Greg Gibson
Vice President, Regulatory Affairs

750 East Pratt Street, Suite 1600
Baltimore, Maryland 21202

UniStar
NUCLEAR ENERGY

10 CFR 50.4
10 CFR 52.79

June 28, 2010

UN#10-177

ATTN: Document Control Desk
U.S. Nuclear Regulatory Commission
Washington, DC 20555-0001

Subject: UniStar Nuclear Energy, NRC Docket No. 52-016
Response to Request for Additional Information for the
Calvert Cliffs Nuclear Power Plant, Unit 3,
RAI 218, Stability of Subsurface Materials and Foundations
Questions 02.05.04-05 and 02.05.04-13

Reference:
1) Surinder Arora (NRC) to Robert Poche (UniStar Nuclear Energy), "FINAL RAI 218 RGS1 4332" email dated March 7, 2010

2) UniStar Nuclear Energy Letter UN#10-105, from Greg Gibson to Document Control Desk, U.S. NRC, RAI No. 218, Stability of Subsurface Materials and Foundations, dated April 7, 2010

The purpose of this letter is to respond to the request for additional information (RAI) identified in the NRC e-mail correspondence to UniStar Nuclear Energy, dated March 07, 2010 (Reference 1). This RAI contains fourteen questions regarding Stability of Subsurface Materials and Foundations as discussed in Section 2.5.4 of the Final Safety Analysis Report (FSAR), as submitted in Part 2 of the Calvert Cliffs Nuclear Power Plant (CCNPP) Unit 3 Combined License Application (COLA), Revision 6. Responses to eight of those questions were provided in Reference 2. In addition, Reference 2 indicated that responses for three of the questions (2.05.04-03, 08, and -13) would be provided by June 30, 2010 and responses for the remaining three questions (2.05.04-05, 09 and 10) would be provided by July 23, 2010.

D096
NRO

The responses to Questions 02.05.04-09 and 2.05.04-10 are tied to the resolution of technical issues that will be answered in the responses to questions in RAI 144 and 145. For these questions, the previous anticipated submittal date of July 23, 2010 remains unchanged.

UniStar Nuclear Energy requires additional time to perform the analyses to finalize the response to RAI Questions 2.05.04-03 and 2.05.02-08. The responses to these questions will be provided with the responses to 02.05.04-09 and 2.05.04-10 by July 23, 2010.

The enclosure to this letter provides the response to RAI 218, Questions 02.05.04-05 and 02.05.04-13 and includes revised COLA content. A Licensing Basis Document Change Request has been initiated to incorporate these changes into a future revision of the COLA.

The enclosed responses do not include any new regulatory commitments. This letter does not contain any sensitive or proprietary information.

If there are any questions regarding this transmittal, please contact me at (410) 470-4205, or Mr. Wayne A. Massie at (410) 470-5503.

I declare under penalty of perjury that the foregoing is true and correct.

Executed on June 28, 2010

Christian Clement

Greg Gibson

Enclosure: Response to NRC Request for Additional Information, RAI No. 218; Stability of Subsurface Materials and Foundations, Questions 02.05.04-05, and 02.05.04-13; Calvert Cliffs Nuclear Power Plant, Unit 3

cc: Surinder Arora, NRC Project Manager, U.S. EPR Projects Branch
Laura Quinn, NRC Environmental Project Manager, U.S. EPR COL Application
Getachew Tesfaye, NRC Project Manager, U.S. EPR DC Application (w/o enclosure)
Loren Plisco, Deputy Regional Administrator, NRC Region II (w/o enclosure)
Silas Kennedy, U.S. NRC Resident Inspector, CCNPP, Units 1 and 2
U.S. NRC Region I Office

GTG/SJS/mdf

Enclosure

Response to NRC Request for Additional Information

RAI No. 218, Stability of Subsurface Materials and Foundations, Questions 02.05.04-05, and 02.05.04-13

Calvert Cliffs Nuclear Power Plant Unit 3

RAI 218

Question 02.05.04-5

Section 2.5.4.5.2 indicates that the excavations will be backfilled with compacted structural fill to the foundation level or, if necessary, lean concrete will be placed as a leveling mat. Since the lean concrete will be used directly underneath the Category 1 structures, please describe the properties of the concrete (such as strength and shear wave velocity), and the criteria that will be used to determine where the lean concrete leveling mat should be used. In addition, describe the controls to ensure that the concrete fill can provide adequate support of both static and dynamic loadings for the foundation.

Response

Concrete fill (also referred to as lean concrete) is comprised of high aggregate and low cement contents. It is used in lieu of compacted structural fill when needed. The properties of concrete fill are typically controlled through controlling its compressive strength. A 28-day compressive strength between 2,000 psi and 3,000 psi is commonly used for concrete fill. A value of 2,500 psi is used for this site.

Two empirical approaches are used to define the shear wave velocity (V_s) of concrete fill. Both approaches are based on a relationship between shear wave velocity and concrete compressive strength. A shear wave velocity for each approach, corresponding to the 2,500 psi strength, is calculated below.

Based on an American Concrete Institute recommendation, for normal weight concrete (unit weight in the range of 140-150 pcf), its modulus of elasticity may be taken as (ACI 349-01):

$$E = 57{,}000\,(f'c)^{0.5} \qquad \text{(Eq. 1)}$$

where

f_c' = compressive strength of concrete in psi
E = elastic modulus of concrete

$$E = 57{,}000\,(2{,}500)^{0.5} = 2{,}850{,}000 \text{ psi} = 410{,}400 \text{ ksf}$$

Based on the theory of elasticity (Bowles):

$$G = E/[2(1+\nu)] \qquad \text{(Eq. 2)}$$

where

G = shear modulus of concrete
ν = Poisson's ratio

The unit weight of non-reinforced concrete is typically in the range of 140-150 pcf. With a Poisson's ratio of 0.15 (Bowles) and unit weight of 145 pcf,

$$G = 410{,}400/[2(1+0.15)] = 178{,}435 \text{ ksf}$$

Also, based on elasticity (Winterkorn and Fang):

$$G = \rho V_s^2 \qquad \text{(Eq. 3)}$$

where

- G = low strain shear modulus (strain level about 10^{-4}%)
- V_s = shear wave velocity
- ρ = density

For concrete, shear modulus is not strain dependent, and thus G can be used as the "low strain" G in Eq. 3 (or G_{max}). Therefore:

$$V_s = (G / \rho)^{0.5} = (178{,}435 \times 32.2/0.145)^{0.5} = 6{,}295 \text{ ft/sec}$$

Correlation is provided between wave velocity and concrete compressive strength (Cho et. al.). Figure 1 defines the relationship among shear wave velocity, compression wave velocity, and compressive strength of concrete. From this figure, for 2,500 psi (about 175 kg/cm^2) strength concrete, shear wave velocity of about 2,300-2,400 m/sec (about 7,500-7,900 ft/sec) is shown.

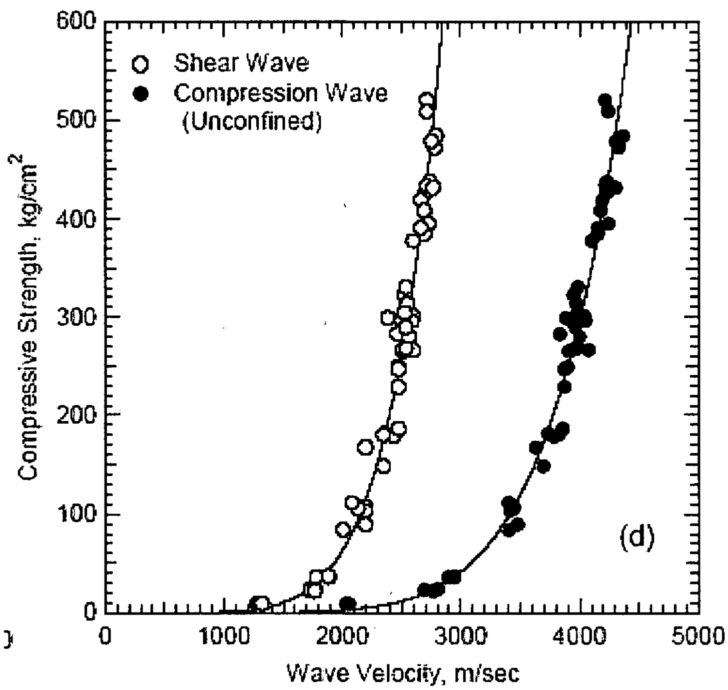


Figure 1: Shear Wave Velocity and Strength of Concrete Material (Cho et. al.)

The two approaches described above indicate V_s values in the range of about 6,000-8,000 ft/sec for 2,500 psi concrete. Therefore, V_s = 7,000 ft/sec is used for concrete fill.

Damping for mass, unreinforced, concrete fill is 1% (Amick, and Monteiro).

As described above, the following properties have been established for the lean concrete:

Compressive strength:	f'c	= 2,500 psi
Modulus of elasticity:	E	= 410,400 ksf
Shear modulus:	G	= 178,435 ksf
Shear wave velocity:	V_s	= 7,000 ft/sec
Damping:		= 1 %

Lean concrete may be used in lieu of soil backfill in the following situations:

- Where the final subgrade elevations are lower than the design foundation elevations, the distance between the as-excavated final subgrade and the design foundation elevation may be filled with lean concrete. This distance may be due to the natural variations in top of foundation bearing material and/or due to removal of unsuitable subgrade soils during foundation excavations.
- Depending on the groundwater condition and where excavations extend below the groundwater level, it may be necessary to place a concrete "plug" at the excavation bottom to counteract excessive upward seepage forces and/or to preserve the integrity of the soil subgrade.
- In restricted spaces, typical soil backfill placement and compaction may not be conducive to rapid execution or obtaining favorable compaction results. Under such circumstances, lean concrete may be used to avoid the difficulties associated with such circumstances.

The adequacy of the concrete fill to support both the static and dynamic foundation loads are addressed through available engineering methods. According to CCNPP Unit 3 FSAR Table 2.5-65, the maximum allowable static and dynamic bearing capacities are 39.3 ksf and 59.0 ksf, respectively.

The design bearing strength of concrete is estimated using ACI standards (ACI 349-01), where:

$$\text{Design bearing strength of concrete} \leq \varphi \times 0.85\, f_c' \qquad \text{(Eq. 4)}$$

where,

φ = reduction factor and is taken as 0.7 for bearing on concrete.

For concrete with 2,500 psi compressive strength, the design bearing strength is about 1,490 psi (210 ksf) which is far greater than the static and dynamic bearing capacities.

Properties of concrete fill are controlled during construction to ensure that concrete with minimum 2,500 psi compressive strength is achieved. Such testing is controlled under ASTM standards (ASTM C39). Details of testing is addressed in project specifications, which include such topics as a concrete mix design to achieve the specified compressive strength, control of heat of hydration by placing relatively thin lifts to minimize cracking as well as control of fly ash content to minimize heat of hydration, methods of sampling and testing, frequency of testing, acceptance criteria, and applicable standards.

References used in this response:

American Concrete Institute (ACI 349-01), Code Requirements for Nuclear Safety Related Concrete Structures 2001.

Amick, H. and Monteiro, P.J.M., (2005) "Modification of concrete damping properties for vibration control in technology facilities," Proceedings, SPIE Conference 5933: Buildings for Nanoscale Research and Beyond, San Diego, CA.

ASTM C39 - 05 "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens," ASTM International, Conshohocken, PA.

Bowles, J.E., Foundation Analysis and Design, 5th Edition, Chapter 2, 1996, McGraw-Hill, NY.

Cho, M., Joh, S., and Kwon, S.A., and Kang, T. (2007) "Nondestructive In-Place Strength Profiling of Concrete Pavements by Resonance Search Technique," Proceedings, Transportation Research Board 86th Annual Meeting, Washington, DC.

Winterkorn, H.F. and Fang, H., Foundation Engineering Handbook, Chapter 24, 1975, Van Nostrand Reinhold Company, NY.

COLA Impact

The following changes will be made in FSAR Section 2.5.4.5.2 to incorporate the information described above. Note that this markup is to the FSAR text provided in UNE letter UN#09-427[1] on October 9, 2009.

2.5.4.5.2 Extent of Excavations, Fills, and Slopes

In the area of CCNPP Unit 3, the current ground elevations range from approximately El. 50 ft to El. 120 ft, with an approximate average El. 88 ft. The finished grade in CCNPP Unit 3 Powerblock Area ranges from about El. 75 ft to El. 85 ft; with the centerline of Unit 3 at approximately El. 85 ft. Earthwork operations are performed to achieve the planned site grades, as shown on the grading plan in Figure 2.5-173. All safety-related structures are contained within the outline of CCNPP Unit 3, except for the water intake structures that are located near the existing intake basin, also shown in Figure 2.5-173. Seismic Category I structures with their corresponding foundation are:

- Nuclear Island Common Basemat (El. 41.5).
- Emergency Power Generating Building (El. 76).
- Essential Service Water Buildings (El. 61.0).
- Ultimate Heat Sink Makeup Water Intake Structure (El. -26.5).
- Ultimate Heat Sink Electrical Building (El. -10.5).

Excavation profiles (at the corresponding locations shown in Figure 2.5-103, Figure 2.5-104, and Figure 2.5-105) are shown in:

- Subsurface and excavation profile Powerblock Area A-A': Figure 2.5-160.
- Subsurface and excavation profile Powerblock Area B-B'; Figure 2.5-161.
- Subsurface and excavation profile Powerblock Area C-C': Figure 2.5-162.
- Subsurface and excavation profile Powerblock Area 0-0': Figure 2.5-163.

1 G. Gibson (UniStar Nuclear Energy) to Document Control Desk (U.S. NRC), "Update to Calvert Cliffs Nuclear Power Plant, Unit 3 FSAR Sections 2.5.4 and 2.5.5," Letter UN#09-427, dated October 9, 2009.

- Subsurface and excavation profile Powerblock Area E-E': Figure 2.5-164.
- Subsurface and excavation profile Intake Area F-F': Figure 2.5-165.

These figures illustrate that excavations for foundations of Seismic Category I structures will result in removing Stratum I Terrace Sand and Stratum IIa Chesapeake Clay/Silt in their entirety, and will extend to the top of Stratum lib Chesapeake Cemented Sand, except in the Intake Area. In the Intake Area, the foundations are supported on Stratum IIc soils, given the interface proximity of Strata lib and IIc.

The depth of excavations to reach Stratum IIb is approximately 40 ft to 45 ft below the final site grade in the Powerblock Area. Since foundations derive support from these soils, variations in the top of this stratum were evaluated, reflected as elevation contours for the top of Stratum IIb in CCNPP Unit 3 and in CLA areas, as shown in Figure 2.5-174. The variation in top elevation of these soils is very little, approximately 5 ft or less (about 1 percent) across each major foundation area. The extent of excavations to final subgrade, however, is determined during construction based on observation of the actual soil conditions encountered and verification of their suitability for foundation support. Once subgrade suitability in Stratum IIb soils is confirmed, the excavations are backfilled with compacted structural fill ~~to the foundation level of structures~~ or, if necessary, lean concrete is placed in lieu of structural fill~~as a leveling mat~~.

The properties of lean concrete are controlled through controlling its compressive strength. A minimum 28-day compressive strength of 2,500 psi is used. Properties of lean concrete are controlled during construction. Detailed project specifications include requirements for mix design, placement, sampling and testing, frequency of testing, applicable standards, and acceptance criteria. Lean concrete may be used in lieu of structural fill in the following cases: below the foundations as leveling mats, to counteract seepage forces at the bottom of the excavation and to help preserve soil subgrade integrity, and in restricted spaces to expedite construction.

Subsequent to foundation construction, the structural fill is extended to the final site grade, or near the final site grade, depending on the details of the final civil design for the project. Compaction and quality control/quality assurance programs for backfilling are addressed in Section 2.5.4.5.3.

. . .

RAI 218

Question 02.05.04-13

Sections 2.5.4.5.4 and 2.5.4.10.2.2 indicate that monitoring program specifications for foundation rebound (heave) and settlement will be developed during the detailed design stage of the project. Since foundation rebound and settlement are expected at the site, and estimated differential settlement of the reactor building will exceed the standard design criterion, please provide a detailed description of the monitoring program including all basic elements, such as the settlement monitoring bench marks, locations of instruments, monitoring and recording frequency, and evaluation of the magnitude of rebound and settlement during and after excavation and construction.

Response

FSAR Section 2.5.4.10.2.2 will be revised to include a more detailed discussion of settlement monitoring.

COLA Impact

The subsection entitled Settlement Monitoring in FSAR Section 2.5.4.10.2.2 will be revised as shown. Note that this markup is to the FSAR text provided in UNE letter UN#09-427[2] on October 9, 2009.

Settlement Monitoring

~~Heave or rebound of the excavation bottom, the effect of dewatering and the effect of Nuclear Island basemat loading during construction will be monitored. This is necessary to confirm that the rate of settlement is consistent with the estimates. A settlement monitoring program will be developed during detailed design. The settlement monitoring program will consist of three primary elements:~~

- ~~Piezometers to measure pore pressures in Stratum IIb and IIC. Vibrating wire piezometers are preferred for this purpose as they are adequately sensitive and responsive and easily record positive and negative changes on a real time basis.~~
- ~~Settlement monuments placed directly on concrete, preferably on the mud mat and on the corners of the structures at grade that are accessible with conventional surveying equipment.~~
- ~~Settlement telltales if monuments are not practical or if fills are used over consolidation type soils and it is necessary to monitor settlement of the consolidation type soils independent of the consolidation of the fill. Telltales can be used after backfill is placed,~~

~~Monitoring locations will be distributed at corners of facilities and throughout the perimeter of the Nuclear Island. Monitoring points will be placed to relate settlement measurements to sections (such as those indicated by Figure 2.5-192) so that actual settlement can be compared directly to model results.~~

~~Plots showing Movement (settlement or heave) versus Time will be maintained along with Estimated Load versus Time curves.~~

---

2 G. Gibson (UniStar Nuclear Energy) to Document Control Desk (U.S. NRC), "Update to Calvert Cliffs Nuclear Power Plant, Unit 3 FSAR Sections 2.5.4 and 2.5.5," Letter UN#09-427, dated October 9, 2009.

A settlement monitoring program will be enforced to record heave of the excavation bottom, the effect of dewatering and the effect of Nuclear Island Basemat loading during and after construction. This is necessary to confirm that the estimated rate of heave and settlement is consistent with the field observations. The purpose of this monitoring program is to assess and document the actual settlements in comparison with the predicted and the acceptable limits. The settlement monitoring program consists of three primary elements:

- Piezometers to measure effects of dewatering and pore pressures in a soil layer prone to consolidation type settlement. Vibrating wire piezometers are preferred for this purpose as they are adequately sensitive and responsive and easily record positive and negative changes on a real time basis. Piezometers should be screened in Stratum II-B (Chesapeake Cemented Sand) and Stratum II-C (Chesapeake Clay/Silt)
- Settlement monuments placed directly on concrete, preferably on the Mud Mat and on the corners of the structures at grade that are accessible with conventional surveying equipment.
- Settlement sensors and extensometers if monuments are not practical or if fills are used over consolidation type soils and it is necessary to monitor settlement of the consolidation type soils independent of the consolidation of the fill.

The instrumentation plan for the Powerblock Area of the site will consist of horizontal settlement sensors, Vibrating Wire (VW) piezometers, surface monuments, concrete anchored monuments, extensometers and one accelerometer. The definitive number of instruments needs to be established during design stages of the monitoring system. The tentative locations of the instruments are shown on Figure 2.5-212.

Tested and calibrated settlement sensors will be used to monitor settlement and heave within the excavation footprint. Settlement sensors will be installed at the bottom of the proposed foundation (bench mark El. 40) before the excavation of the Powerblock Area is started. The sensors will be placed at the approximate locations shown on Figure 2.5-212 and the required cables will be routed away from the fill area.

The settlement sensors have two important components, the sensor and the reservoir. The sensor will be located inside the limits of the structural backfill while the reservoir is located outside the fill limits in a borehole attached by a Borros anchor (Dunnicliff, 1988). The reservoir needs to be located on stable ground because it reads difference in settlement between the reservoir and sensor. The wires connecting the sensor to the reservoir are suited for direct burial. The wires shall be buried below the frost line for protection and to minimize temperature differentials that could result in erroneous settlement or heave measurements.

Figure 2.5-212 shows a tentative distribution and placement of VW piezometers to be installed around the Perimeter of the Powerblock Area. The VW piezometers will be used to measure ground water elevations and associated changes in pore pressure during dewatering, excavation, structural backfill placement, and plant construction.

Extensometers shall be installed in the Powerblock Area. These will be installed adjacent to the Reactor building, bench mark elevation 41.5, adjacent to the Turbine building, adjacent to the Essential Service Water Building (ESWB) Nos. 1, 2, and adjacent to ESWB Nos. 3 and 4. At least one extensometer will be installed adjacent to the Radioactive Waste Processing Building. The bench mark for the Turbine Building, ESWB and Radioactive Processing Building is El. 59.5. The extensometers shall be calibrated rod type borehole extensometers. The extensometers will either be protected by raising the standpipe out of the ground approximately one foot or by placing the extensometer approximately 10 to 12 inches below top of the ground surface.

After the structural backfill has been placed to the final grade, Surface Monuments (SM), bench mark El. 80 shall be placed on the surface of the backfill at approximate locations shown on Figure 2.5-212. The monuments shall consist of a one foot diameter concrete cylinders placed a minimum of three feet below final grade and be fitted with a brass dome cap with a point for survey use.

On the side of foundation mats, no later than 28 days after construction, National Geodetic Survey (NGS) (USDC, 1978) survey disks will be placed by drilling a cavity on the side of foundation mats. The cavity will be backfilled with a mortar mix and the survey disk will be anchored into the foundation mat. The disk needs to be located at strategic points of the mat and have a direct view to a benchmark or to other survey points that can relate to a benchmark.

One accelerometer shall be installed to record any seismic events that occur during or after construction. The accelerometer shall be placed within the mat foundation of the Reactor Building.

The Instrumentation Plan for the Makeup Water Intake Structure (MWIS) will consist of settlement sensors, extensometers and one accelerometer. Tentative location of these instruments is shown on Figure 2.5-213. Calibrated settlement sensors will be used to monitor settlement and heave within the excavation footprint of the UHS. Extensometers will be installed adjacent to the Circulating Water Makeup Intake Structure and adjacent to the UHS Makeup Water Intake Structure. The bench mark for the extensometers is El. -26.5. The extensometers shall be calibrated rod type borehole extensometers. The extensometers will either be protected by raising the standpipe out of the ground approximately one foot or placing the extensometer approximately 10 to 12 inches below top of the ground surface. Finally, one accelerometer shall be installed to record any seismic events that occur during or after construction. The accelerometer shall be placed within the foundation of the MWIS.

Each instrument will be read to determine baseline conditions after installation. For the settlement sensors, the baseline readings will be taken before any site earthwork has been performed. The baseline survey should be completed with a minimum of three different readings taken over several days to verify that the readings have stabilized.

Each instrument should be read at least twice a day in the initial stages of this project. During later stages of the project, the reading frequency may be adjusted to once per day and longer at the discretion of the Engineer.

Plots showing movement (settlement or heave) versus time should be maintained during construction, along with estimated load versus time curves. The site should remain dewatered until the curves go asymptotic, at which time connections between buildings can be made. Monitoring should continue after these connections are made in order to assure asymptotic conditions. After construction is completed, all instruments will be monitored for at least one year. At that time, the Engineer will define frequency and instruments to maintain a long-term monitoring program.

The following references will be added to FSAR Section 2.5.4.13 References:

Dunnicliff, 1988, Geotechnical Instrumentation for Monitoring Field Performance, John Dunnicliff, John Wiley & Sons, Inc., 1988.

USDC, 1978, National Oceanic and Atmospheric Administration (NOAA) Manual NOS NGS 1, Geodetic Benchmarks, U.S. Department of Commerce, 1978.

Figure 2.5-212 – {Settlement Monitoring Instrumentation in the Power Block Area}

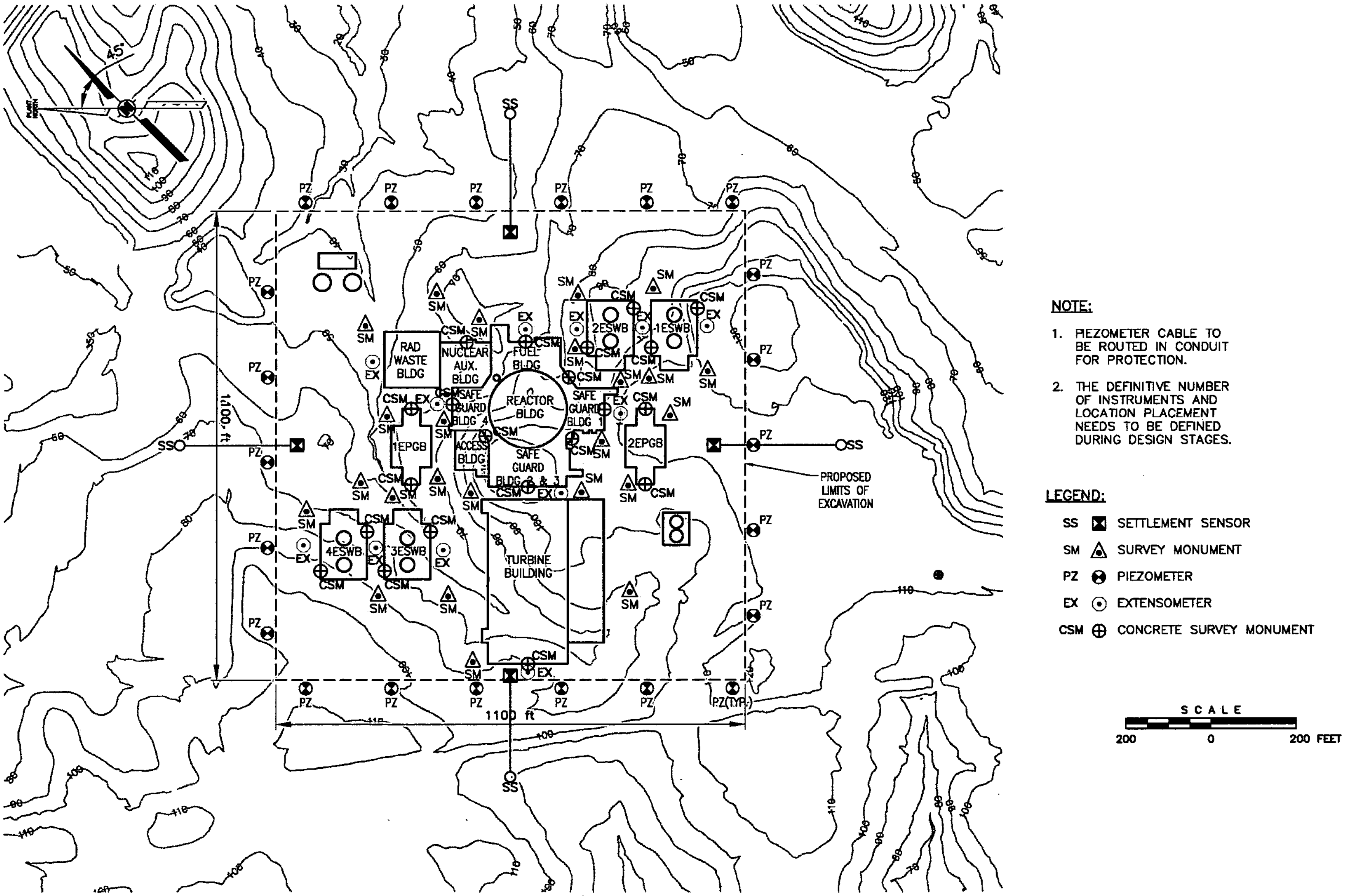
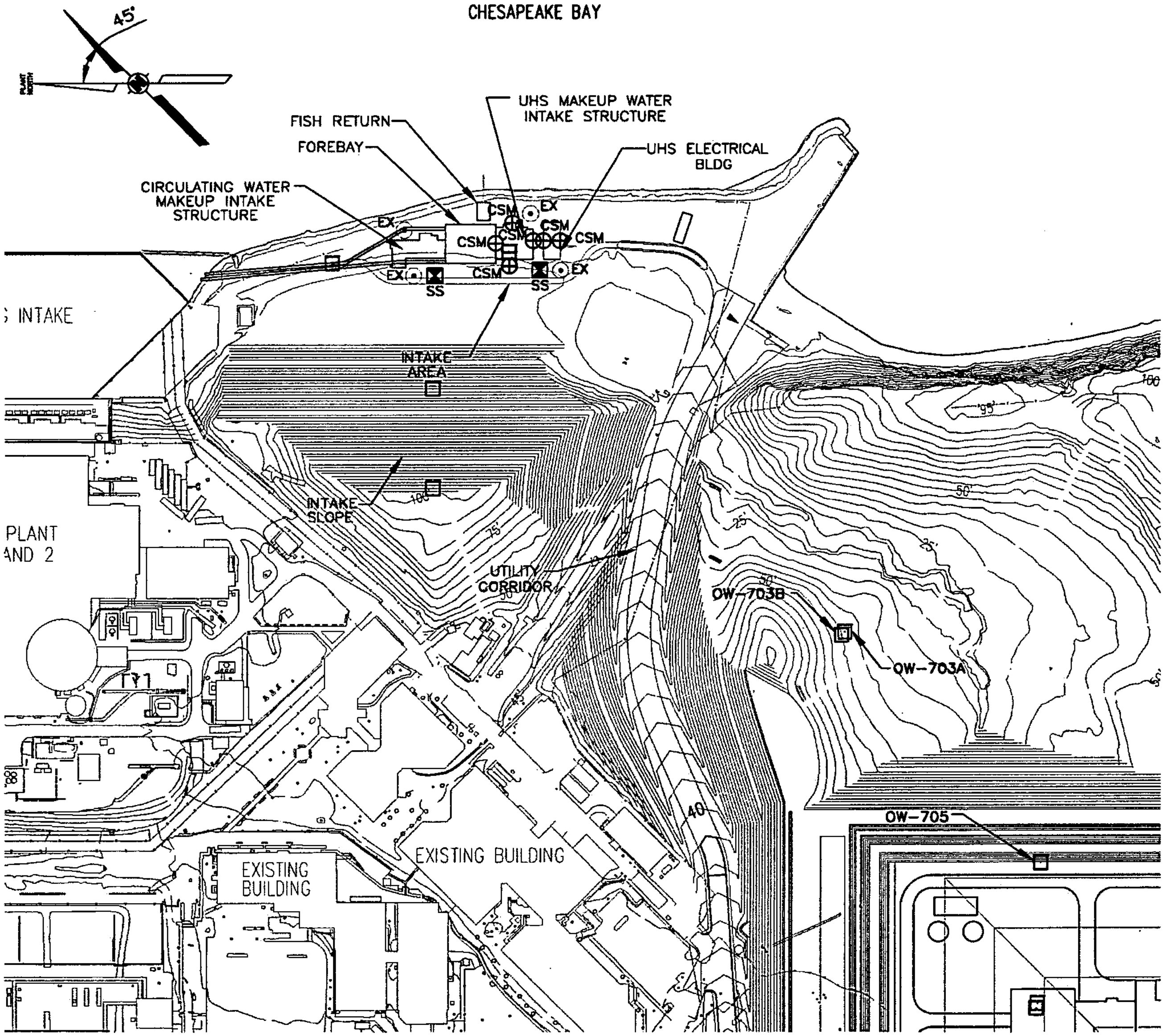


Figure 2.5-213 – {Settlement Monitoring Instrumentation at the Intake Structure}



NOTE:

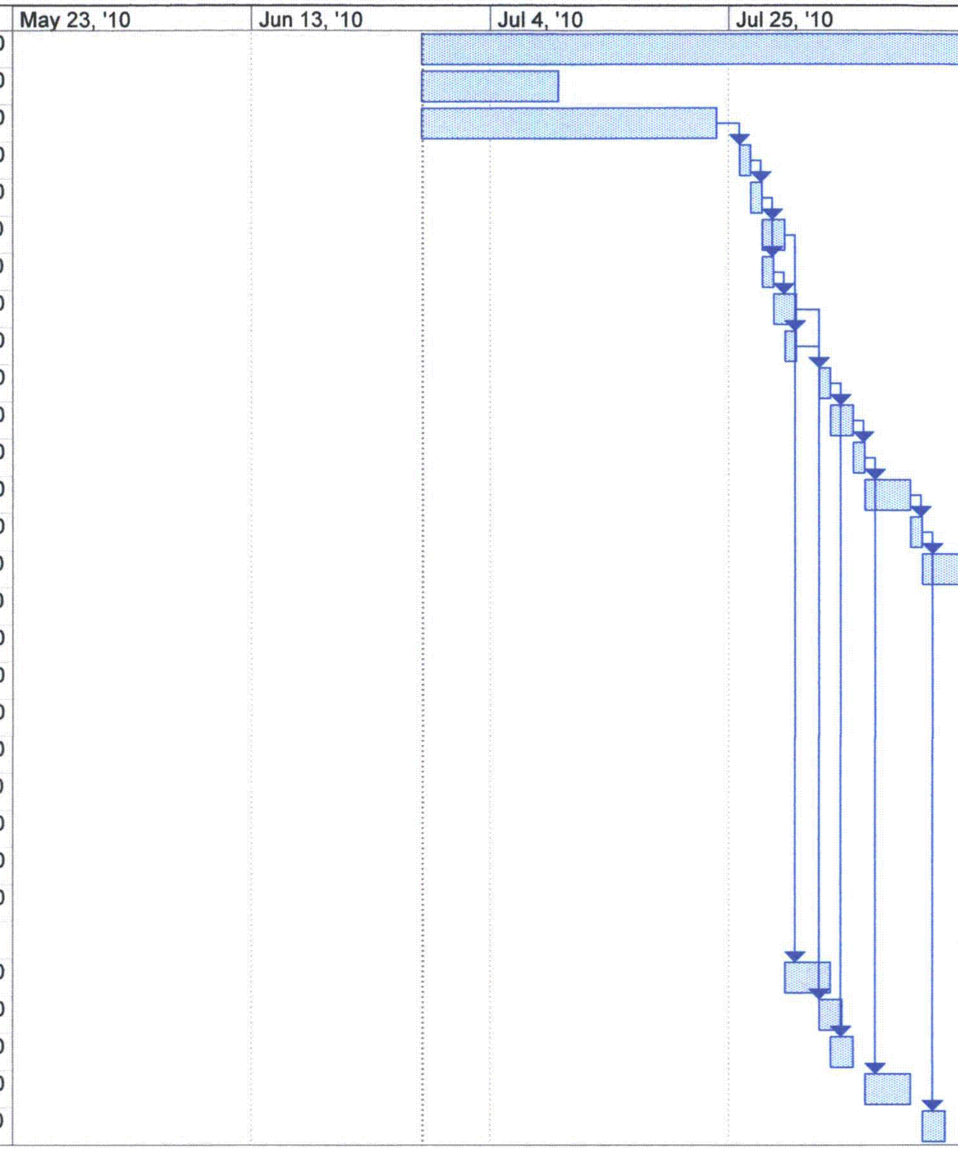
1. PIEZOMETER CABLE TO BE ROUTED IN CONDUIT FOR PROTECTION.
2. THE DEFINITIVE NUMBER OF INSTRUMENTS AND LOCATION PLACEMENT NEEDS TO BE DEFINED DURING DESIGN STAGES.

LEGEND:

SS SETTLEMENT SENSOR
EX EXTENSOMETER
OW-703A OBSERVATION WELL (A&B INDICATE WELL PAIR)
CSM CONCRETE SURVEY MOMENT

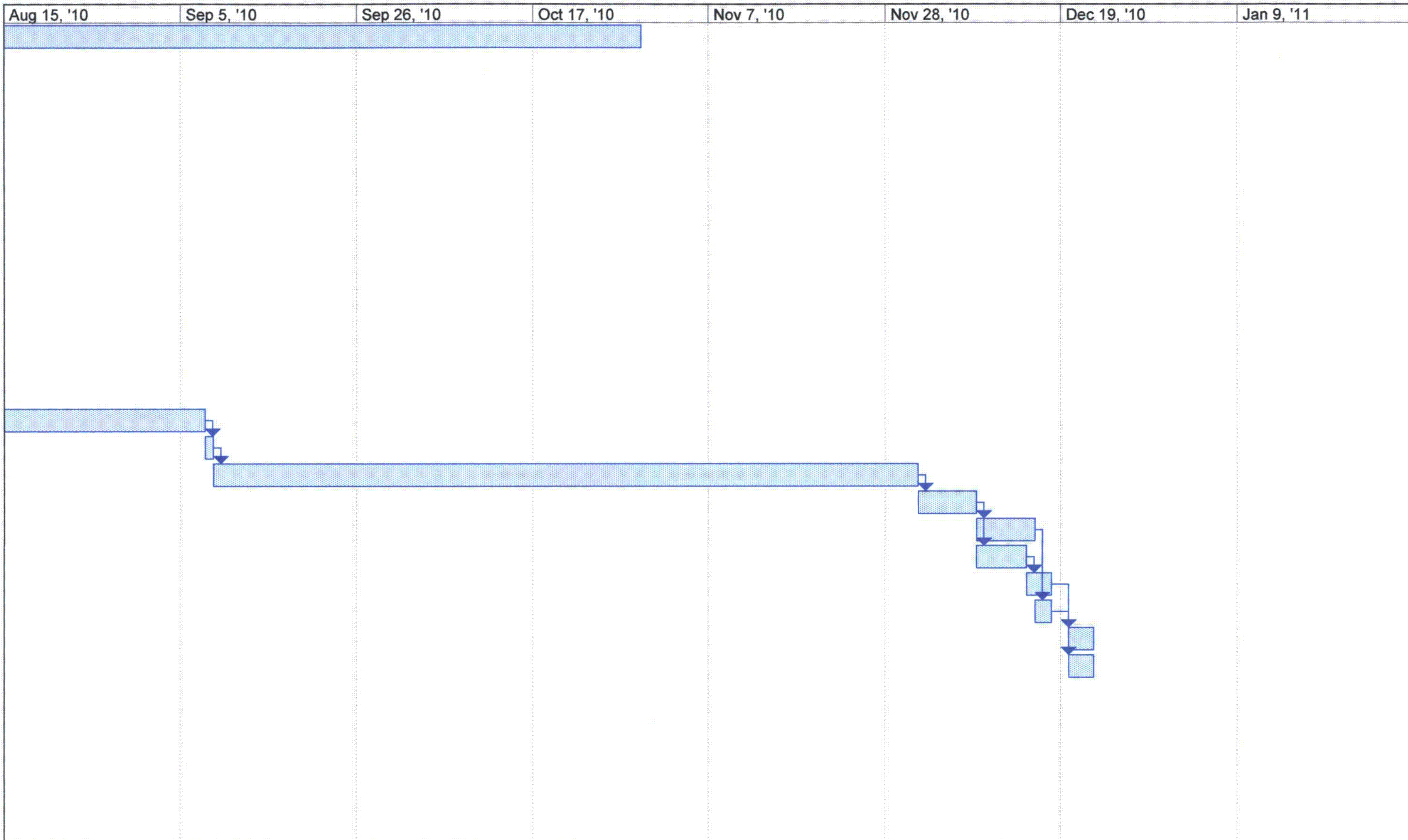
SCALE
200 0 200 FEET

ID		Task Name	Duration	Start
1		Publish CC3 Rev7	90 days	Mon 6/28/10
2		Establish Implementation Rate	10 days	Mon 6/28/10
3		Implement all GN LBDCR	20 days	Mon 6/28/10
4		FSAR 3.5 I GN -09-382LBDCR	1 day?	Mon 7/26/10
5		FSAR 2.2 BB 10-0043 LBDCR	1 day	Tue 7/27/10
6		Publish BB FSAR 2.2	2 days	Wed 7/28/10
7		FSAR 3.3 BB 09-0041/0382 LBI	1 day	Wed 7/28/10
8		Publish BB FSAR 3.3	2 days	Thu 7/29/10
9		FSAR 3.5 BB 10-0081 LBDCR	1 day	Fri 7/30/10
10		Publish BB FSAR 3.5	1 day?	Mon 8/2/10
11		FSAR 2.3 BB 10-0004/0042/09-3	2 days	Tue 8/3/10
12		Publish BB FSAR 2.3	1 day?	Thu 8/5/10
13		FSAR 4.5 BB 10-0004/09-0353/(	2 days	Fri 8/6/10
14		Publish BB FSAR 4.5	1 day?	Tue 8/10/10
15		Implement remaining BB LBDCR	20 days	Wed 8/11/10
16		Publish All BB LBDCRs	1 day?	Wed 9/8/10
17		Implement all CC3 LBDCR	60 days	Thu 9/9/10
18		Implement all GN DC LBDCR	5 days	Thu 12/2/10
19		Implement all BB DC LBDCR	5 days	Thu 12/9/10
20		Implement all CC3 DC LBDCR	4 days	Thu 12/9/10
21		Issue Living CC3 COLA	3 days	Wed 12/15/10
22		Issue Living BB COLA	2 days	Thu 12/16/10
23		Publish CC3 Rev 7	3 days	Mon 12/20/10
24		Publish BB Rev 7	3 days	Mon 12/20/10
25				
26		Submit BB FSAR 2.2	2 days	Fri 7/30/10
27		Submit BB FSAR3.3	2 days	Mon 8/2/10
28		Submit BB FSAR 3.5	2 days	Tue 8/3/10
29		Submit BB FSAR 2.3	2 days	Fri 8/6/10
30		Submit BB FSAR 4.5 & 6.4	2 days	Wed 8/11/10



Project: COLA Publishing Project CC3
Date: Mon 6/28/10

Task
Split
Progress
Milestone
Summary
Project Summary
External Tasks
External Milestone
Deadline



Project: COLA Publishing Project CC3
Date: Mon 6/28/10

Task		Milestone		External Tasks	
Split		Summary		External Milestone	
Progress		Project Summary		Deadline	