5 and 6 show the O-Cell movement curves and the equivalent top down curve. Coincidentally the ultimate in both end bearing and shear occurred at about the same load and at about the same deflection, 28 MN and 15mm (3150 tons and 0.75 *inches). This is what you would call value engineering incorporated in the* design, the way the previous examples should have been carried out.

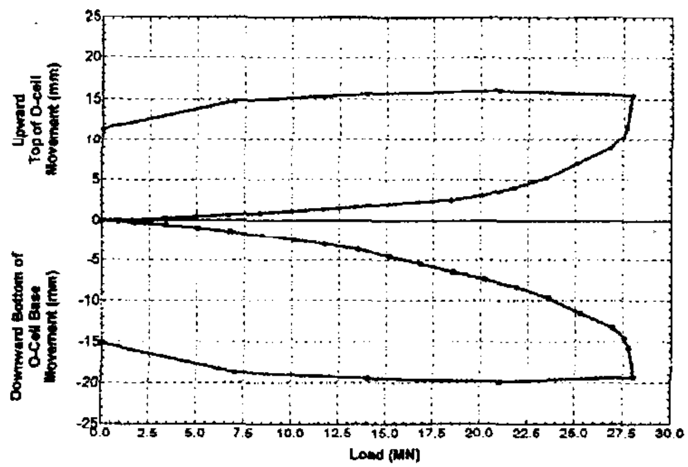

Fig 5 - Osterberg Cell Load-Movement Curves – Bridge in Massachusetts

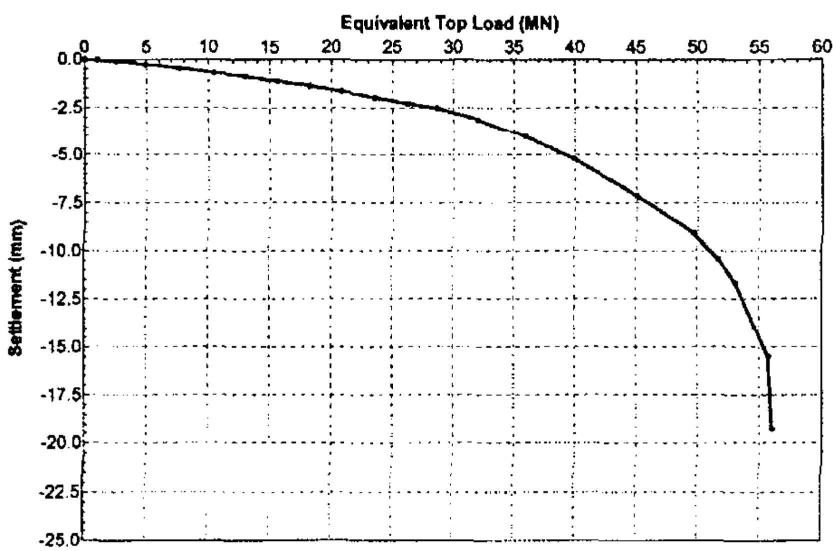

Fig 6 - Equivalent Top Load-Settlement Curve – Bridge in Massachusetts

CONCLUSIONS

1. Value engineering is not being used as frequently as it should be used. There are far more examples of where value engineering could save large sums of money than where value engineering did save money.
2. The reasons why value engineering is not used more frequently are:
   a. In many cases the group evaluating the value engineering is the same group that did the original design
   b. The limitations and obsolescence of building codes are obstacles.
   c. The fear of being sued if something should go wrong.
   d. There is no incentive for the designer to go through a value engineering assessment. In fact it is a disincentive because he generally does not get paid for the extra time involved.
   e. When the project is paid for with public money, then we tend to spend it more freely and to be "more safe" than if it is not public money.
   f. On many projects the owner frequently leaves it up to the designer and is not aware that there is a possibility to save on the design.
   g. Many design firms are not aware of the process of value engineering, and many of those who are aware believe it is used on only public works projects.

WHAT CAN BE DONE ABOUT IT

1. We geotechnical engineers should prepare a document explaining the process of value engineering for public projects, another document for private projects and a third document for smaller projects. These documents should be widely distributed to engineers, contractors and owners.
2. Owners and contractors should be made aware of the possibility of saving money and improving designs by using the value engineering process.
3. When a value engineering study results in savings, the engineer should share in the savings and be paid for the extra time spent in the process.
4. The value engineering study should be done by a competent independent third party and not by the designer, contractor or owner's representative.
5. We geotechnical engineers should be honest with ourselves and look upon each project as if we are spending our own money for it.

REFERENCES

1.OSTERBERG, JORJ O., "The Osterberg Load Test Method for Drilled Shafts and Driven Piles – The first Ten Years". Proceedings of the Seventh International Conference on Piling and Deep Foundations, Vienna, Austria, June 1998. *Published by the Deep Foundations Institute.*

2. OGURA,H., SUMI,M., KISHIDA,H. and YOSHIFUKA,T., (1996) "Application of the Pile Toe Test to Cast-In-Place and Precast Piles." Translated by MADAN KARKEE, Foundation Drilling Magazine, December 1996/January 1997.

3. SCHMERTMANN, JOHN, & HAYES, JOHN (1997), "Observations from Osterberg Cell Tests of Bored Piles." FULCRUM, The Newsletter of the Deep Foundations Institute, Winter '96-'97 pp. 11-14.

4. SCHMERTMANN, JOHN, & HAYES, JOHN, & MOLNIT, THOMAS, & OSTERBERG, J.O., "O-Cell Testing Case Histories Demonstrate the Importance of Bored Pile (Drilled Shaft) Construction Technique" (1998), Proceedings of the Fourth International Conference on Case HIstories, University of Missouri, Rolla, Missouri, March, 1998.

5. OSTERBERG, J.O., (1994) "Recent Advances in Load Testing Driven Piles and Drilled Shafts Using the Osterberg Load Cell Method", Proceedings of the Geotechnical Lecture Series, Geotechnical Division, Illinois Section, American Society of Civil Engineeers, Chicago, Illinois.

6. SCHMERTMANN, JOHN (1993). "The Bottom-Up, Osterberg Method for Static Testing of Shafts and Piles", Proceedings: Progress in Geotechnical Engineering Practice. 13th Central Pennsylvania Geotechnical Seminar, Hershey, Pennsylvania.