Dynamic Passive Pressure on Abutments and Pile Caps

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Quarterly Report Aug-Oct 2006

During this quarter, data reduction and analysis of pile cap load tests associated with Task 3 continued and field testing associated with Task 4 was completed. Preparations for field testing associated with Task 5 began and a commitment was received from Becho, Inc. to construct the additional drilled shafts to provide an adequate reaction force at no cost.

The data analysis associated with the field tests described in Task 3 focused on measurements of the earth pressure on the pile cap face. Earth pressure was measured by two Tekscan pressure sensitive film panels taped to the pile cap face and four conventional earth pressure cells which were embedded in the pile cap face. The pressures measured by these two systems were somewhat different but still reasonably consistent. The measured pressures from both the Tekscan pressure panels and the pressure cells were multiplied by the tributary area associated with each to obtain a plot of passive force on the cap as a function of deflection. The forces obtained in this manner were generally in good agreement with the difference in force measured by the hydraulic actuators with and without backfill soil in place.

Field load tests were performed on pile caps with four different pile/pile cap connection details. A summary of the basic test set-up, test procedure, and test results for the pile cap connection tests associated with Task 4 is provided in this report. In addition, comparisons between measured response and response computed using computer models prior to the field tests are presented. In general, the predicted load-deflection curve for the elastically restrained pile head connection is in remarkably good agreement with the measured curves from the field tests.

Pile Cap Details

All four of the pile caps were 6.5 ft long, 3 ft wide and 3 ft high. Two steel pipe piles driven to a depth of 40 feet and spaced at 3.5 feet on centers were attached to each pile cap using four different connection details. Each pile had an inside diameter of 12 inches with a 3/8 inch wall thickness. Reinforcement in each pile cap consisted of #7 bars spaced at 6 inches in the longitudinal and transverse directions both top and bottom with a minimum of 2 inches of clear cover on the top and 3 inches on the bottom. Holes were cut in the pile so that the longitudinal bars from the bottom reinforcement grid could extend through the piles.

Figure 1 provides a cross section of Test Cap 1 in which the piles were embedded 6 inches into the pile cap. The reinforcement cage in each pile consisted of 4-#6 bars 6.75 ft long confined by 9 inch diameter #4 spirals at a 6 inch pitch. The reinforcing bars extended to the top reinforcement grid in the cap. Test Cap 2 was identical to Cap 1 except that the piles were embedded 12 inches into the cap rather than 6 inches. Test Cap 3 and Test Cap 4 were both constructed without any reinforcement cage to connect the piles to the pile cap. A cross-section for Test Cap 4 is shown in Figure 2 where the pile extended 24 inches into the pile cap. Test Cap 3 was very similar to Test Cap 4 except that the piles only had a 12 inch embedment length. In addition, a steel cap was placed over the top of the piles in Test Cap 4 and they were not filled with concrete as was the case with the other three tests.

Table 1 Summary of details for each pile cap test.

Pile Cap	Embedment	Connection Steel	Cap Steel
1	6 inches	4-#6 bars, 6.75 ft long, #4 Spiral @ 6" pitch	#7 bars @ 6" grids
2	12 inches	4-#6 bars, 6.25 ft long, #4 Spiral @ 6" pitch	#7 bars @ 6" grids
3	12 inches	No connection steel, steel plate over pile top and no concrete in piles, Oregon DOT detail	#7 bars @ 6" grids
4	24 inches	No connection steel but piles were filled with concrete	#7 bars @ 6" grids

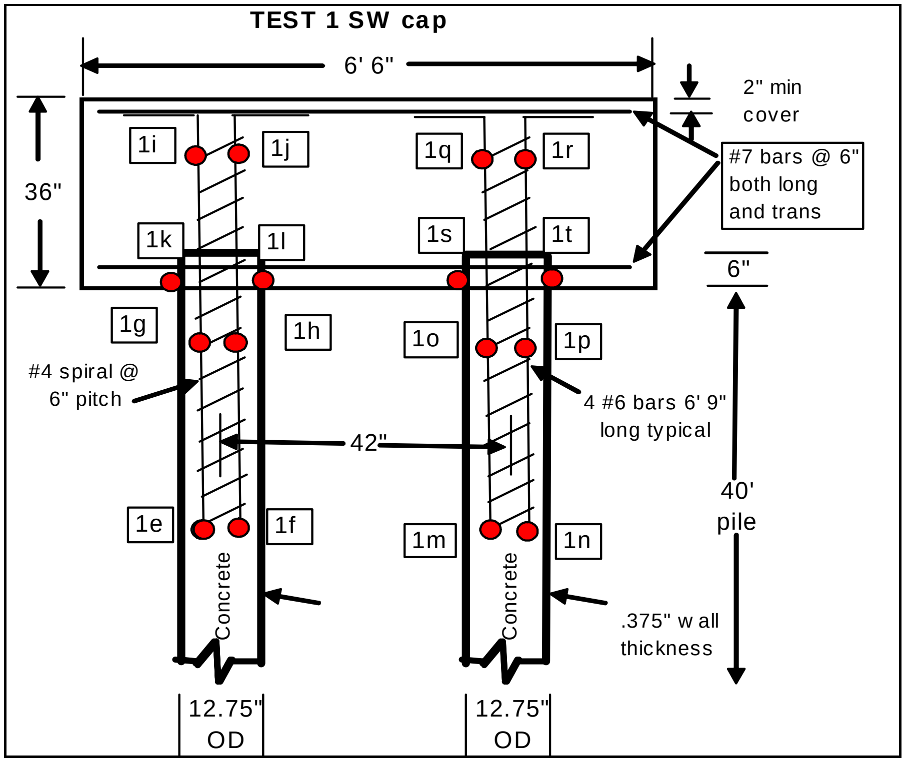

Figure 1 Test Cap 1 details and instrumentation

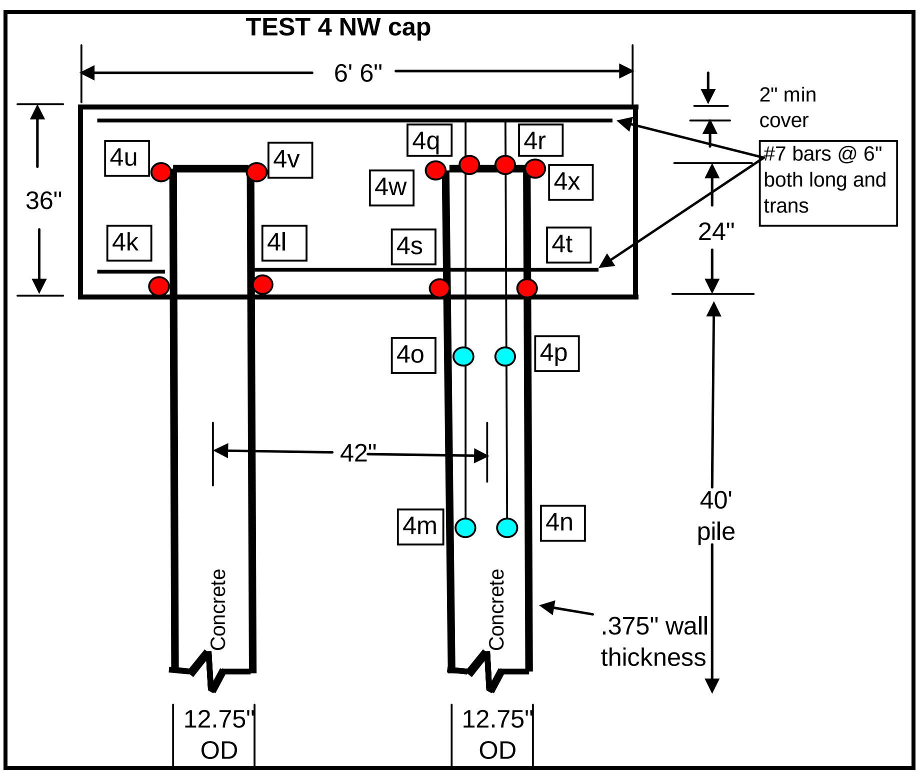

Figure 2 Test Cap 4 details and instrumentation

Instrumentation and Setup

Electrical resistance strain gauges were installed on the reinforcing bars as well as the piles. Figure 1 shows the typical locations of the strain gauges for Test Caps 1 and 2 while Figure 2 shows the strain gauge locations for Test Cap 4. Relatively few gauges were used for Test Cap 3 because there was no reinforcing steel connecting the pile to the pile cap. Six string potentiometers were also installed on the exterior of the each cap to be tested; two on the top face and four on the front face to provide deflection and rotation data. A load cell was attached to the hydraulic ram to measure load and this value was checked against the pressure in the pump. Data was collected from each of these instruments at a rate of 1 reading per second and the results have been collected, reduced and when possible summarized in charts. The reaction for Tests Caps 1 and 2 was provided by a large geopier cap that was about 8 feet from the pile caps, this setup for Test Cap 1 is shown in Figure 3. Test Caps 3 and 4 were directly in line with Test Caps 1 and 2 and were therefore able to utilize the same geopier along with spacers between Caps 1 and 2, for a reaction.

Figure 3 Pile cap test setup and with reaction provided by Geopier cap.

Predictive Analysis

Two computer modeling programs (LPILE and GROUP) were used to simulate pile response to lateral loading prior to the testing program. These programs allow the user to input details such as pile properties, soil parameters and applied load and then provide output consisting of deflections, rotations, moments and shear as a function of load as well as depth. These two models provided valuable data for comparison. Calculations performed using GROUP prior to testing, as will be shown, provided results which proved to be very similar to the observed data collected from the instrumentation. Many of the figures provided subsequently compare the meausred response with the results computed by GROUP.

Test Procedure

Load was applied in an incremental manner and five cycles of loading were applied at each load increment. Initially, a load control approach was employed but repeated load applications produced a significant amount increase in deflection and tended to obscure the virgin load-deflection curve. As a result, a deflection controlled approach was adopted for the remainder of the tests. Deflection increments were typically in ¼ inch at low loads followed by ½ inch increments at higher loads. Each test lasted around 90 minutes because the pile caps and monitoring equipment were inspected between load increments.

Results for Test 1 and Test 2

Test Caps 1 and 2 both had a steel reinforcement cage connection consisting of a #4 spiral with 4 #7 bars longitudinal bar extending to the top reinforcing mat on the pile cap, as described previously. Test Cap 2 had a 12 inch embedment of the piles into the cap while Test Cap 1 only had a 6 inch pile embedment. As expected, these two pile caps performed very similarly with load-deflection curves within 5% of one another. Test 2 was expected to perform slightly better due to the increased pile embedment, yet the test data showed that Test Cap 1 was actually slightly stronger. This could be a result of minor variations in the soil conditions at the site.

Figure 4 provides plots with logarithmic trend linse of the measured load-deflection curves for both Test Caps 1 and 2 as well as the load-deflection curves predicted by GROUP. As shown in Figure 4, analyses were performed with GROUP using three different boundary conditions: fixed, pinned and elastically restrained. Many pile cap analyses assume that the pile cap provides a completely fixed head connection yet this is seldom achievable in the field. A pinned head assumption leads to much greater deflection for a given load which would represent a very conservative design approach. The elastically restrained condition utilizes a rotational stiffness coefficient $Km\theta$ which was estimated using equations developed by Mokwa and Duncan (2003). The elastically restrained condition is believed to be the best approximation to the true field conditions; however, as shown in Figure 4, in this case the results for the elastically restrained boundary condition were almost identical to those for the fixed head condition. The agreement between the measured load-deflection curves for the Test Caps 1 and 2 are both very similar to what was predicted using group for the elastically retrained case.

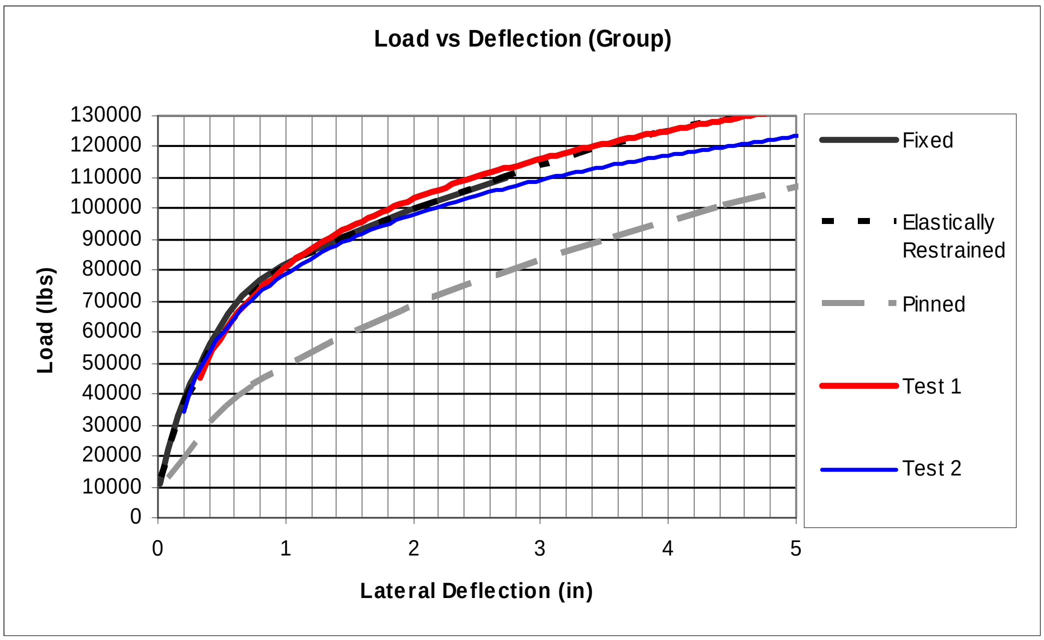

Figure 4 Measured versus predicted load-deflection curves for Test Caps 1 and 2.

While the agreement between measured and computed load-deflection curves was somewhat better for Test Cap 1 than for Test Cap 2, the opposite was true with respect to the amount of rotation. Figure 5 shows logarithmic trend lines of the observed rotation in degrees on the abscissa relative to the applied lateral load in lbs on the ordinate. Because rotation was computed using both string pots on the top and front faces of the pile cap, Test Cap 2 has two separate yet similar measured load-rotation curves. It should be noted that one of the string pots on the top face of Test Cap 1 failed and therefore Test Cap 1 only has an observed rotation from the front face meausrements. Once again, the measured rotational response is very similar to the predicted by GROUP. As shown in Figure 5, relatively little rotation was observed until a lateral load of 80 kips was applied. At this point, both analysis and field observations indicate that the back pile began to experience a pull-out failure. This caused the cap to lift off of soil surface on the back side of the cap and magnified the rotation of the cap.

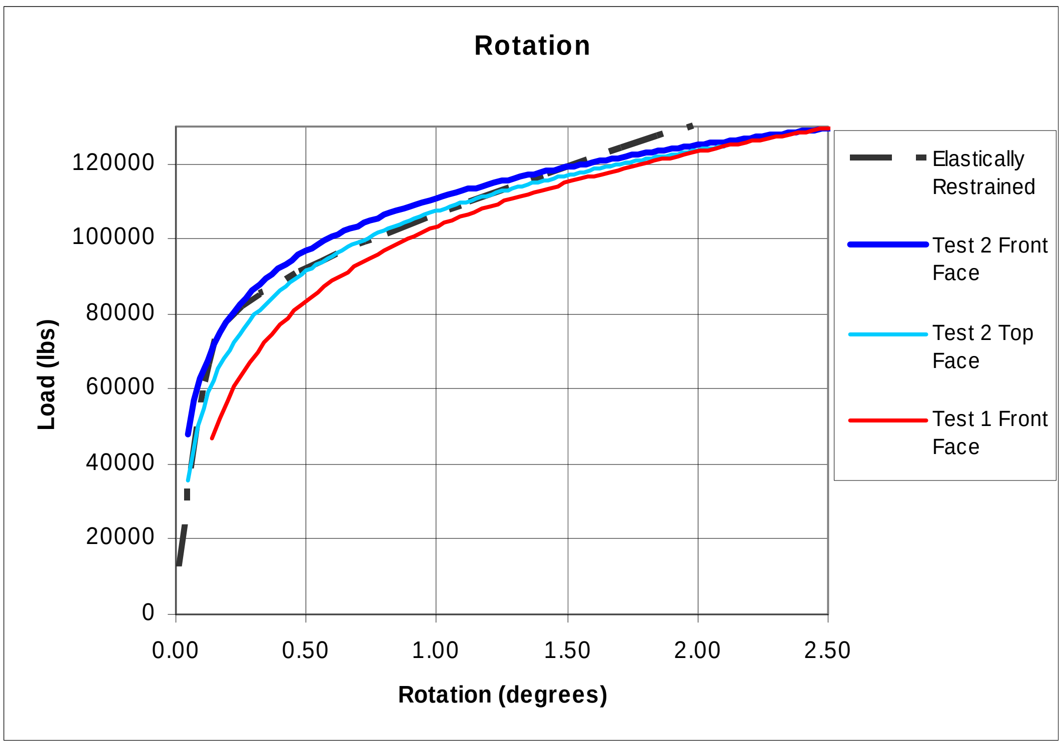

Figure 5 Measured and predicted load-rotation curves for Test Caps 1 and 2.

The picture in Figure 6 shows Test Cap 1 failing by an excessive amount of uplift and rotation. The connection between the pile and pile cap appeared to be adequate as no significant slippage was observed. Test Cap 2 failed in a very similar manner to that described for Test Cap 1.

Figure 6 Uplift and rotation of Test Cap 1 at failure.

Results for Test 3 and Test 4

The first two pile caps tested were typical of pile cap connections employed by Utah DOT in the field. From a cost standpoint, it would be desirable to eliminate the connection reinforcement from the design and construction of future pile caps. This approach has been used by Oregon DOT in their designs. Test 3 and Test 4 were both constructed without a reinforcing cage connecting the piles and pile cap to determine if the connection capacity could be developed along the length of the embedded pile. Test Cap 3 has a 12 inch pile embedment with a hollow steel pile, while Test Cap 4 is embedded 24 inches into the pile cap as shown in Figure 2 with a concrete filled pile. Test Cap 4 performed very similar to Test Caps 1 and 2 and failure occurred in the same way. Test 3 however failed in the connection. Initial cracking shown in Figure 7 began to develop at a lateral load of 80 kips, and as each additional loading cycle was applied, a significant amount of additional cracking developed as shown in Figure 8. With the lateral force being applied in direct line of the connection (one foot above grade), the pile cap experienced a large amount of tensile and shear forces.

Figure 7 Initial cracking Test Cap 3.

Figure 8 Failure of Test Cap 3.

The connection in Test Cap 3 was definitely not adequate; an interesting note is that the observed deflection and rotation was still comparable to those for the other three tests. Figure 8 and Figure 9 show the observed deflection and rotation of all four tests. It is noted that the rotation shown was calculated using an average of that observed from the top and front faces.

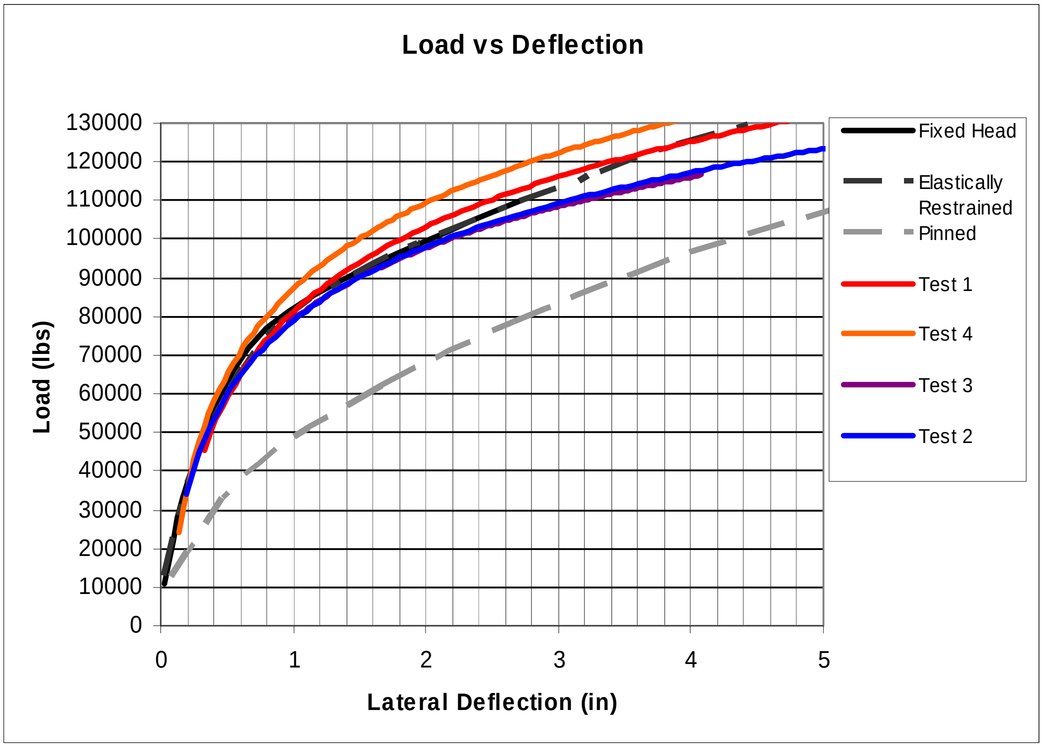

Figure 9 Observed load-deflection curves for all Tests along with curves predicted by GROUP.

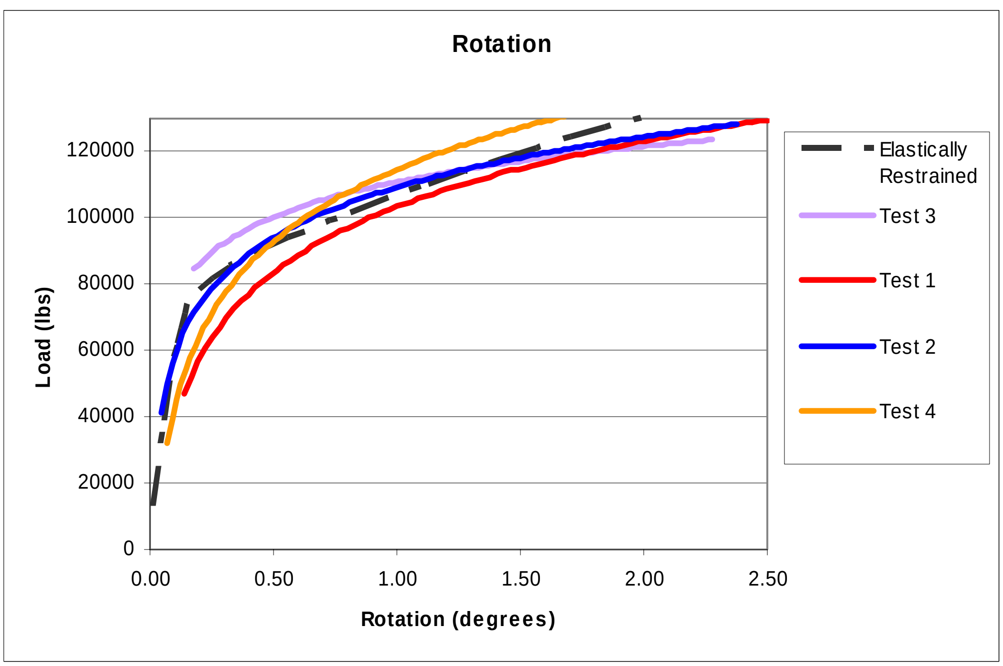

Figure 10 Observed load-rotation curves for all tests along with curves predicted by GROUP.

Preliminary Conclusions for Pile Cap Connection Tests

Test Caps 1, 2, and 4 all appeared to have had an adequate connection while Test Cap 3 was clearly not sufficient. For the Test Caps with a reinforcing cage connection between the pile and the cap, the depth of embedment had relatively little effect on performance and the vertical reinforcing steel was sufficient to maintain the structural integrity of the cap. Even though the load-deflection and load-rotation curves for Test 3 were relatively similar to the others connections, the integrity of the cap was destroyed due to the lack of vertical reinforcing steel. In addition, any type of cracking causes great concern even on a small scale due to the potential for corrosion.

The results from Test Caps 3 and 4 indicate that piles without a reinforcing cage connection can be adequate if embedment is sufficient. Increasing the embedment from 12 to 24 inches provided an adequate connection along with sufficient vertical reinforcement to maintain the structural integrity of the cap.

Plans for the Next Quarter

1. Analyses of the pile cap tests associated with Task 3 will continue and the damping will be determined as a function of displacement level for each test.
2. A final report summary will be drafted on the tests results and analysis for the pile cap tests associated with Task 4.
3. The reaction system necessary to conduct tests associated with Tasks 5 and 6 will be designed and constructed over the next two quarters so that testing can proceed at end of April 2007.

Budget Considerations

At the end of the quarter, $56,508 had been expended on work associated with Tasks 1, 2, 3 and 4. The total budget associated with the project tasks is $265,395. Therefore, approximately 21% of the budget has been spent for these tasks. We estimate that approximately 20% of the work on the project has now been completed. Therefore, the project appears to be on track from a budget standpoint.