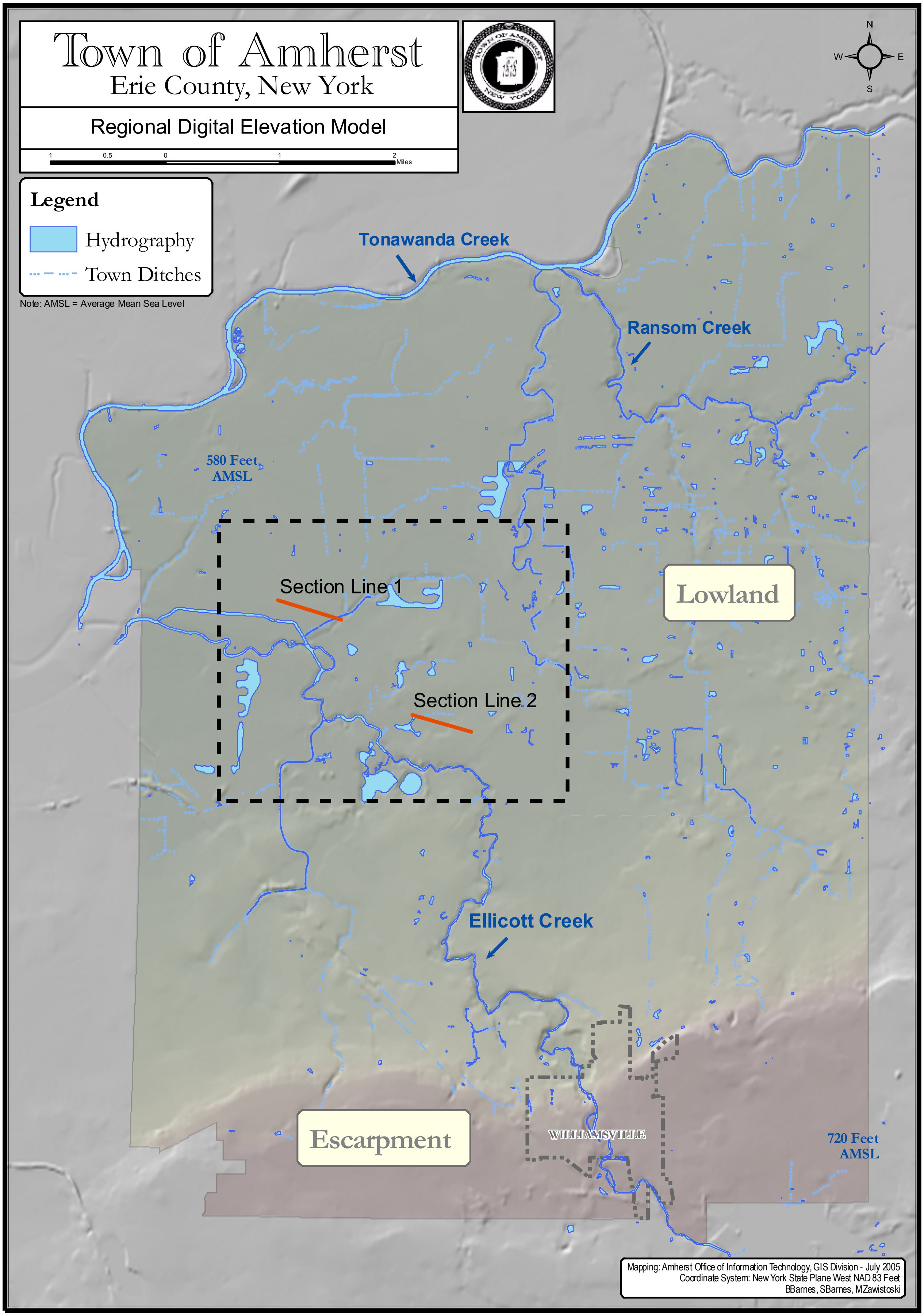
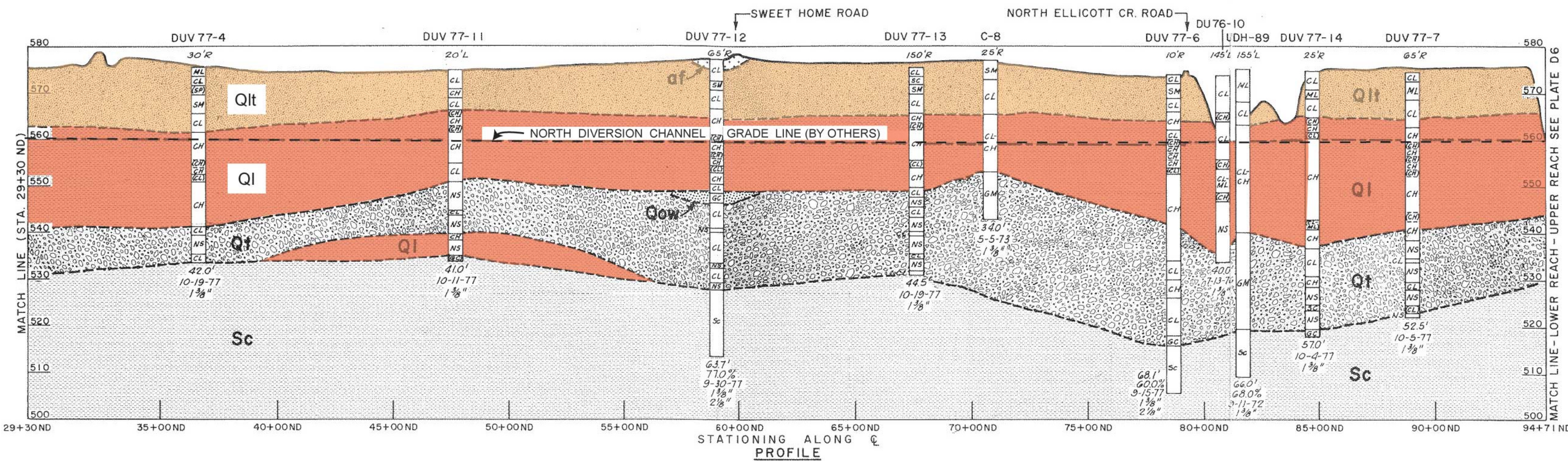
SECTION 6 – APPENDICES

6.1 GEOLOGIC CROSS SECTION

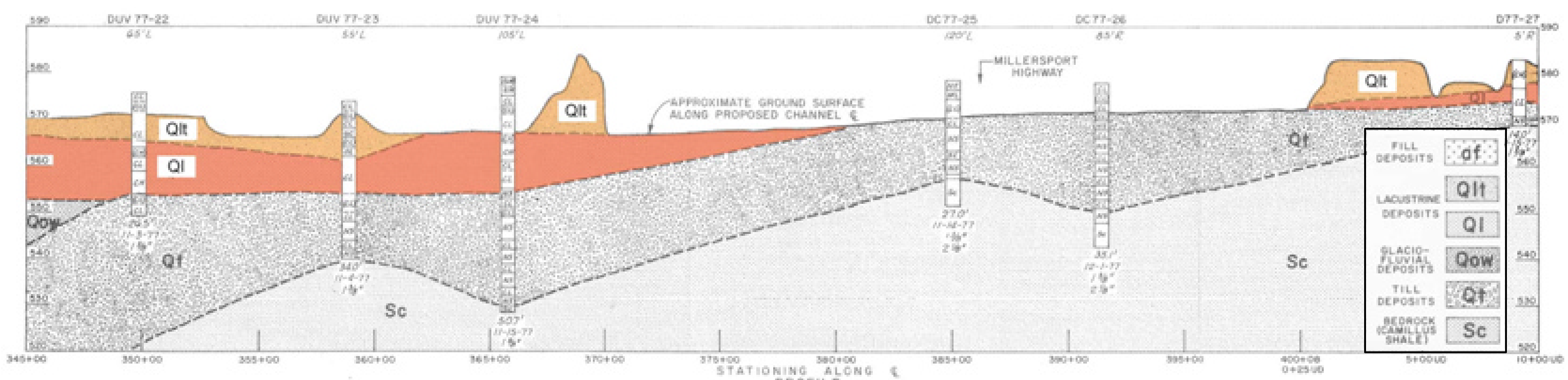


Appendix 6.1.1: Approximate location of cross sections shown in Appendix 6.1.2.

Section Line 1



Section Line 2



Appendix 6.1.2. Geol ogic cross sections along Ellicott Creek (after USACE, 1979)

6.2 MINERALOGICAL AND REMOTE SENSING REPORTS

Report to USACE
Mineralogy of Amherst Drill Core Samples

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Aim: All the homes in Amherst which have suffered damage due to differential movement and/or sinking, are associated with glacial lacustrine clays, either near surface or at some depth. To understand the mechanisms which have damaged these homes, it is necessary to understand the nature of the clay and particularly the minerals present in the clay. This part of the study program is aimed at elucidating the quantitative mineralogy of these materials by analysis using X-ray diffraction.

Macroscopic Description: The clays typically are a dark red in color and are for the most part very plastic. In many localities, the clay underlies a relatively firm silty clay. The boundary between the two is often sharp, occurring over a space of only a few feet. The boundary may be even more pronounced than the data indicate because the measurement of the geotechnical properties necessarily is averaged over several feet, thereby potentially blurring the boundary. The samples we studied were taken from drill core samples supplied principally by Earth Dimensions, Elma NY. Some samples were also supplied by USACE.

Mineral Identification; General: Given that the clay underlying much of Amherst was deposited in a lake or a series of lakes created as the last glacial advanced receded, one would expect a mineralogy reflecting the erosion of the pre-glacial terrain. This erosion would have generated quartz, calcite, various clay minerals, and dolomite as the principal constituents. This supposition was easily verified by the first X-ray diffraction patterns made with clay samples.

Mineral Identification; Specifics: X-ray diffraction is the preferred analytical tool for identifying the minerals present in a soil or rock or sediment. Each mineral has a characteristic diffraction pattern and these have been cataloged for many decades. The collection of such data has resulted in a single data base published and maintained by the ICDD (International Centre for Diffraction Data, Newtown Square, PA). In our laboratory, the diffractometer is a Siemens D500 with is a digitally recording unit run by a PC via software written by MDI (Materials Data Inc., 1224 Concannon Blvd., Livermore, CA). The software has two functions: one is to drive the D500 (i.e., step scan, count for a specified time, and record the intensity for each step) and the second is a package that allows graphical presentation of the diffraction data and analysis of the total pattern by referral to the ICDD data base.

Having identified the minerals present, the next step for our project was to quantify the abundance of each mineral present is a single sample. This is not a trivial task. The difficulty comes from the fact that while one can record the diffraction of a mineral, say quartz, the data recorded are not absolute values, they are relative to an unknown intensity. If we examined a mixture of two minerals, we would have information about the diffraction intensities of one mineral with reference to another, but again there is no absolute reference. One option is to calculate the diffraction pattern, point by point, using the known information about the structure of each mineral. The calculated pattern is compared to the observed pattern and the proportions of each constituent are adjusted to get the best fit of calculated and observed patterns. This latter is typically referred to as the Rietveld refinement process. The problem with the Rietveld approach for the Amherst clay study is that at present it is not possible to include clay minerals in the Rietveld refinement because clays are very disordered materials and we do not have the ability to calculate the diffraction patterns for these materials.

An alternative is to add a known amount of a standard to the sample before recording the diffraction pattern. Our work utilized ZnO as the internal standard. In addition, rather than attempting to calculate diffraction patterns, one can use a library of standard patterns. These can we added to the calculated diffraction pattern and the proportions adjusted to yield the best fit with the observed pattern. This approach has been utilized by the software package ROCKJOCK written by Dr. D. Eberl at the USGS in Boulder, CO.

Sample Preparation: In order to obtain an accurate estimate of the weight percents of the minerals in a sample, the sample must be properly prepared and packed in the sample holder prior to running on the D500. One of the major problems in powder diffraction is the coarseness of the powder and the non-uniformity of the particle size distribution of the powder. The ideal would be to have a powder of micron-sized particles all of the same size. This ideal can be approached rather well using a Micronizing Mill (McCrone Associates, 850 Pasquinelli Drive, Westmont, IL). The mill using a vibrating cylindrical container which is loaded with smaller cylindrical abrasive grinders fabricated from aluminum oxide. The sample is dispersed in methanol and placed in the interstices of the grinding elements. When activated, the grinding elements rub past each other at low velocity thereby grinding the sample uniformly. The addition of a few ml. of methanol ensures that the sample is not appreciably heated by the grinding.

When the grinding is finished, the sample and methanol is drained from the container and the remaining sample is washed out with excess methanol. The resulting dispersion is slowly evaporated; the solid powder is now in a cake form which must be broken apart mechanically. This can be done by rubbing the cake against a 40 mesh screen. The powdered sample is then loaded into a sample holder so as to minimize any preferred orientation. This is especially important for clay minerals and minerals with a pronounced cleavage such as calcite and dolomite.

The sample holders we have used are of two types. One loads the sample from the side into a cavity formed by the sample holder and a frosted glass slide which is later removed. The second is a back loading sample holder. Here the holder is a plastic plate with a suitably sized hole drilled completely through. This is placed on a frosted glass slide and the sample loosely fills the cylindrical cavity. A plunger then forces the powder further into the hole creating a “solid” powder plug.

The sample holder is then placed in the D500 and the pattern is scanned from 5 to 65º 2? in a step of 0.02º 2? with a count time of 2 sec per data point. The resulting data file is transformed into the correct format for analysis in ROCKJOCK.

Computer Analysis: It is necessary for the analysis to know which minerals are actually in the sample. Initially, the identification was accomplished by using the search/match software in the D500 package. The software does not reliably identify minor components of the sample. We proceeded to use ROCKJOCK with the major minerals (quartz, illite, and chlorite) as the input. The residual, i.e. the peaks not accounted for by the three minerals listed above are due to the as yet unidentified minerals. We investigated each of these peaks using our knowledge of the diffraction patterns of common minerals along with a knowledge of the local rocks. The secondary minerals are pyrite, calcite, dolomite, and feldspar. We feel that we have identified all minerals present at levels greater than 1 wt%.

Results:

Clay minerals: The illite present is a mixture of the $2M_1$ and 1Md. The quantities of illite vary from about 11 wt% to as much as 40 wt%. All samples examined contained illite. There are several kinds of chlorite; the exact number of different chlorites is not presently clear. The quantity of chlorite varies from about 7 wt% to about 16 wt%.

Non-clay minerals: Quartz is present in all samples at levels varying between about 17 wt% to as much as 43 wt%. The quartz is very fine grained and may be coated with organic matter. This would explain the high plasticity of the samples. The feldspars are in the range of 10 wt%, calcite varies between 0 to about 24 wt%, dolomite varies between 0 and 14 wt%, and pyrite is present at less than 1 wt%.

Conclusions: The plasticity of the samples we examined is due to the high water content associated with fine-grained silicate and other minerals especially illite. Illitic-soils are known to undergo shrink-swell behavior as a function of water content so that drying out the wet clay soil will result in a marked shrinkage which may partly be reversible when the soil is re-wetted. This behavior would go a long way toward explaining the damage to a large number of houses in the Amherst area.

Table 3.1 - Laboratory Test Results for Backfill Soil Samples

0	1	2	3	4	5	6	7	8	9	10	11
UB #	SITE	SAMPLE DEPTH (FEET)	POTENTIAL EXPANSION (ASTM D4829)	EXPANSIVE? (Residential Code of NYS, Section R403.1.8.1)	%1Md Illite	%2M1 Illite	%Chlorite Tusc.	%Quartz	%Calcite	%Inter-mediate Microcline Feldspar	%Albite Feldspar (Cleave - landite)
				Mean	**18.8**	**6.9**	**6.6**	**30.5**	**7.6**	**6.4**	**5.1**
37	1	1 - 3	LOW	YES	18	4.8	4.4	43.3	2.5	8	4.1
38	2	0 – 4.5	MEDIUM	YES	16.6	3.4	3.6	35.3	9.8	5.5	3.5
32	3	2 – 4.7	MEDIUM	YES	18.7	4.5	6.1	36.1	7	6.2	4.2
39	5	0.5 – 4	MEDIUM	YES	22.9	9.2	7.1	26.4	10.2	7	4.5
34	6[1]	1.5 – 4.2	MEDIUM	YES		11.4	8.2	40.7	6	9.2	11.5
35	7	1 – 4.2	MEDIUM	YES	19.3	6.5	8	26.1	11.7	4.3	5.6
40	8[2]	1 – 5	MEDIUM	YES	22.9	2	6.1	36.4	6.2	6.5	6.3
44	15	1 – 4	HIGH	YES	21.6	8.8	10.6	24.7	8.5	5.9	5.4
45	16	1 – 4	MEDIUM	YES	2.7	11.6	6.4	33.9	7.7	6.6	3.6
46	17	1.5 – 4.5	HIGH	YES	17.5	10.6	10.3	25.5	7.3	8	3.4
52	18	1 - 5	MEDIUM	YES		10.7		21.6	14.6	2.9	4.2
51	19	1 - 4	HIGH	YES	24.4	9.9	9.7	23.7	7.8	5.8	7.8
47	20	2 - 4	HIGH	YES	28.9		5.3	32	6.6	7	4.5
50	21	1 - 4	MEDIUM	YES	16.3	8	5.6	25.7	10.7	5	3.8
60	22	1 – 4	MEDIUM	YES	23.7	6.6	6.9	30.5	4.5	7	4.5
59	23	2.5 – 4.5	MEDIUM	YES	23.9		5.1	37.3	7.1	7.9	5.7
61	24	1 – 4.9	HIGH	YES	30.1	4.9	9.9	27	5.8	7.2	5.8
62	25	1 – 3.2	HIGH	YES	27.6	10.8	8.2	25.7	3.6	4.5	3.6
56	26	0 – 3.5	MEDIUM	YES	21.9	7.9	4.3	28.2	7.7	7.2	5.4
			Standard Deviation		**8.8**	**3.0**	**2.2**	**6.2**	**2.9**	**1.5**	**1.9**
			Median		**21.6**	**7.9**	**6.4**	**28.2**	**7.3**	**6.6**	**4.5**

[1] No significant damage observed at Site 6
[2] No significant damage observed at Site 8

Table 3. 2 - Laboratory Test Results for Stiff Foundation Soil Samples (no till)

1		**2**	**3**	**4**	**5**	**6**	**7**	**7**	**8**	**9**	**10**
UB#	**SITE**	**SAMPLE DEPTH (FEET)**	**POTENTIAL EXPANSION (ASTM D4829)**	**EXPANSION INDEX (ASTM D4829)**	**% 1Md Illite**	**% 2M1 Illite**	**%Chlorite Tusc.**	**% Quartz**	**% Calcite**	**% Inter-mediate Microcline Feldspar**	**% Albite Feldspar (Cleave - landite)**
30	4	6 – 9	HIGH	93	21.8	9.8	10.2	19.9	14.1	4	5.1
26	8	6 – 7.5	HIGH	94	18.5	9.9	4.5	19.9	16.6	6.4	7.5
25	9	≈ 7	MEDIUM	72	17.2	9.8	8.9	19.8	17.6	5.9	7.8
28	10	≈ 7	MEDIUM	52		18.8	10.5	26.3	9.1	6.9	9.4
33	11	≈ 6	MEDIUM	67	12.7	13	9.4	21	5.4	6	5.7
41	12	≈ 6	MEDIUM	82	15.2	8.8	10.9	19.1	24.6	5.1	4.7
43	14	≈ 7	MEDIUM	81		21.1	11	24.4	7.6	8	10
53	18	5.5 – 7.5	MEDIUM	78	16.9	10.4	7.4	17.1	16.5	5.6	3.9
48	20	7 – 10	HIGH	122	21.3	14.8	12.6	20.1	9	5.1	3.8
55 57	28	≈ 1.5	HIGH	118	27.7 29.3	5.9 9.2	8.8 7	19.5 20.3	15.7 13.2	5.8 7.2	6 7
.		.		> 20							
				Mean	**20.1**	**12.0**	**9.2**	**20.7**	**13.6**	**6.0**	**6.4**
				Standard Deviation	**5.6**	**4.6**	**2.3**	**2.5**	**5.5**	**1.2**	**2.1**
				Median	**18.5**	**9.8**	**9.4**	**19.9**	**14.1**	**5.9**	**6.0**

Table 3. 4 - Laboratory Test Results for Upper Portion of Soft Foundation Soils

1		2	3	4	5	6	7	8	9	10	11
UB#	**SITE**	**SAMPLE DEPTH (FEET)**	**POTENTIAL EXPANSION (ASTM D4829)**	**EXPANSIVE? (Residential Code of NYS, Section R403.1.8.1)**	**%1Md Illite**	**%2M1 Illite**	**% Chlorite Tusc.**	**% Quartz**	**% Calcite**	**% Inter-mediate Microcline Feldspar**	**% Albite Feldspar (Cleave - landite)**
31	4	9 - 13	HIGH	YES		12	7.3	18.7	8.8	3.4	
29	5	8 - 11	HIGH	YES	14	13.4	11.3	21	10.4	4.3	4.2
27	8[3]	9 – 12	HIGH	YES	13.1	18.3	13	17.5	8.6	9.7	5.2
53	18	7 – 8.5	HIGH	YES	16.9	10.4	7.4	17.1	16.5	5.6	3.9
49	20	10 – 11.5	HIGH	YES	20.9	15.7	10.2	15.9	9.3	4.6	5.1

	5	6	7	8	9	10	11
Mean	**12.9**	**13.9**	**9.8**	**18.0**	**10.7**	**5.5**	**3.7**
Standard Deviation	**3.5**	**3.1**	**2.5**	**1.9**	**3.3**	**2.5**	**0.6**
Median	**14.0**	**13.4**	**10.2**	**17.5**	**9.3**	**4.6**	**4.2**

[3] No significant damage observed at Site 8

Radar Interferometry Task: Amherst, NY

Prepared by

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and

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Date

January 31, 2005

1. Project and Assignment

Areas in Amherst, NY have experienced differential soil settling over the past decade. This has caused damage to property throughout the town, including damaged house foundations. In order to address concerns related to this problem, a study was conducted to examine and better identify the areas significantly affected by these changes.

The Earth Sciences Remote Sensing Lab was tasked with applying space based radar interferometry techniques to infer to what extent radar interferometry techniques could be used to delineate areas affected by this phenomenon and to investigate how one can monitor the changes in surface elevation through time in the Amherst, NY area.

2. Radar Interferometry

Radar interferometry is a technique which uses multiple radar images to infer topography, and subtle topographic changes. With the appropriate conditions, it is possible to use variations of the technique to measure changes in topography of smaller than 0.1mm/yr, up to several cm /yr [*Massonnet and Feigl*, 1998]. This technique has been used to map deformation and fault slip from earthquakes [*Sandwell et al.*, 2002], mine subsidence[*Carnec and Delacourt*], aquifer compaction from pumping [*Burbey*], and landslides [*Amelung and Day*, 2002], as well as seasonal changes due to groundwater [*Hoffmann et al.*]. The ideal place to apply these techniques is arid areas, where vegetation and atmosphere have little effect. However, newer refinements to radar interferometry allow it to be applied successfully over a wider range of conditions.

The Synthetic Aperture Radar (SAR) Interferometry technique exploits the information contained in the phase of 2 images or more that were acquired over the same location; it makes use of the difference in phase (interferometric phase) between two radar scenes to determine exact differences in range from the satellite, and subsequently to determine the precise x, y, and z location of the reflector, enabling the extraction of topography or subtle changes in topography.

The following Fig. (Fig. 1) shows the basic configuration of a pair of images used in repeat pass interferometry. ρ is the range to a target from the satellite reference position, $\rho + \delta\rho$ is the range to that same target acquired in the second pass. B is the baseline, or physical distance between the location of the satellite in the first and second pass. θ is the look angle, and α is the angle between the baseline vector and the tangent plane. It is then possible to define $\delta\rho$ as a function of B, θ, α, ρ, and λ, the wavelength of the radar beam. $\delta\rho$ is proportional to the phase difference component of the radar return ϕ, measured at the two radar platforms: $\phi=(4\pi/\lambda)\delta\rho$. The common terminology for the reference scene is the master scene, and the repeat scene is the slave scene.

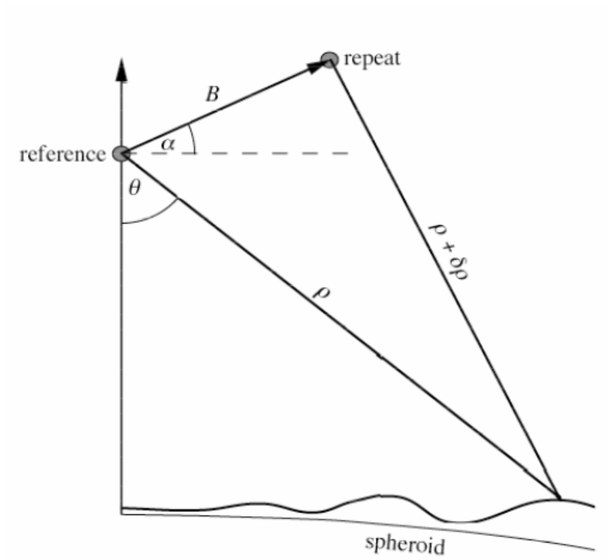

Figure 1: Geometry of Repeat Pass Interferometry [*Sandwell and Price*, 1998]

The factors which contribute to phase differences between two radar scenes include topography, deformation, and atmospheric effects. $\phi = \phi_{Topo} + \phi_{def} + \phi_{atm} + \phi_{noise}$.
The phase ϕ is recorded cyclically from $-\pi<\phi<\pi$, so there is by default an ambiguity in determining ρ from ϕ.

This process is described extensively by Gabriel and Massonnet elsewhere[*Gabriel et al.*, 1989] [*Massonnet and Feigl*, 1998] .

There are 3 basic families of the Radar interferometry techniques currently in use and under development. These are the basic 2-4 pass differential INSAR (DINSAR) techniques, as well as two classes of multi-temporal techniques which use numbers of scenes ranging from tens to hundreds. The multi-temporal techniques are expansions and refinements of the basic 2-4 pass techniques. They repeat many of the same steps, and then extract usable information from results which are ambiguous in the 2-4 pass techniques.

The basis for all of these techniques is the generation of an interferogram. To generate an interferogram, the two scenes need to be co-registered in radar-space. This means that the slave scene (or a subset thereof) has to be co-registered to the master scene (or a subset of the master scene). This is done in DORIS using the orbits to provide an initial estimate of the registration, and then the images are iteratively correlated using the cross correlation amplitude of the radar signal in individual subsets of the radar images. The radar images can be filtered (optional) to improve the registration. The slave image is then re-sampled to the master image. The interferogram is then calculated from the co-registered images as the dot-product of the complex images. This step is repeated for every interferogram that is generated. Any interferogram generated like this will have a phase component related to the curvature of the earth's surface. Th curvature is then calculated and removed before any further processing is done.

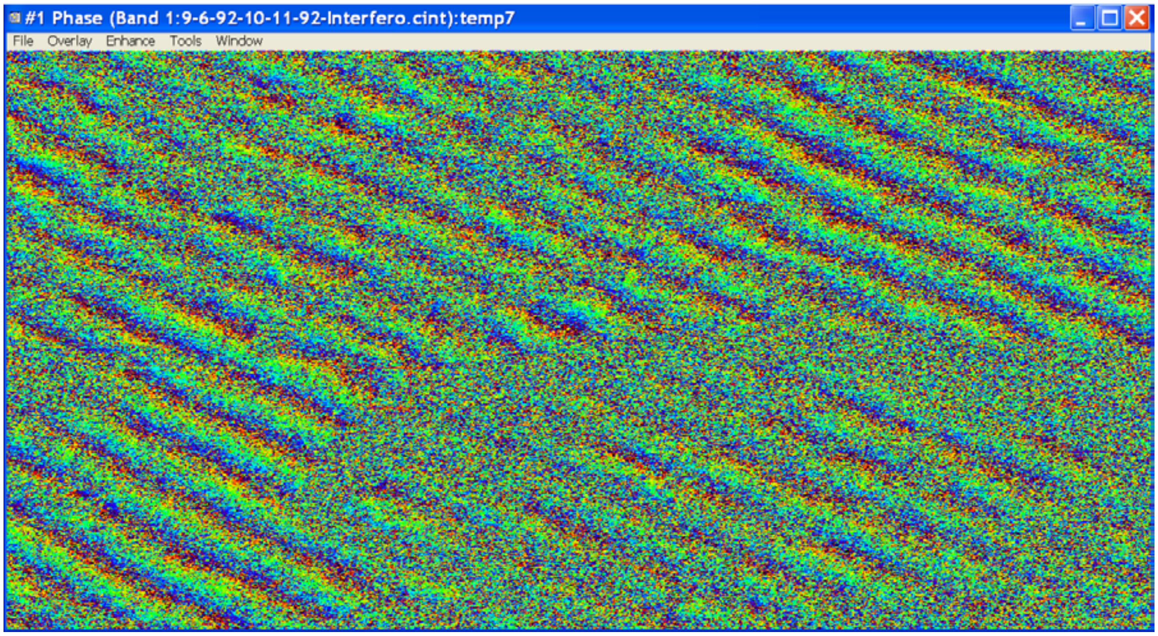

Figure 2: Sample interferogram generated from a pair of radr images acquired on 9-2-92 and 10-31-92.

The second necessary component of these techniques is phase unwrapping. In the above interferogram, repeating ripple pattern is evident; the ripples trend from the lower left to the upper right. This is caused by the phase ranging from $-\pi<\phi<\pi$, cyclically. In order for this to be turned into a measure of range, the cycles have to be added together, so that the phase numbers then extend from 0 to ~20π. instead of the original cyclical distribution ($-\pi<\phi<\pi$). This is called unwrapping, and is a major challenge in interferometry. The method we use (snaphu) is decribed in full in [*Chen and Zebker*, ; *Zebker and Lu*]. If correlation between scenes is low, or coherence in the interferogram is low, then this step becomes almost impossible.

In the 2 pass INSAR technique an interferogram is generated from 2 scenes, which span a deformation event (the master is acquired before, the slave after). ϕ_{atm} and ϕ_{noise} are assumed to be negligible. ϕ_{Topo} is calculated from a DEM which has been registered with the master scene, and subtracted from ϕ to yield ϕ_{def}. This method is fairly simple, but it relies heavily on the availability of a high quality DEM and excellent registration between the DEM and the master. Any error in the DEM or in registration will cause ambiguities in detecting and mapping deformation

In 3 pass interferometry, instead of a DEM being used, an unwrapped interferogram is used to remove the ϕ_{Topo} component. Noise and atmospheric contributions are again considered to be negligible. An interferogram from a co-registered master and slave with a very small temporal and spatial baseline (i.e. 1 day) is generated. An additional slave image on the other side of the deformation (also with a small spatial baseline) is co-registered to the same master, and an interferogram is generated. The interferogram from the first pair is unwrapped, scaled to match the second baseline, and re-wrapped. The second interferogram is subtracted from the first, removing the topographic phase, leaving ϕ_{def}.

In the 4 pass interferometry, a master-slave pair with very small temporal and spatial baselines is acquired before and after the deformation event. Each slave is co-registered to the appropriate master. An interferogram is generated for each pair. One interferogram is unwrapped and re-sampled to match the radar coordinates of the other pair. It is then scaled and re-wrapped. The second interferogram is subtracted from the first, yielding ϕ_{def}. This method has several distinct advantages over the 3-pass method. The ϕ_{noise} due to de-correlation is significantly reduced. All four scenes do not need to share the same small baseline range, but pairs can be selected to minimize spatial and temporal baselines. This significantly increases the detection of the resultant deformation.

The multi-temporal methods generate a far higher number of interferogram pairs, throughout a deformation event. By making a high number of interferograms, and making educated assumptions about the nature of the deformation (i.e. linear deformation) and of the atmospheric contributions, the errors associated with the solution can be minimized. In the SBAS approach, patches of coherent data are processed. In the point scatterer techniques, individual objects (single rooftops, etc) which are exceptionally good scatterers are used instead.

The selection of the family of techniques to be used depends on data quality, environment, deformation type, availability of scenes, and processing time. The two techniques which we have focused on in this exercise are 3-Pass DINSAR and the Small Baseline techniques. In addition to these techniques, there is also the Permanent/Persistent Scatterer family of techniques, which we have chosen not to use. The following summarizes the main characteristics of each of these techniques, as well as the advantages and disadvantages of each method:

- Basic 2, 3 or 4 Pass DINSAR
    - 2 to 4 Scenes required
    - Good Coherence between scenes mandatory
    - Atmospheric effects assumed negligible
    - 2-pass
        - Needs supplemental DEM information
        - Subject to inaccuracies in DEM
        - Pair needs to bracket deformation event
        - DEM needs to be accurately radarcoded
    - 3-pass
        - Subject to atmospheric effects
        - One pair needs to be very close together in time(1 day) on one side of the deformation event, the third scene has to be on the other side of the event
        - Difficult to maintain correlation over long times (years)
        - Scenes registered to common master

- 4-pass
  - 4 scenes, Pairs of scenes from before and after the deformation event
  - Each pair needs small baseline, good correlation
  - Allows for longer time for deformation to occur
  - Scenes co-registered as pairs, pairs co-registered to each other.
- Multi-temporal techniques:
  - Small Baseline Techniques (SBAS)
    - 10-20 scenes and more are used
    - Assumes areas of good coherence in interferogram
    - Entire scenes need not be coherent
    - Scenes spatially resampled to one common scene, directly or through cascading sequence [*Refice et al.*, 2003]
    - Only useful for gradual deformations (i.e subsidence)
    - Examples can be found in [*Lanari et al.*, 2004a; *Lanari et al.*, 2004b]
    - High number of interferograms generated
    - Processing time intensive
  - Permanent Scatterer Techniques
    - >40 scenes
    - Good coherence at individual points (Permanent/Persistent Scatterers)
    - Deformation can be gradual (subsidence) or sharp (faulting)
    - Non-linear estimate of deformation
    - Scenes spatially resampled to one common scene, directly or through cascading sequence
    - Very high number of interferograms generated
    - Processing time intensive
    - Examples include: [*Ferretti et al.*, 2000 2001; *Ferretti et al.*, 2001 2001]

3. Methodology

We have attempted to conduct the 2 pass method but the results were not satisfactory, primarily due to the inherent ambiguity that could result from difficulties in registration and the inaccuracies in the digital elevation. The next step was to apply the three and four pass methods. The results from the 3 pass method were also unsatisfactory because de-correlation over the long period (years) of deformation. Our best results which we report here are from the 4 pass method. Although not reported here, we have started to investigate the suitability of the data if we were to apply the SBAS method. Our goal is to improve on the 4-pass results.

3.1 Scene Selection

In the 2, 3 and 4 pass methods, the selection of scenes is critical. The selected scenes have to bracket the event of interest, a baseline distance needs to be maintained, and the pairs from which an interferogram is generated cannot be too far apart in time, or they become de-correlated. Additionally, we are limited by what scenes have been recorded, and when they were obtained. In selecting the optimal scenes, we established a set of criteria which are most suited for the application of the 3-pass interferometry technique. The selected scenes were acquired when the foliage on the trees was minimal, had a small perpendicular baseline, the acquisition time for the scenes would encompass the periods of soil subsidence presumably coinciding with dryer periods in Western New York. Our initial plan was to order 11 scenes, and to combine the scenes in a variety of ways, in an attempt to obtain good 3 pass interferometric solutions.

Processing of these scenes showed that de-correlation between scenes was too great given the long time period covered by the investigated scenes. AS a consequence there wasn't sufficient coherence across the entire image to use the 3 pass technique. Having said that, we did have good coherence in smaller areas, especially where there were strong reflectors, such as rooftops, and other man-made structures. Our next step was to consider the 4 step technique. The latter, by definition would eliminate the problems arising from the long-period de-correlation described above. Since the scenes we had ordered were targeted for the 3 pass technique, we did not have a suitable set to investigate the 4 pass technique.

We determined that although the scenes which we had were not suitable for conducting the 3-pass DINSAR, they were good candidates for either of the multi-temporal processing methods. Based on the nature of the coherent areas observed, and the available budget, we opted to examine the SBAS approach to processing the scenes. At the beginning of December, 2004, additional scenes were ordered. The selection of this second batch of scenes was based on a different set of criteria. We looked for pairs of scenes with the smallest baseline differences, scenes that were acquired in proximity (in time) to one another, and pairs of scenes that have minimal snow and foliage foliage. In some cases, we accepted pairs of scenes which did not have low baselines relative to the whole dataset, if the scenes of this particular pair were acquired one or two days apart. These additional scenes arrived at the beginning of January, 2005. The scenes and dates of acquisition are listed in Appendix A. Given the time constraints for delivery of results (report due February 2nd), absence of funding to acquire enough scenes (~40-50 scenes) for conducting multi-temporal methods (SBAS technique), we investigated the use of 4 pass technique. Between the first and second batch of scenes, we now had enough scenes to perform several 4-pass DINSAR deformation extractions. The results are discussed below.

3.2Processing Done:

All of the interferometric processing was done using the Delft object-oriented radar interferometric software (DORIS) [*Kampes et al.*, 2003]. The phase unwrapping was done using snaphu [*Chen and Zebker*, 2002]. Additional filtering and display was conducted in ENVI (from RSI Inc.). Processing was done on both Microsoft (using windows and cygwin) and linux based systems. Orbits were obtained using the program getorb, with orbits provided by Delft Institute for Earth Oriented Research [*Scharroo and Visser*, 1998]

For all of the scenes, an initial examination was conducted in ENVI, to check for scene quality, and to insure that the scenes could be read. The scenes were then processed in DORIS. (A sample input card for DORIS is found in Appendix B.). All scenes are processed in radar-image space (in this case slant-range), and re-projected to a map projection at the final stages. As a consequence, all of the images shown here are mirror image of the space-based image. Figures 5 and 6 are exceptions; they have been re-projected in UTM.

For each of the generated interferograms, the image headers were read into DORIS (for both master and slave). The images were subset to the area of interest for the study. Based on the header information, precise orbital information for each scene was obtained. The images were over-sampled by a factor of 2 in the range direction to improve the co-registration. A course correlation was obtained between the 2 scenes. The scenes were then azimuth filtered together to match the spectra of the scenes to oner another. A fine registration was then conducted. Based on this fine registration, the slave image was re-sampled to the master image. Both images were filtered in range. An interferogram was generated for the image pair. This image was then flat-earth corrected, to eliminate fringes created by the curvature of the earth (Fig. 2). Coherence was calculated for the images, and the images were then filtered in phase using the Goldstein filter. This phase filtered product was the final step of many of the interferograms. Examples of these products are shown in Fig. 3.

As seen in Fig. 3, not all image pairs produce good interferograms. Over one hundred interferograms were generated using the above mentioned steps. Each interferogram contains some information about topography and deformation. However that information decreases with increased spatial and temporal baselines. The best interferograms were generated between scene pairs, such as 11-28-95 and 11-29-95. These scenes were acquired very close to each other, both in space (small baseline difference) and time (1 day apart). The one exception to this is the pair that was acquired at 3-12-96 and at 3-13-96; the pair has good temporal and spatial baselines (1 day, 55m). In this case, the snow on the ground was problematic causing poor correlation. The snow cover was inferred from archival meteorological records. Even though the interferograms have a significant amount of de-correlation noise, they do still contain a significant amount of phase data as well.

Several interferograms were unwrapped, with the best results coming from the 11-28-95,11-29-95 pair. This interferogram pair was used together with the 9-6-92 and 10-11-92 pair to generate a 4-pass DINSAR product. Several other pairs with high coherence were also processed, but the aforementioned set of pairs proved to generate the best 4-pass DINSAR product.

This DINSAR product was then lightly filtered to remove data from areas of low coherence (Fig. 4). This process removes pixels for which the phase solution is likely to be wrong.

This product was re-projected to UTM to allow better visualization of the areas of change. The re-projected product and is shown again in Fig. 5.

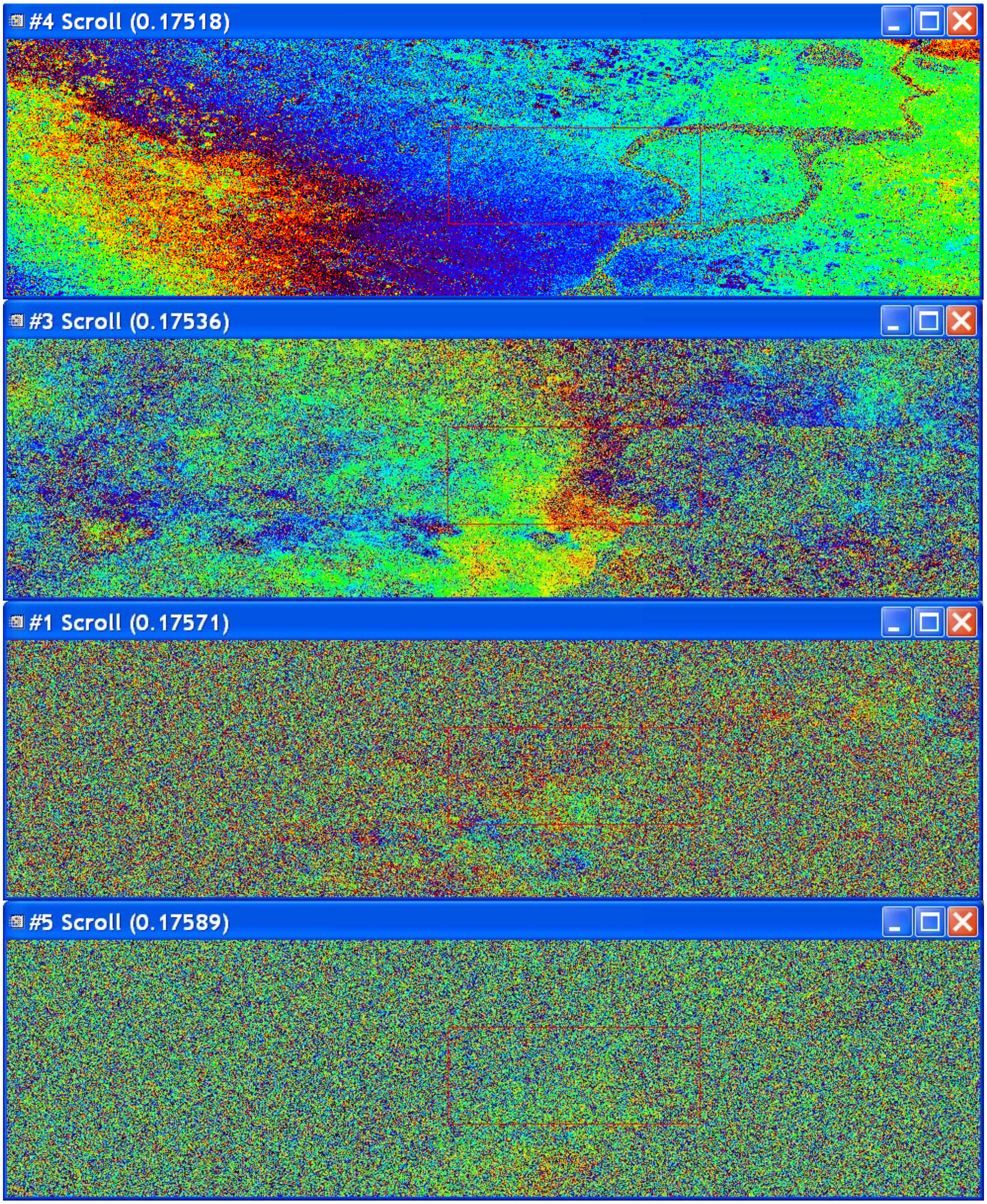

Figure: 3 Series of interferograms showing loss of coherence with increased temporal separation, even with good spatial basline. (From Top, Master:11-28-95 Slave:11-29-95, $B_{perp=}$43m; M: 10-11-92, S:9-6-92, $B_{perp=}$-17m; M:10-11-92, S:10-24-95 $B_{perp=}$273m; M: 10-11-92, 3-12-03, $B_{perp=}$54m) . Images are in slant-range radar space, Grand Island is on the right, and Buffalo and Amherst are on the left. The Niagara river and the power reservoirs are visible on the right, as is the Niagara Escarpment, at the top right.

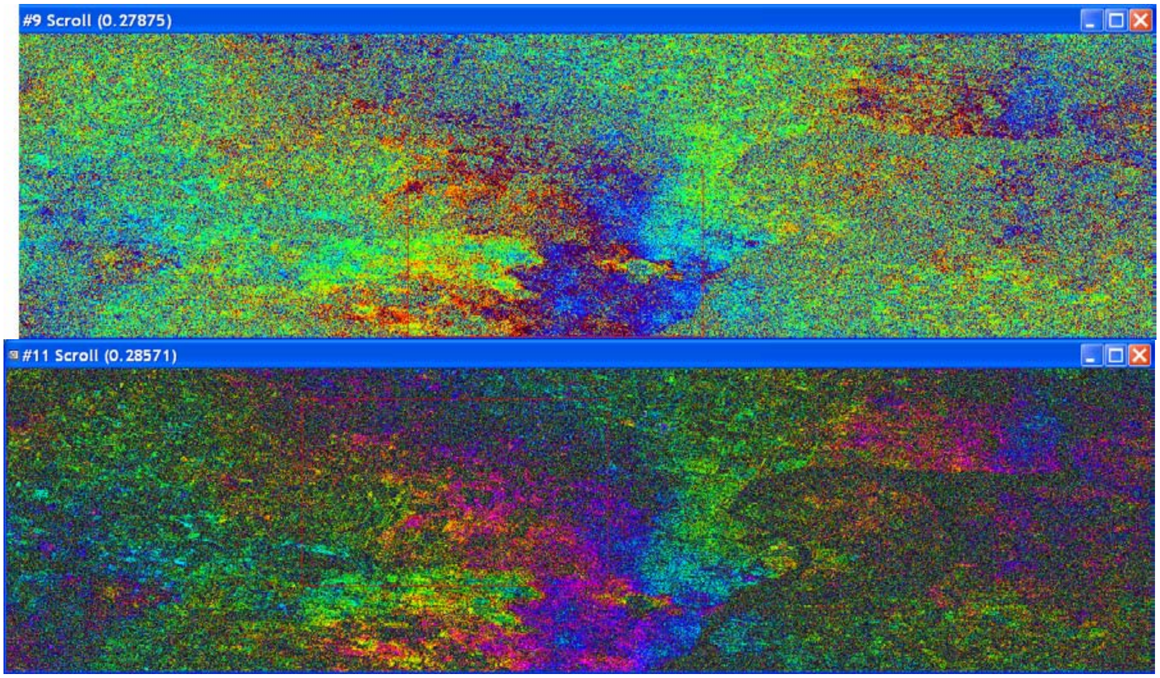

Figure 4: Results from the 4-Pass DINSAR, unfiltered (above), and filtered based on coherence (below)

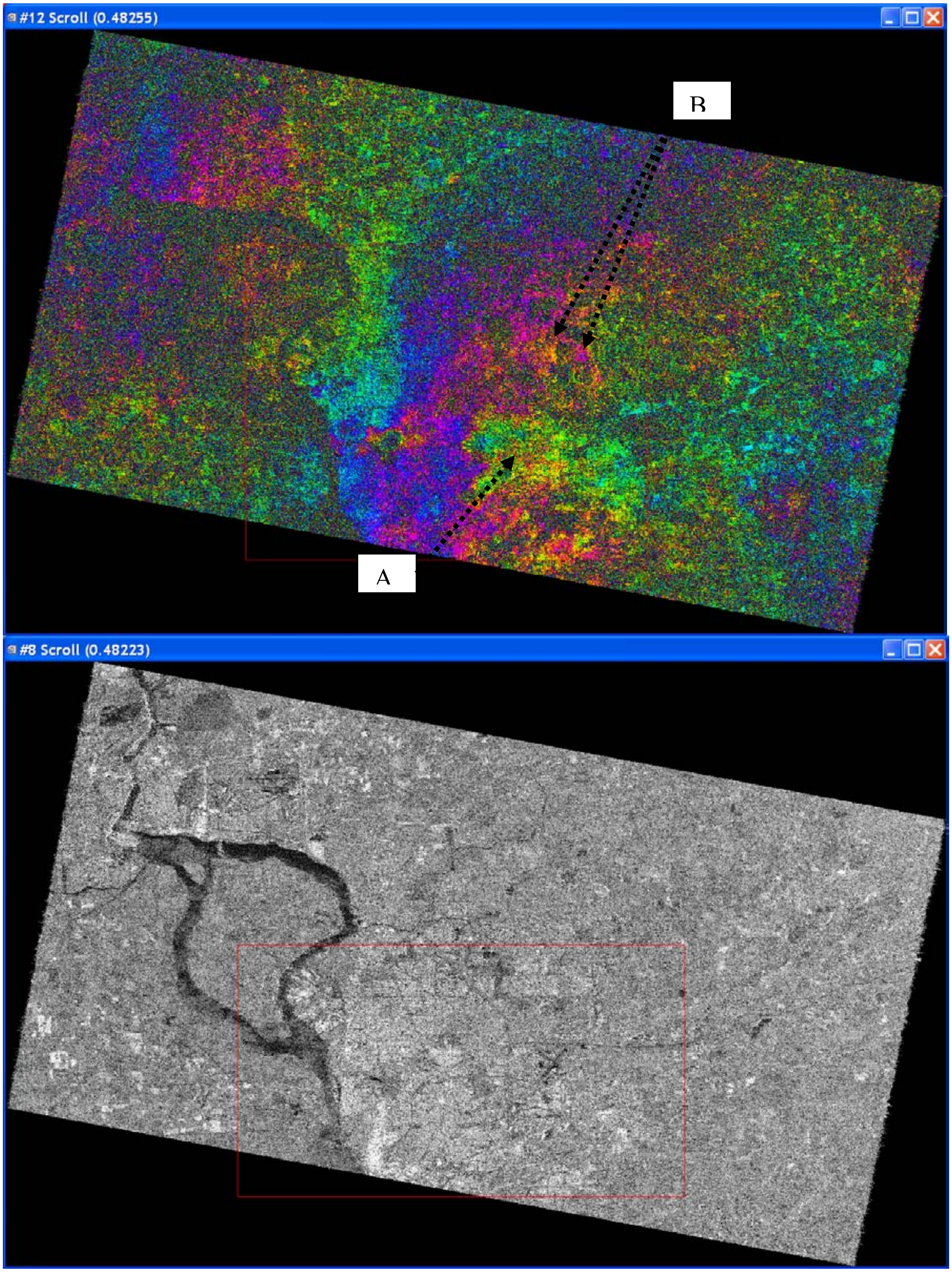

Figure 5: 4-Pass DINSAR result (phase), resampled to UTM (above) and reference image (below) showing areas of interest in DINSAR result.

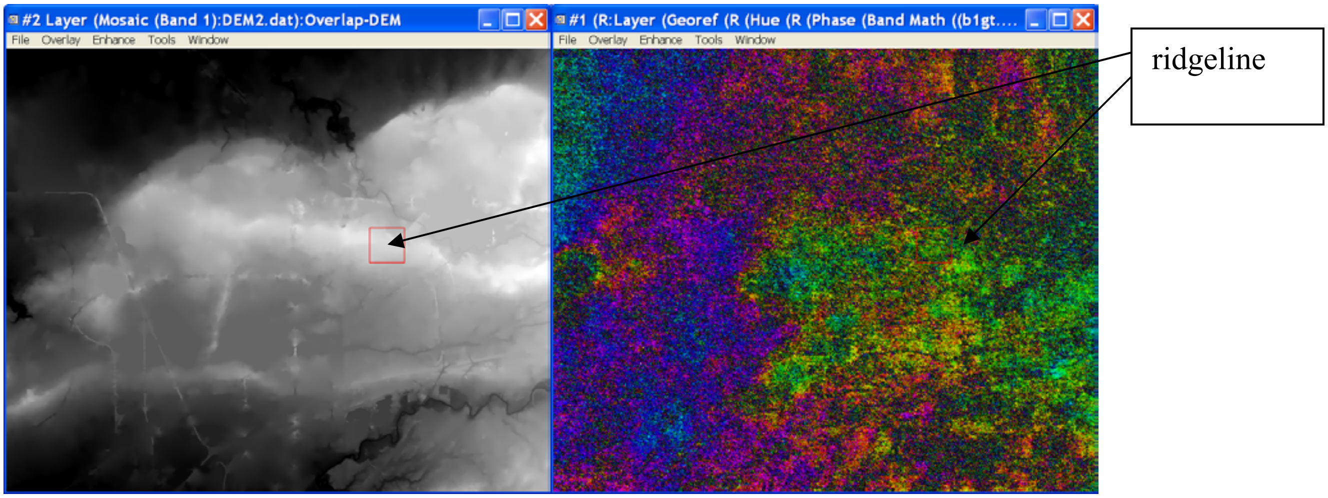

Figure 6: Comparison between DEM mode (left) and DINSAR (right) shows that ridgeline aligns in part with the interferometric feature.

4. Results:

In the course of processing, approximately 105 different interferograms have been generated from the acquired scenes. These were evaluated by visual inspection of the coherence images. We chose the two best interferograms spanning one of the dryer periods in Western New York (92-95). The results presented here reflect interpretation of the 4-pass DINSAR solution generated using these interferograms.

Preliminary results show that we are able to observe changes in the Amherst area.

The 4-pass DINSAR results (Fig. 5) show a coherence filtered, phase difference image over Buffalo and Amherst. There are three readily observed features to this scene.

The first is a left to right, long wavelength phase signal, related to residual topography. This feature is manifested as the gradual change through blue-purple-red-orange-yellow-green-cyan-blue.

The second is in area A on Fig. 5, which is a medium wavelength feature which is centered over the airport. This feature could be either related to a residual topographic signature. Supprt for this hypothesis is that it aligns well with a ridge line (Fig. 6). Alternatively, it could indicate a larger scale deformation. With the processing done to date, we are unable to differentiate between the two possible interpretations.

The final area of interest ("B" on Fig.s 5 and 7), is the areas in Amherst which go from purple to yellow to purple to yellow as the scene is viewed from left to right. These are unlikely to be topographic residuals, which appear as longer wavelength features. One obvious set of these features is found between Maple and Sheridan (Fig. 7). These features are most likely due to local differential surface deformation. At this point we cannot entirely rule out a subtle residual topographic effect as a possible cause. We do not see a correlation between the distribution of theses features and topographic expressions and thus we feel that topographic control is highly unlikely. Future plans will involve further verification of these features using the multi-temporal techniques.

The initial 4-pass DINSAR results which we show could be significantly refined and filled in using multi-temporal techniques. This processing will involve the incorporation of additional scenes, and a significant amount of man-power and processing time. Given the available resources, we were not able to complete this type of processing. However, given the nature of the datasets, and the results obtained so far, we believe the multi-temporal techniques will be an optimum approach.

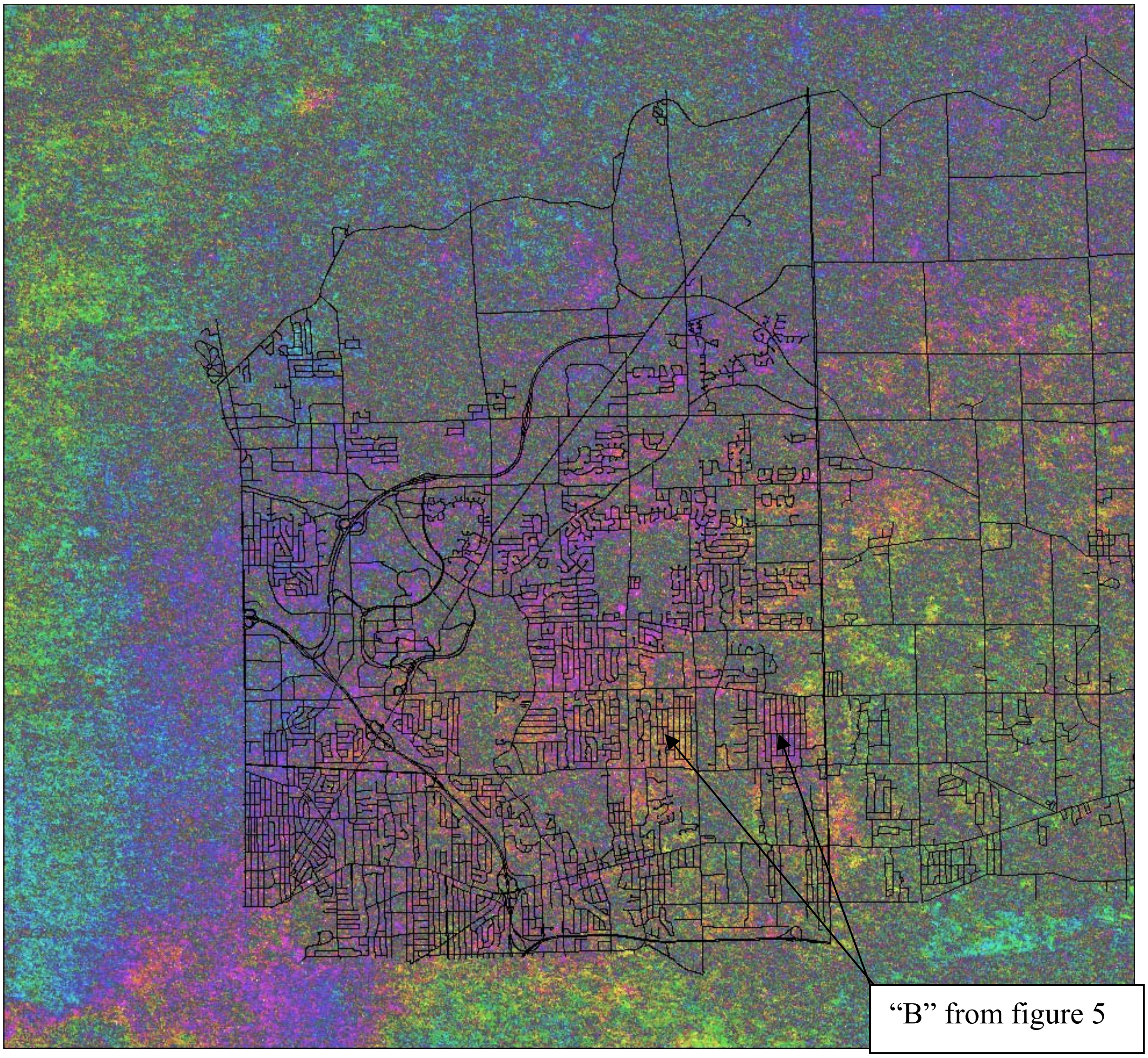

Fig. 7. Four-pass DINSAR deformation result over Amherst, NY, with street map overlain for reference

5. Recommendations:

Initial results of the interferometry processing are very promising. There is still a large amount of information which can be extracted from the ERS SAR images. What is shown in Fig. 7 can be significantly refined to remove several of the sources of error, and potentially be expanded into areas which in this image have poorer correlation. The longer wavelength residual topographic signal can then also be removed.
To accomplish this, we need to continue processing the scenes using the SBAS technique. We also recommend purchasing additional scenes (up to 20) if we proceed with the SBAS technique. If we were to choose to process the data using the PS technique, we would need between 30 and 40 more scenes for the best solution. This would allow the best refinement of the definition of subsidence areas, and the removal of errors.

6. References:

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Appendix A: Scenes Acquired:

Date	Satellite	Orbit	Track/Frame
9-6-92	ERS-1	05982	326/2745
10-11-92	ERS-1	06483	326/2745
9-26-93	ERS-1	11493	326/2745
10-31-93	ERS-1	11994	326/2745
8-15-95	ERS-1	21356	326/2745
10-24-95	ERS-1	22358	326/2745
11-28-95	ERS-1	22859	326/2745
11-29-95	ERS-2	03186	326/2745
3-12-96	ERS-1	24362	326/2745
3-13-96	ERS-2	04689	326/2745
9-4-96	ERS-2	07194	326/2745
4-2-97	ERS-2	10200	326/2745
8-20-97	ERS-2	12204	326/2745
9-24-97	ERS-2	12705	326/2745
10-14-98	ERS-2	18216	326/2745
11-18-98	ERS-2	18717	326/2745
9-29-99	ERS-2	23226	326/2745
3-12-2003	ERS-2	41262	326/2745

Appendix B. Expenses:

A total of 18 scenes were ordered from Radarsat Canada.

Radarsat Scenes Ordered:	
Order # 1 (11 Scenes)	$ 3830
Order # 2 (7 Scenes)	$ 2520
Data Total	$ 6380
University of Buffalo F+A Costs (56%)	$3573
Total Expended	$ 9953

In addition, Western Michigan has contributed personnel and computer processing on this project.
This has included approximately 8 weeks of staff time over the course of this project.
Approximately 200 hours of processing time was performed on Western Michigan computers.
This represents an in-kind contribution of $22,000 of staff time (including appropriate fringe and F&A rates).

Appendix C. Sample input card for DORIS processing

```
SCREEN    INFO
BEEP WARNING
MEMORY    1024
OVERWRITE ON
BATCH ON
LISTINPUT ON
ORB_INTERP      POLYFIT

PROCESS M_READFILES
M_IN_METHOD    ERS
M_IN_NULL /cygdrive/d/INSAR/data/11-28-95/NUL_DAT.001
M_IN_VOL  /cygdrive/d/INSAR/data/11-28-95/VDF_DAT.001
M_IN_LEA  /cygdrive/d/INSAR/data/11-28-95/LEA_01.001
M_IN_DAT  /cygdrive/d/INSAR/data/11-28-95/DAT_01.001

PROCESS M_PORBITS
m_orbdir  /cygdrive/d/INSAR/ORBITS/ers-1
m_orb_interval 1
m_orb_extratime 5

PROCESS M_CROP
m_CROP_IN /cygdrive/d/INSAR/data/11-28-95//DAT_01.001
m_CROP_OUT /cygdrive/d/INSAR/Processed/11-28-95.raw
m_DBOW_GEO 43.0 -78.8 7400 2800

PROCESS S_READFILES
S_IN_METHOD    ERS
S_IN_NULL /cygdrive/d/INSAR/data/11-29-95/NUL_DAT.001
S_IN_VOL  /cygdrive/d/INSAR/data/11-29-95/VDF_DAT.001
S_IN_LEA  /cygdrive/d/INSAR/data/11-29-95/LEA_01.001
S_IN_DAT  /cygdrive/d/INSAR/data/11-29-95/DAT_01.001

PROCESS S_PORBITS
S_orbdir  /cygdrive/d/INSAR/ORBITS/ers-2
S_orb_interval 1
S_orb_extratime 5


PROCESS S_CROP
S_CROP_IN /cygdrive/d/INSAR/data/11-29-95/DAT_01.001
S_CROP_OUT /cygdrive/d/INSAR/Processed/11-29-95.raw
S_DBOW_GEO  43.0 -78.8 7400 2800

PROCESS S_OVS
PROCESS M_OVS
```

```
M_OVS_FACT_RNG 2
S_OVS_FACT_RNG 2


M_OVS_OUT /cygdrive/d/INSAR/Processed/11-28-95_ovs-M
S_OVS_OUT /cygdrive/d/INSAR/Processed/11-29-95_ovs


PROCESS COARSEORB

PROCESS COARSECORR
cc_winsize 256 256
cc_initoff orbit
cc_nwin 21

PROCESS M_FILTAZI
PROCESS S_FILTAZI
AF_BLOCKSIZE 2048
c AF_OUT_MASTER /cygdrive/d/INSAR/Processed/m_azi
AF_OUT_SLAVE /cygdrive/d/INSAR/Processed/11-29-95_azi
AF_HAMMING .75

PROCESS FINE
FC_INITOFF coarsecorr
FC_NWIN 400
FC_WINSIZE 128 64
FC_ACC 12 12
fc_plot 0.35 BG

PROCESS COREGPM
cpm_plot bg
CPM_THRESHOLD 0.3
CPM_WEIGHT quadratic
CPM_DEGREE 2
CPM_MAXITER 20


PROCESS RESAMPLE
c RS_METHOD RECT
RS_METHOD       knab6p
RS_OUT_FILE /cygdrive/d/INSAR/Processed/11-29-
95resampled.raw
RS_DBOW 1739 9138 2033 7632
RS_OUT_FORMAT CR4

PROCESS FILTRANGE
RF_OUT_MASTER /cygdrive/d/INSAR/Processed/m_range1
```

```
RF_OUT_SLAVE /cygdrive/d/INSAR/Processed/s_range1
RF_FFTLENGTH    128                     // 2500 m
RF_OVERLAP      32                      //
RF_NLMEAN       15                      // odd, 60 m
RF_THRESHOLD    5                       // SNR
RF_HAMMING      0.75                    // alpha
RF_OVERSAMPLE   2
RF_WEIGHTCORR   OFF

PROCESS INTERFERO
INT_OUT_CINT /cygdrive/d/INSAR/Processed/11-29-95-11-28-95-
Interfero.cint


PROCESS COMPREFPHA

PROCESS SUBTRREFPHA
SRP_METHOD EXACT
SRP_OUT_CINT   /cygdrive/d/INSAR/Processed/11-29-95-11-28-
95-Flat-Corr.cint


PROCESS COHERENCE
COH_OUT_CCOH /cygdrive/d/INSAR/Processed/11-29-95-11-28-95-
complexcoherence
COH_OUT_COH /cygdrive/d/INSAR/Processed/11-29-95-11-28-95-
coherence
COH_MULTILOOK  5 1

c PROCESS COMPREFDEM
CRD_IN_FORMAT R4
CRD_IN_DEM  /cygdrive/d/INSAR/Processed/DEM2
CRD_IN_SIZE 6707 9394
CRD_IN_DELTA 0.0009259 0.0009259
CRD_IN_UL 43.275 -79.068
CRD_OUT_DEM /cygdrive/d/INSAR/Processed/DEM-RESAMP
CRD_OUT_FILE RADARCODEDDEM

c PROCESS  SUBTRREFDEM
c SRD_OUT_CINT /cygdrive/d/INSAR/Processed/SUBTRDEM

PROCESS FILTPHASE
PF_METHOD    goldstein
PF_ALPHA     0.5
PF_OVERLAP   4
PF_BLOCKSIZE 32
PF_KERNEL    5 1 1 1 1 1
```

```
PF_OUT_FILE /cygdrive/d/INSAR/Processed/11-29-95-11-28-95-
FiltPhase

PROCESS UNWRAP
UW_OUT_FORMAT  hgt
UW_SNAPHU_MODE TOPO
UW_SNAPHU_COH  /cygdrive/d/INSAR/Processed/11-29-95-
coherence
UW_SNAPHU_LOG snaphu-Oct92.log
UW_OUT_FILE /cygdrive/d/INSAR/Processed/11-29-95-11-28-95-
unwrapped-interfero
c UW_SNAPHU_LOG     snaphu.log
UW_SNAPHU_INIT    MST

PROCESS SLANT2H


LOGFILE   11-29-95a.out
M_RESFILE 11-28-95c-Slave.out
S_RESFILE 11-29-95a-Slave.out
I_RESFILE 11-29-95to11-28-95Interferogram.out

STOP
```

6.3 TYPICAL OHIO WATER BUDGET

APPENDIX 6.3. TYPICAL OHIO WATER BUDGET

(Source: Ohio State University Fact Sheet, Water Res. of Erie County, AEX-480.22-98)

Based on long-term statewide weather records, Ohio receives an average of 38 inches of precipitation. These values would approximate much of Western New York.

38 inches	= total precipitation (rain & snow)
- 10 inches (26%)[1]	= direct runoff[2]
- 2 inches (5%)	= evapotranspiration (short-term)
26 inches (68%)	= infiltrate
- 20 inches(53%)	= evapotranspiration (long-term)
6 inches (16%)	= recharges groundwater
- 2 inches (5%)	= discharge into lake, streams, springs
- 4 inches (11%)	= discharged as drinking water (wells) or evapotranspiration

[1] All percentages are based on total precipitation and do not sum to 100%; [2] Some watersheds have runoff 30 to 50%.

6.4 TONAWANDA LANDFILL DATA

Tonawanda landfill data was provided by Glen May at the New York State Department of Environmental Conservation, Buffalo, NY (716-851-7200).

LEGEND:

MONITORING WELL LOCATION

SHALLOW ZONE GROUNDWATER CONTOUR MAP – SEPTEMBER 1997

DIVISION OF ENVIRONMENTAL REMEDIATION

DATE: 07/22/03	DRAWING: GW Study 2.dwg
SITE NAME: TONAWANDA GROUNDWATER STUDY	FIGURE 3

				Niagara					
Date	BM-8	BM-12	BM-15	River					
01/04/95	595.16	595.17	589.55						
02/06/95	595.23	595.50	590.82						
05/15/95	594.80	595.43	594.02						
12/20/95	594.29	591.94							
03/25/96	594.36	597.73	594.62						
06/12/96	594.61	594.98	593.04						
09/16/96	593.80	594.00	591.33						
12/19/96	595.17	594.77	592.94						

Table 4. Groundwater Elevations from Shallow Zone Wells at the Spaulding Composites Site											
Date	OW-1	OW-2	OW-3	OW-4	OW-6	OW-7	OW-8	OW-9	OW-10	OW-11	OW-12
12/27/95	600.60	599.74	598.38	598.97	594.89	592.94	588.45	586.19	590.55	593.56	606.37
03/25/96	601.67	600.76	599.62	599.88	595.75	593.30	588.73	586.77	591.78	594.22	606.64
07/22/96	598.22	597.53	597.18	597.91	594.67	591.97	586.46	585.67	587.31	594.76	603.63
11/21/96	600.94	600.17	598.98	599.45	594.74	592.17	588.59	586.63	592.22	594.63	606.79
09/22/97	597.50	597.32	596.90	597.74	593.70	592.17	588.34	584.89	586.19	592.84	601.22
12/18/98	593.56	595.84	596.20	595.22	593.51	591.96	587.40	582.89	582.91	592.29	596.54

Table 5. Groundwater Elevations from Intermediate, Deep and Upper Bedrock Zone Wells at the Spaulding Composites Site							
Date	OW-A1	OW-B2	OBW-2	BW-9	BW-10	BW-12	BW-C3
12/27/95	600.01	595.72	570.28	569.36	570.30	570.26	570.29
03/25/96	601.58	597.31	569.70	568.74	569.72	569.64	569.69
07/22/96	598.33	595.90	569.31	568.62	569.29	569.24	569.28
11/21/96	600.69	596.31	570.06	569.18	569.68	570.06	570.07
09/22/97		596.86	569.25	568.52	569.21	569.20	569.19
12/18/98	596.70	595.72	569.22	568.34	569.20	569.19	569.62

**Table B-1.
Monitoring Well Instrumentation Summary for Shallow Zone Wells Installed in the Study Area.**

Well Designation	Ground Surface Elevation (ft. AMSL)	Top of Riser Elevation (ft. AMSL)	Total Boring Depth (ft. BGS)	Sandpack Interval (ft. BGS)	Sandpack Interval (ft. AMSL)	Well Screen Interval (ft. BGS)	Well Screen Interval (ft. AMSL)	Screened Unit
Polymer Applications Site (Registry Number 915044)								
B-4S	592.46	594.26	6.0	2.50 to 6.00	589.96 to 586.46	4.00 to 6.00	588.46 to 586.46	Miscellaneous Fill
B-5S	589.23	591.14	5.0	3.00 to 5.00	586.23 to 584.23	2.50 to 5.00	586.73 to 584.23	Miscellaneous Fill; Reddish Brown Silty Clay
B-6S	579.84	582.13	5.0	3.00 to 5.00	576.84 to 574.84	3.50 to 5.00	576.34 to 574.84	Miscellaneous Fill
B-7S	578.26	578.12	6.0	3.00 to 6.00	575.26 to 572.26	4.50 to 6.00	573.76 to 572.26	Miscellaneous Fill
GW-3S	591.69	594.53	10.0	1.50 to 10.00	590.19 to 581.69	2.00 to 10.00	589.69 to 581.69	Miscellaneous Fill; Reddish Brown Silty Clay
GW-4S	589.43	592.40	10.0	1.50 to 10.00	587.93 to 579.43	2.00 to 10.00	587.43 to 579.43	Miscellaneous Fill; Reddish Brown Silty Clay
MW-9S	592.11	593.82	10.2	3.50 to 10.00	588.61 to 582.11	4.00 to 10.00	588.11 to 582.11	Reddish Brown Silty Clay
MW-11S	577.27	579.22	14.2	3.00 to 14.00	574.27 to 563.27	4.00 to 14.00	573.27 to 563.27	Reddish Brown Silty Clay
MW-12S	578.91	580.77	12.0	3.00 to 12.00	575.91 to 566.91	4.00 to 12.00	574.91 to 566.91	Brown/Black Silt & Fine Sand; Reddish Brown Silty Clay
MW-13S	575.54	577.58	10.0	3.50 to 10.00	572.04 to 565.54	4.00 to 10.00	571.54 to 565.54	Miscellaneous Fill; Reddish Brown Silty Clay
MW-14S	575.68	577.99	12.0	3.00 to 12.00	572.68 to 563.68	4.00 to 12.00	571.68 to 563.68	Miscellaneous Fill; Reddish Brown Silty Clay
Dunlop Tire Corporation Site (Registry Number 915018)								
OMW-B3	577.85	580.58	16.0	6.00 to 15.00	571.85 to 562.85	9.50 to 14.50	568.35 to 563.35	Peat; Reddish Brown Silty
OMW-C1	601.04	603.84	18.0	5.00 to 17.50	596.04 to 583.54	7.00 to 17.00	594.04 to 584.04	Reddish Brown Silty Clay
E.I. DuPont Yerkes Plant Site (Registry Number 915019)								
MW-1S	600.88	602.74	5.5	?? to ??	?? to ??	3.50 to 5.50	597.38 to 595.38	Grey Organic Silt & Clay
MW-2S	600.33	602.85	5.5	?? to ??	?? to ??	2.50 to 4.50	597.83 to 595.83	Reddish Brown Silty Clay
MW-3S	603.10	604.88	7.5	?? to ??	?? to ??	5.50 to 7.50	597.60 to 595.60	Miscellaneous Fill

Table B-1 (continued).
Monitoring Well Instrumentation Summary for Shallow Zone Wells Installed in the Study Area.

Well Designation	Ground Surface Elevation (ft. AMSL)	Top of Riser Elevation (ft. AMSL)	Total Boring Depth (ft. BGS)	Sandpack Interval (ft. BGS)	Sandpack Interval (ft. AMSL)	Well Screen Interval (ft. BGS)	Well Screen Interval (ft. AMSL)	Screened Unit
E.I. DuPont Yerkes Plant Site (continued)								
MW-4S	602.17	604.26	5.0	?? to ??	?? to ??	3.00 to 5.00	599.17 to 597.17	Miscellaneous Fill; Reddish
DYF-1	589.43	592.24	14.0	3.00 to 14.00	586.43 to 575.43	4.00 to 14.00	585.43 to 575.43	Reddish Brown Silty Clay
DYF-2	587.83	591.35	14.0	3.00 to 14.00	584.83 to 573.83	4.00 to 14.00	578.83 to 573.83	Reddish Brown Silty Clay
DYF-3	592.20	595.27	14.0	3.00 to 14.00	589.20 to 578.20	4.00 to 14.00	588.20 to 578.20	Reddish Brown Silty Clay
DYF-4	597.29	600.12	14.0	3.00 to 14.00	594.29 to 583.29	4.00 to 14.00	593.29 to 583.29	Reddish Brown Silty Clay
DYF-5	602.36	605.24	14.0	3.00 to 14.00	599.36 to 588.36	4.00 to 14.00	598.36 to 588.36	Reddish Brown Silty Clay

Ft. AMSL Feet Above Mean Sea Level. Ft. BGS Feet Below Ground Surface.

Table B-2.
Monitoring Well Instrumentation Summary for Intermediate Zone Wells Installed in the Study Area.

Well Designation	Ground Surface Elevation (ft. AMSL)	Top of Riser Elevation (ft. AMSL)	Total Boring Depth (ft. BGS)	Sandpack Interval (ft. BGS)	Sandpack Interval (ft. AMSL)	Well Screen Interval (ft. BGS)	Well Screen Interval (ft. AMSL)	Screened Unit
Polymer Applications Site (Registry Number 915044)								
B-2D	581.44	583.71	23.7	12.40 to 23.70	569.04 to 557.74	13.70 to 23.70	567.74 to 557.74	Reddish Brown Silty Clay
B-3D	589.21	591.14	21.0	10.00 to 21.00	579.21 to 568.21	11.00 to 21.00	578.21 to 568.21	Reddish Brown Silty Clay
B-4D	591.93	594.13	20.0	9.00 to 20.00	582.93 to 571.93	10.00 to 20.00	581.93 to 571.93	Reddish Brown Silty Clay
B-5D	589.16	591.24	20.0	9.00 to 20.00	580.16 to 569.16	10.00 to 20.00	579.16 to 569.16	Reddish Brown Silty Clay
B-6D	578.66	580.89	20.0	9.00 to 20.00	569.66 to 558.66	10.00 to 20.00	568.66 to 558.66	Reddish Brown Silty Clay
B-7D	578.42	578.15	20.0	9.00 to 20.00	569.42 to 558.42	10.00 to 20.00	568.42 to 558.42	Reddish Brown Silty Clay
Dunlop Tire Corporation Site (Registry Number 915018)								
OMW-A4	582.0*	584.18	24.0	5.50 to 24.00	576.50 to 558.00	13.00 to 23.00	569.00 to 559.00	Reddish Brown Silty Clay
OMW-A6	594.28	593.74	24.5	11.00 to 24.50	583.28 to 569.78	13.50 to 23.50	580.78 to 570.78	Reddish Brown Silty Clay
OMW-B4	586.0*	587.73	22.0	9.00 to 22.50	577.00 to 563.50	10.50 to 20.50	575.50 to 565.50	Reddish Brown Silty Clay
OMW-C5	601.39	604.37	32.0	12.50 to 30.00	588.89 to 571.39	16.00 to 26.00	585.39 to 575.39	Reddish Brown Silty Clay
OMW-C7	599.3*	601.40	22.0	6.00 to 22.00	593.30 to 577.30	11.00 to 21.00	588.30 to 578.30	Reddish Brown Silty Clay
E.I. DuPont Yerkes Plant Site (Registry Number 915019)								
MW-3I	603.10	605.01	??	?? to ??	?? to ??	?? to 21.50	?? to 581.60	Reddish Brown Silty Clay
Niagara Mohawk Power Corporation - Huntley Plant								
NM-A	575.5*	577.84	??	?? to ??	?? to ??	?? to 19.60	?? to 555.90	No Construction Diagram Available
NM-B	575.75*	578.58	??	?? to ??	?? to ??	?? to 18.00	?? to 557.75	No Construction Diagram Available
USGS Monitoring Wells								
81-2TB	576.66	580.73	18.5	16.50 to 18.50	560.16 to 558.16	16.50 to 18.50	560.16 to 558.16	Grey Sand

Ft. AMSL Feet Above Mean Sea Level. **Ft. BGS** Feet Below Ground Surface. ***** Estimated Elevation.

Table B-3.
Monitoring Well Instrumentation Summary for Deep Zone Wells Installed in the Study Area.

Well Designation	Ground Surface Elevation (ft. AMSL)	Top of Riser Elevation (ft. AMSL)	Total Boring Depth (ft. BGS)	Sandpack Interval (ft. BGS)	Sandpack Interval (ft. AMSL)	Well Screen Interval (ft. BGS)	Well Screen Interval (ft. AMSL)	Screened Unit
Polymer Applications Site (Registry Number 915044)								
GW-1DD	589.38	591.61	60.0	52.50 to 60.00	536.88 to 529.38	55.00 to 60.00	534.38 to 529.38	Grey Silt, Sand & Gravel
GW-2DD	592.25	594.36	61.0	52.50 to 61.00	539.75 to 531.25	55.00 to 61.00	537.25 to 531.25	Reddish Brown Silty Clay; Grey Silt, Sand & Gravel
MW-8DD	580.89	582.11	55.0	48.00 to 55.00	532.89 to 525.89	50.00 to 55.00	530.89 to 525.89	Grey Silt, Sand & Gravel; Camillus Shale
MW-9DD	593.47	595.07	66.0	59.00 to 66.00	534.47 to 527.47	61.00 to 66.00	532.47 to 527.47	Grey Silt, Sand & Gravel
MW-10DD	575.88	577.59	50.0	36.00 to 43.00	539.88 to 532.88	37.00 to 42.00	538.88 to 533.88	Grey Clay; Grey Silt
MW-11DD	577.40	579.24	51.0	44.00 to 51.00	533.40 to 526.40	46.00 to 51.00	531.40 to 526.40	Grey Silt, Sand & Gravel; Camillus Shale
E.I. DuPont Yerkes Plant Site (Registry Number 915019)								
MW-1D	600.88	602.80	72.0	?? to ??	?? to ??	62.00 to 72.00	538.88 to 528.88	Grey Sand & Silt;
MW-2D	600.33	602.59	76.0	?? to ??	?? to ??	66.00 to 76.00	534.33 to 524.33	Grey Till; Camillus Shale
MW-3D	603.10	604.57	84.0	?? to ??	?? to ??	74.00 to 84.00	529.10 to 519.10	Grey Sand & Silt; Camillus Shale
MW-4D	602.17	604.54	84.0	?? to ??	?? to ??	74.00 to 84.00	528.17 to 518.17	Grey Clay; Grey Sand & Gravel; Camillus Shale
MW-7D	605.00	605.79	77.5	?? to ??	?? to ??	67.50 to 77.50	537.50 to 527.50	Grey Silt, Sand & Gravel; Camillus Shale
3M O-Cel-O Sponge Site (Registry Number 915148)								
MW-1	602.41	602.06	69.0	62.00 to 69.00	540.41 to 533.41	63.00 to 68.00	539.41 to 534.41	Reddish Brown Silty Clay
MW-2	602.62	602.21	71.4	65.00 to 70.00	537.62 to 532.62	66.00 to 70.00	536.62 to 532.62	Grey Sandy Gravel & Clay
MW-3	602.14	603.88	72.5	64.00 to 72.00	538.14 to 530.14	66.00 to 72.00	536.14 to 530.14	Reddish Brown Silty Clay; Grey Gravelly Sand & Clay
MW-4	602.04	601.84	77.0	67.20 to 77.20	534.84 to 524.84	69.20 to 74.20	532.84 to 527.84	Grey Silt, Sand & Gravel

Ft. AMSL **Feet Above Mean Sea Level.** **Ft. BGS** **Feet Below Ground Surface.**

TONAWANDA HYDROGEOLOGIC STUDY

Groundwater Elevations from Shallow Zone Wells

Date	B-4S	B-5S	GW-3S	GW-4S	MW-9S	DYF-1	DYF-3	Niagara River
8/21/1995	588.96	587.62	589.24	587.44	589.43	586.70	590.63	566.07
9/14/1995	587.76	587.51	588.73	586.84	588.57	586.41	589.60	565.80
10/12/1995	590.42	587.70	589.53	588.12	589.40	586.41	589.41	565.62
11/9/1995	590.82	587.74	589.61	588.14	589.82	585.73	589.21	565.44
1/12/1996	590.71	587.74	588.45	586.75	589.72	585.00	589.40	565.95

Date	B-6S	MW-12S	DYF-2	MW-11S	MW-13S	MW-14S	OMW-B3	Niagara River
8/21/1995	575.81	576.75	577.67	571.33	569.68	568.66	571.54	566.07
9/14/1995	575.42	576.26	575.37	570.54	569.56	568.37	570.64	565.80
10/12/1995	577.28	577.17	574.22	570.28	571.60	570.23	572.06	565.62
11/9/1995	577.43	576.86	573.65	572.92	571.76	570.57	573.54	565.44
1/12/1996	577.47	576.45	573.65	573.01	571.51	570.32	574.69	565.95

Date	OMW-C1	MW-1S	MW-2S	MW-3S	MW-4S	DYF-4	DYF-5	Niagara River
8/21/1995	596.98	597.06	598.05	600.06	599.80	594.62	596.72	566.07
9/14/1995	595.92	596.82	598.14	599.50	598.88	593.52	596.30	565.80
10/12/1995	596.76	598.66	599.72	600.52	600.44	594.81	595.93	565.62
11/9/1995	599.38	597.96	600.07	601.68	601.48	595.26	598.98	565.44
1/12/1996	600.52	598.56		602.07	601.76	595.38	599.51	565.95

Table C-1.
Historical Groundwater Elevations in Shallow Hydrogeologic Zone Wells Installed in the Study Area.
(All water levels and elevations measured in feet)

Well Designation	Top of Riser Elevation	October 3, 1983		October 17, 1983		June 25, 1990		August 8, 1990		June 14, 1993		January 4, 1994	
		Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation
Polymer Applications Site (Registry Number 915044)													
B-4S	594.26	4.25	590.01	4.08	590.18	4.12	590.14	4.72	589.54	4.01	590.25	3.24	591.02
B-5S	591.14	4.17	586.97	3.83	587.31	3.48	587.66	3.52	587.62	3.36	587.78	3.40	587.74
B-6S	582.13									4.78	577.35		
B-7S	578.12			5.17	572.95								
GW-3S	594.53					4.97	589.56	5.22	589.31	4.80	589.73	7.70	586.83
GW-4S	592.40					5.20	587.20	5.92	586.48			4.54	587.86
MW-9S	593.82												
MW-11S	579.22												
MW-12S	580.77												
MW-13S	577.58												
MW-14S	577.99												

Well Designation	Top of Riser Elevation	March 11, 1994		March 15, 1994		May 31, 1994		June 28, 1994		July 13, 1994			
		Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation		
Polymer Applications Site (Registry Number 915044)													
B-4S	594.26			2.22	592.04	4.83	589.43	2.90	591.36	4.35	589.91		
B-5S	591.14			3.24	587.90			3.42	587.72	3.65	587.49		
B-6S	582.13			5.54	576.59	5.86	576.27	5.96	576.17	6.04	576.09		
B-7S	578.12					1.07	577.05	0.92	577.20	1.06	577.06		
GW-3S	594.53			4.32	590.21			4.59	589.94	5.32	589.21		
GW-4S	592.40			2.64	589.76			4.40	588.00	5.50	586.90		
MW-9S	593.82	9.10	584.72	8.74	585.08	3.72	590.10	3.65	590.17	4.05	589.77		
MW-11S	579.22	5.96	573.26	4.70	574.52	5.55	573.67	6.18	573.04	6.51	572.71		
MW-12S	580.77	3.15	577.62	11.72	569.05	4.06	576.71	3.50	577.27	4.16	576.61		
MW-13S	577.58	5.68	571.90	5.50	572.08	6.12	571.46	5.90	571.68	6.54	571.04		
MW-14S	577.99	7.42	570.57	7.02	570.97	7.73	570.26	7.54	570.45	8.02	569.97		

Table C-1 (Continued).
Historical Groundwater Elevations in Shallow Hydrogeologic Zone Wells Installed in the Study Area.
(All water levels and elevations measured in feet)

Well Designation	Top of Riser Elevation	December 21, 1982		December 22, 1982		January 6, 1983		January 10, 1983		March 7, 1983		March 22, 1983	
		Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation
Dunlop Tire Corporation Site (Registry Number 915018)													
OMW-1	593.66	14.30	579.36	14.20	579.46	14.20	579.46	14.20	579.46	7.40	586.26	6.40	587.26
OMW-2	589.22	13.30	575.92	13.20	576.02	13.20	576.02	13.20	576.02	10.25	578.97	10.00	579.22
OMW-3	604.27	15.20	589.07	15.10	589.17	15.20	589.07	15.20	589.07	10.75	593.52	9.00	595.27
OMW-4	610.36	11.30	599.06	11.30	599.06	11.20	599.16	11.20	599.16	11.30	599.06	11.30	599.06
OMW-B2	586.73												
OMW-B3	580.58												
OMW-C1	603.84												
OMW-C6	603.00												

Well Designation	Top of Riser Elevation	April 5, 1983		April 19, 1983		May 2, 1983		May 16, 1983		May 31, 1983		September 21, 1983	
		Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation
Dunlop Tire Corporation Site (Registry Number 915018)													
OMW-1	593.66	5.80	587.86	5.40	588.26	5.50	588.16	5.15	588.51	5.95	587.71	8.17	585.49
OMW-2	589.22	8.55	580.67	7.40	581.82	6.95	582.27	6.55	582.67	6.95	582.27	5.63	583.59
OMW-3	604.27	7.50	596.77	6.30	597.97	5.60	598.67	5.25	599.02	4.90	599.37	9.96	594.31
OMW-4	610.36	11.30	599.06	11.30	599.06	11.30	599.06	11.30	599.06	11.30	599.06	8.94	601.42
OMW-B2	586.73												
OMW-B3	580.58												
OMW-C1	603.84												
OMW-C6	603.00												

Table C-1 (Continued).
Historical Groundwater Elevations in Shallow Hydrogeologic Zone Wells Installed in the Study Area.
(All water levels and elevations measured in feet)

Well Designation	Top of Riser Elevation	May 1, 1991		May 2, 1991		May 3, 1991		May 6, 1991		May 7, 1991		May 8, 1991	
		Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation
Dunlop Tire Corporation Site (Registry Number 915018)													
OMW-1	592.87*									4.39	588.48		
OMW-2	588.45*									5.06	583.39		
OMW-3	603.44*	2.93	600.51							3.20	600.24		
OMW-4	609.50*	7.94	601.56							8.12	601.38		
OMW-B2	586.73	18.42	568.31	18.34	568.39	18.28	568.45	18.00	568.73	17.94	568.79	17.90	568.83
OMW-B3	580.58	NA	Dry	15.50	565.08	14.20	566.38	9.46	571.12	8.87	571.71	8.80	571.78
OMW-C1	603.84					17.74	586.10	10.80	593.04	9.08	594.76	7.58	596.26
OMW-C6	603.00					19.05	583.95	18.58	584.42	18.44	584.56	18.38	584.62

Well Designation	Top of Riser Elevation	May 17, 1991		July 3, 1991		September 27, 1991							
		Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation						
Dunlop Tire Corporation Site (Registry Number 915018)													
OMW-1	592.87*	11.72	581.15	11.30	581.57	8.07	584.80						
OMW-2	588.45*	11.43	577.02	12.10	576.35	7.85	580.60						
OMW-3	603.44*	12.02	591.42	12.46	590.98	10.01	593.43						
OMW-4	609.50*	8.23	601.27	8.57	600.93	NA	Dry						
OMW-B2	586.73	18.46	568.27	16.96	569.77	8.55	578.18						
OMW-B3	580.58	14.03	566.55	7.38	573.20	9.45	571.13						
OMW-C1	603.84	9.96	593.88	6.85	596.99	9.75	594.09						
OMW-C6	603.00	18.18	584.82										

Table C-1 (Continued).
Historical Groundwater Elevations in Shallow Hydrogeologic Zone Wells Installed in the Study Area.
(All water levels and elevations measured in feet)

Well Designation	Top of Riser Elevation	October 2, 1979		December 28. 1990		September 1992		December 1992		March 1993		June 1993	
		Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation
E.I. DuPont Yerkes Plant Site (Registry Number 915019)													
MW-1S	602.74	4.17	598.57	3.03	599.71	3.57	599.17	3.10	599.64	2.20	600.54	3.62	599.12
MW-2S	602.85	4.17	598.68	2.48	600.37	3.21	599.64	2.43	600.42	2.15	600.70	2.80	600.05
MW-3S	604.88	3.63	601.25	2.81	602.07	3.02	601.86	2.82	602.06	2.48	602.40	3.06	601.82
MW-4S	604.26	3.42	600.84	2.57	601.69	2.54	601.72	2.58	601.68	2.38	601.88	2.61	601.65
DYF-1	592.24					15.08	577.16	9.82	582.42	7.50	584.74	10.09	582.15
DYF-2	591.35					17.15	574.20	14.93	576.42	11.27	580.08	12.21	579.14
DYF-3	595.27					5.53	589.74	4.98	590.29	4.61	590.66	4.38	590.89
DYF-4	600.12					4.73	595.39	4.37	595.75	3.98	596.14	4.42	595.70
DYF-5	605.24					5.43	599.81	4.50	600.74	3.80	601.44	5.32	599.92

Well Designation	Top of Riser Elevation	April 24, 1981		May 1, 1981		May 7, 1981		May 15, 1981		May 22, 1981		May 28, 1981	
		Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation
Niagara Mohawk Huntley Plant													
B-7	575.10**	8.78	566.32	8.78	566.32	8.87	566.23	9.03	566.07	8.70	566.40	9.03	566.07
B-14	572.88**	3.90	568.98	3.86	569.02	3.90	568.98	4.03	568.85	3.90	568.98	3.78	569.10
B-16	580.69**	11.44	569.25	11.60	569.09	11.60	569.09	11.65	569.04	11.60	569.09	11.65	569.04
B-17	578.27**	8.63	569.64	7.88	570.39	8.29	569.98	8.88	569.39	8.71	569.56	9.29	568.98
B-18	573.43**	6.17	567.26	5.09	568.34	5.09	568.34	5.13	568.30	5.17	568.26	5.42	568.01

Table C-1 (Continued).
Historical Groundwater Elevations in Shallow Hydrogeologic Zone Wells Installed in the Study Area.
(All water levels and elevations measured in feet)

Well Designation	Top of Riser Elevation	June 5, 1981		June 12, 1981		March 21, 1995							
		Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation						
Niagara Mohawk Huntley Plant													
B-7	575.10**	8.66	566.44	8.95	566.15								
B-14	572.88**	3.90	568.98	3.94	568.94	9.60	563.28						
B-16	580.69**	11.60	569.09	11.69	569.00								
B-17	578.27**	9.29	568.98	8.38	569.89								
B-18	573.43**	5.05	568.38	5.34	568.09	9.85	563.58						

* Top of Riser Resurveyed During URS Investigation. NA Not Applicable. ** Top of Riser Elevation Unknown. Referenced Elevation is Ground Surface.

Table C-2.
Historical Groundwater Elevations in Intermediate Hydrogeologic Zone Wells Installed in the Study Area.
(All water levels and elevations measured in feet)

Well Designation	Top of Riser Elevation	August 30, 1983		September 15, 1983		October 3, 1983		October 17, 1983		September 21, 1990		June 14, 1993	
		Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation
Polymer Applications Site (Registry Number 915044)													
B-2D	583.71	7.47	576.24	11.37	572.34	14.00	569.71	14.25	569.46	10.57	573.14	8.66	575.05
B-3D	591.14							4.58	586.56			3.75	587.39
B-4D	594.13					13.08	581.05	5.67	588.46			3.40	590.73
B-5D	591.24					20.00	571.24	19.00	572.24			4.01	587.23
B-6D	580.89					20.58	560.31	19.58	561.31			6.02	574.87
B-7D	578.15					16.17	561.98						

Well Designation	Top of Riser Elevation	January 4, 1994		March 15, 1994		May 31, 1994		June 28, 1994		July 13, 1994			
		Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation
Polymer Applications Site (Registry Number 915044)													
B-2D	583.71	8.60	575.11	7.65	576.06			8.03	575.68	8.00	575.71		
B-3D	591.14	3.53	587.61	3.00	588.14			3.61	587.53	4.45	586.69		
B-4D	594.13	3.88	590.25	2.57	591.56			3.03	591.10	3.90	590.23		
B-5D	591.24	4.30	586.94	11.04	580.20			4.18	587.06	4.90	586.34		
B-6D	580.89	5.80	575.09	14.00	566.89			7.20	573.69	6.72	574.17		
B-7D	578.15					1.96	576.19	0.74	577.41	1.57	576.58		

Well Designation	Top of Riser Elevation	April 29, 1991		April 30, 1991		May 1, 1991		May 2, 1991		May 3, 1991		May 6, 1991	
		Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation
Dunlop Tire Corporation Site (Registry Number 915018)													
OMW-A3	598.22	11.20	587.02	9.80	588.42	7.38	590.84	6.37	591.85	5.76	592.46	5.39	592.83
OMW-C5	604.37							NA	Dry	NA	Dry	NA	Dry

Table C-2 (Continued).
Historical Groundwater Elevations in Intermediate Hydrogeologic Zone Wells Installed in the Study Area.
(All water levels and elevations measured in feet)

Well Designation	Top of Riser Elevation	May 7, 1991		May 8, 1991		May 17, 1991		July 3, 1991		September 27, 1991			
		Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation
Dunlop Tire Corporation Site (Registry Number 915018)													
OMW-A3	598.22	5.50	592.72	5.63	592.59	5.45	592.77	10.98	587.24	16.75	581.47		
OMW-C5	604.37	29.34	575.03	29.38	574.99	27.70	576.67	22.41	581.96	21.15	583.22		
Well Designation	**Top of Riser Elevation**	**April 24, 1981**		**May 1, 1981**		**May 7, 1981**		**May 15, 1981**		**May 22, 1981**		**May 28, 1981**	
		Depth to Water	**Elevation**	**Depth to Water**	**Elevation**	**Depth to Water**	**Elevation**	**Depth to Water**	**Elevation**	**Depth to Water**	**Elevation**	**Depth to Water**	**Elevation**
Niagara Mohawk Huntley Plant													
B-8	575.10**	8.43	566.67	9.02	566.08	8.77	566.33	8.85	566.25	8.52	566.58	8.68	566.42
B-15	573.41**	6.01	567.40	7.59	565.82	7.59	565.82	7.68	565.73	7.26	566.15	7.34	566.07
B-17A	578.27**												
SB-ST-25A	577.11**												
Well Designation	**Top of Riser Elevation**	**June 5, 1981**		**June 12, 1981**		**May 27, 1983**		**May 31, 1983**		**June 1, 1983**		**June 13, 1983**	
		Depth to Water	**Elevation**	**Depth to Water**	**Elevation**	**Depth to Water**	**Elevation**	**Depth to Water**	**Elevation**	**Depth to Water**	**Elevation**	**Depth to Water**	**Elevation**
Niagara Mohawk Huntley Plant													
B-8	575.10**	8.52	566.58	8.77	566.33								
B-15	573.41**	7.30	566.11	7.51	565.90								
B-17A	578.27**												
SB-ST-25A	577.11**					8.20	568.91	8.25	568.86	8.25	568.86	8.40	568.71
Well Designation	**Top of Riser Elevation**	**June 15, 1983**		**June 17, 1983**		**June 21, 1983**		**June 22, 1983**		**September 26, 1983**		**September 27, 1983**	
		Depth to Water	**Elevation**	**Depth to Water**	**Elevation**	**Depth to Water**	**Elevation**	**Depth to Water**	**Elevation**	**Depth to Water**	**Elevation**	**Depth to Water**	**Elevation**
Niagara Mohawk Huntley Plant													
B-8	575.10**												
B-15	573.41**												

Table C-2 (Continued).
Historical Groundwater Elevations in Intermediate Hydrogeologic Zone Wells Installed in the Study Area.
(All water levels and elevations measured in feet)

Well Designation	Top of Riser Elevation	June 15, 1983		June 17, 1983		June 21, 1983		June 22, 1983		September 26, 1983		September 27, 1983	
		Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation
Niagara Mohawk Huntley Plant													
B-17A	578.27**												
SB-ST-25A	577.11**	8.30	568.81	8.40	568.71	8.45	568.66	8.38	568.73	8.50	568.61	8.60	568.51

Well Designation	Top of Riser Elevation	September 29, 1983		March 21, 1995									
		Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation
Niagara Mohawk Huntley Plant													
B-8	575.10**												
B-15	573.41**			10.50	562.91								
B-17A	578.27**			6.80	571.47								
SB-ST-25A	577.11**	8.79	568.32										

** Top of Riser Elevation Unknown. Referenced Elevation is Ground Surface. NA Not Applicable.

Table C-3.
Historical Groundwater Elevations in Deep Hydrogeologic Zone Wells Installed in the Study Area.
(All water levels and elevations measured in feet)

Well Designation	Top of Riser Elevation	August 30, 1983		September 15, 1983		September 21, 1983		October 3, 1983		October 17, 1983		June 25, 1990	
		Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation
Polymer Applications Site (Registry Number 915044)													
B-1DD	582.15	40.30	541.85	40.10	542.05	39.70	542.45	42.67	539.48	45.92	536.23	15.75	566.40
GW-1DD	591.61											25.07	566.54
GW-2DD	594.36											27.41	566.95
MW-8DD	582.11												
MW-9DD	595.07												
MW-10DD	577.59												
MW-11DD	579.24												

Well Designation	Top of Riser Elevation	August 8, 1990		June 14, 1993		January 4, 1994		March 11, 1994		March 15, 1994		May 31, 1994	
		Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation
Polymer Applications Site (Registry Number 915044)													
B-1DD*	582.15	38.02	544.13			16.36	565.79						
GW-1DD	591.61	38.03	553.58			25.80	565.81			35.09	556.52		
GW-2DD	594.36	37.33	557.03	26.96	567.40	27.78	566.58			27.47	566.89		
MW-8DD	582.11							16.14	565.97	15.82	566.29	15.27	566.84
MW-9DD	595.07							10.93	584.14	15.12	579.95	27.06	568.01
MW-10DD	577.59							7.15	570.44	8.91	568.68	10.05	567.54
MW-11DD	579.24							13.27	565.97	13.02	566.22	12.35	566.89

Well Designation	Top of Riser Elevation	June 28, 1994		July 13, 1994									
		Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation
Polymer Applications Site (Registry Number 915044)													
B-1DD*	582.15												
GW-1DD	591.61	45.39	546.22	46.08	545.53								

Table C-3 (Continued).
Historical Groundwater Elevations in Deep Hydrogeologic Zone Wells Installed in the Study Area.
(All water levels and elevations measured in feet)

Well Designation	Top of Riser Elevation	June 28, 1994		July 13, 1994									
		Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation
Polymer Applications Site (Registry Number 915044)													
GW-2DD	594.36	42.40	551.96	42.80	551.56								
MW-8DD	582.11	45.90	536.21	36.50	545.61								
MW-9DD	595.07	31.21	563.86	34.21	560.86								
MW-10DD	577.59	12.50	565.09	13.43	564.16								
MW-11DD	579.24	31.85	547.39	32.47	546.77								

Well Designation	Top of Riser Elevation	October 2, 1979		December 28, 1990		September 1992		December 1992		March 1993		June 1993	
		Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation
E.I. DuPont Yerkes Plant Site (Registry Number 915019)													
MW-1D	602.80	37.58	565.22	36.61	566.19	36.53	566.27	36.08	566.72	36.22	566.58	35.65	567.15
MW-2D	602.59	38.00	564.59	36.12	566.47	36.57	566.02	35.65	566.94	35.68	566.91	35.22	567.37
MW-3D	604.57	42.00	562.57	36.00	568.57	38.75	565.82	35.80	568.77	35.66	568.91	35.39	569.18
MW-4D	604.54	42.00	562.54	35.67	568.87	38.87	565.67	35.37	569.17	35.09	569.45	34.89	569.65
MW-7D	605.79	39.42	566.37	37.34	568.45	37.83	567.96	36.92	568.87	36.98	568.81	36.47	569.32

Well Designation	Top of Riser Elevation	May 8, 1992											
		Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation
3M O-Cel-O Sponge Site (Registry Number 915148)													
MW-1	604.03	34.88	569.15										
MW-2	605.56	36.43	569.13										
MW-3	604.09	34.98	569.11										

* Well Damaged; Replaced by Well MW-8DD.

Table C-4.
Historical Groundwater Elevations in Upper Bedrock Hydrogeologic Zone Wells Installed in the Study Area.
(All water levels and elevations measured in feet)

Well Designation	Top of Riser Elevation	December 21, 1982		December 22, 1982		January 6, 1983		January 10, 1983		March 7, 1983		March 22, 1983	
		Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation
Dunlop Tire Corporation Site (Registry Number 915018)													
BMW-1	588.62	54.00	534.62	54.30	534.32	60.00	528.62	50.80	537.82	55.45	533.17	51.75	536.87
BMW-2	610.62	61.50	549.12	61.30	549.32	54.70	555.92	57.90	552.72	62.45	548.17	61.70	548.92
Well Designation	**Top of Riser Elevation**	**April 5, 1983**		**April 19, 1983**		**May 2, 1983**		**May 16, 1983**		**May 31, 1983**		**September 21, 1983**	
		Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation
Dunlop Tire Corporation Site (Registry Number 915018)													
BMW-1	588.62	46.50	542.12	50.10	538.52	50.70	537.92	53.30	535.32	45.80	542.82	45.98	542.64
BMW-2	610.62	58.10	552.52	59.30	551.32	59.90	550.72	60.75	549.87	56.55	554.07	55.45	555.17
Well Designation	**Top of Riser Elevation**	**May 1, 1991**		**May 7, 1991**		**May 17, 1991**		**July 3, 1991**		**September 27, 1991**			
		Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation
Dunlop Tire Corporation Site (Registry Number 915018)													
BMW-1	587.74*			21.74	566.00	21.18	566.56	40.24	547.50	49.48	538.26		
BMW-2	609.79*	41.86	567.93	41.80	567.99	41.71	568.08	53.36	556.43	59.25	550.54		
Well Designation	**Top of Riser Elevation**	**May 27, 1983**		**May 31, 1983**		**June 1, 1983**		**June 13, 1983**		**June 15, 1983**		**June 17, 1983**	
		Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation
Niagara Mohawk Huntley Plant													
SB-ST-21	573.83**							14.12	559.71	13.90	559.93	14.05	559.78
SB-ST-25A	577.11**	38.05	539.06	29.80	547.31	32.75	544.36	32.15	544.96	34.25	542.86	36.15	540.96
A1	577.91**												
A2	575.59**												
A3	573.04**												
A4	575.44**												

Table C-4 (Continued).
Historical Groundwater Elevations in Upper Bedrock Hydrogeologic Zone Wells Installed in the Study Area.
(All water levels and elevations measured in feet)

Well Designation	Top of Riser Elevation	June 21, 1983		June 22, 1983		September 26, 1983		September 27, 1983		September 29, 1983		September 30, 1983	
		Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation
Niagara Mohawk Huntley Plant													
SB-ST-21	573.83**	12.90	560.93	13.00	560.83	9.40	564.43	9.80	564.03	10.05	563.78		
SB-ST-25A	577.11**	33.20	543.91	34.10	543.01	26.73	550.38	29.50	547.61	31.96	545.15		
A1	577.91**											38.80	539.11
A2	575.59**											10.46	565.13
A3	573.04**											8.13	564.91
A4	575.44**											34.98	540.46

Well Designation	Top of Riser Elevation	October 3, 1983		March 21, 1995									
		Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation	Depth to Water	Elevation
Niagara Mohawk Huntley Plant													
SB-ST-21	573.83**	9.81	564.02										
SB-ST-25A	577.11**	28.69	548.42										
A1	577.91**	31.99	545.92	15.00	562.91								
A2	575.59**	9.87	565.72	10.30	565.29								
A3	573.04**	7.32	565.72	9.50	563.54								
A4	575.44**	26.89	548.55	9.97	565.47								

* Top of Riser Resurveyed During URS Investigation. ** Top of Riser Elevation Unknown. Referenced Elevation is Ground Surface.

6.5 BASEMENT WALL STRUCTURAL MODELING PARAMETERS

6.5.1 Wall Properties

The wall was modeled as being solid concrete with a thickness of eight inches (8”) and a height of seven feet (7’). The concrete was modeled with a strength of 3,000 psi. The wall was also modeled as having soil on one side to a depth of six feet (6’), with the top of the wall one foot above the ground surface. Two wall lengths were used for comparison, a short wall 20-feet in length, and a long wall 40-feet in length. To perform the finite element analysis the wall was divided into 1-foot square elements along its length. Each element was the full thickness of the wall (eight inches).

6.5.2 Boundary Conditions

Several different boundary conditions were used to simulate conditions as found in the field. Supports were modeled as fixed, simple, or free. Fixed supports do not allow any translational movement, but do allow rotation in all directions. Simple supports are also able to rotate and only restrict translation in one direction. The simple supports were modeled to resist translation in the direction of the soil pressure in each of the models where simple supports were used. Free boundaries are able to rotate and translate in any direction.

Four boundary condition models were used in the analysis. They are as follows:

<u>Fixed-Simple:</u> In this model the left, right, and bottom boundaries were fixed and the top boundary was simply supported. The fixed supports model the performance of an intact basement. The simple support models the performance where the superstructure of the house is in contact with the top of the basement wall and provides lateral support.

<u>Fixed-Free:</u> In this model the left, right, and bottom boundaries were fixed and the top boundary was free. This was to simulate the condition where the top of the basement wall was not laterally supported by the structure of the house.

<u>Simple-Simple:</u> In this model the left, right, bottom, and top boundaries were simply supported. The left, right, and bottom simple supports model an alternate mode of performance for an intact basement. The top simple support models the performance of the superstructure laterally supporting the basement wall.

<u>Simple-Free:</u> In this model the left, right, and bottom boundaries were simply supported and the top was free. This model simulates the above model, with the top of the basement wall not laterally supported by the structure of the house.

6.5.3 Estimating Lateral Earth Pressures on Existing Walls due to Expansive Backfill Soils

As discussed in section 3.2.3, the laboratory test results for samples of basement wall backfill soils confirm that they contain expansive soils. When expansive soils are placed against basement walls, these soils can induce lateral pressures not accounted for in traditional Rankine and Coulomb earth pressure theories.

Section 5-3 of *Foundations in Expansive Soils* (USDOA, 1983) offers guidance for predicting lateral pressures from expansive soils against basement walls. The following equation is used to calculate lateral pressures from expansive soils at a given depth:

$$\sigma_h(z) = K_o \sigma_v(z) \leq \sigma_p(z)$$

where,

$\sigma_h(z)$= lateral pressure at depth z;

K_o = at-rest earth pressure coefficient for expansive backfill;

$\sigma_v(z)$= effective vertical stress at depth z (based on measured moisture contents and specific gravities for typical backfill soils, the moist and saturated unit-weight of typical backfill in Amherst can be estimated as 125 PCF);

$\sigma_p(z)$= passive earth pressure offered by undisturbed soils adjacent to backfill at depth z.

The use of K_o values in the range of 1 to 2 is recommended in *Foundations in Expansive Soils* (USDOA, 1983). This cited range of K_o is believed to be based on radial pressure measurements obtained during one-dimension compression tests of over consolidated clays (Brooker and Ireland, 1965). The actual earth pressures exerted by swelling backfills depends on a number of factors including the expansiveness of the backfill, localized surface drainage conditions, initial moisture content, cyclical moistening and drying of the backfill, desiccation cracking, infilling of desiccation cracks, etc. Therefore, K_o should not be assumed to be proportionally related to expansion index, plasticity index, and/or liquid limit. In Amherst, the undisturbed soils adjacent to basement wall backfill typically are over consolidated by desiccation and possess relatively high shear strength. These over consolidated soils are capable of developing relatively high passive earth pressures. Therefore, for typical conditions in Amherst, σ_p will exceed $K_o\sigma_v$. Surcharge loading of the ground surface from porches or other structures adjacent to basement walls should be considered when calculating σ_v.

6.5.4 Load Cases

Lateral earth-pressure profiles for the three types of backfill were calculated for both wet and dry conditions. For the wet condition the groundwater surface was assumed to be at the midpoint of the soil layer [midpoint between the footing and ground surface?]. The loads on the wall for each soil and the parameters used to calculate them are shown below.

Fine-Grained, Expansive		
Dry	*Wet*	
N/A	2.5	pressure @ water level (psi)
5	5	pressure @ footing (psi)
120	120	Unit weight (pcf)
N/A	N/A	Friction Angle (degrees)
1-2	**1**-2	K_0
N/A	3	Water El. (ft)

Fine-Grained,-Non- Expansive		
Dry	*Wet*	
N/A	1.102	pressure @ water level (psi)
2.204	2.931	pressure @ footing (psi)
120	120	Unit weight (pcf)
34^0	34^0	Friction Angle (degrees)
0.441	0.441	K_0
N/A	3	Water El. (ft)

Coarse-Grained		
Dry	*Wet*	
N/A	1.010	pressure @ water level (psi)
2.020	2.747	pressure @ footing (psi)
110	110	Unit weight (pcf)
34^0	34^0	Friction Angle (degrees)

0.441	0.441	K_0
N/A	3	Water El. (ft)

Based on these calculations the expansive clay soil generates the largest loading condition on the wall so therefore this load case was used in the finite element analysis.

6.6 GEOTECHNICAL CALCULATIONS

US Army Corps of Engineers - Buffalo District

Originator: RV

Date:

Project: Amherst Foundation Study

Subject: Derivation of theoretical S_v equation

$$S_{v(Theor)} = \frac{\varepsilon_v}{(\Delta w)} = \frac{(\Delta H)}{H_0 (\Delta w)}$$

For small isotropic strains:

$$3 \times \frac{(\Delta H)}{H_0} = \frac{(\Delta V)}{V_0}$$

Therefore,

$$S_{v(Theor)} = \frac{(\Delta V)}{3 V_0 (\Delta w)} = \frac{(\Delta V)}{3 V_0 \left(\frac{\Delta W_w}{W_s}\right)} = \frac{(\Delta V) W_s}{3 V_0 (\Delta W_w)}$$

All of the volume change is due to a loss/gain of water:

$$(\Delta W_w) = \gamma_w \times (\Delta V)$$

Therefore,

$$S_{v(Theor)} = \frac{(\Delta V) W_s}{3 V_0 \gamma_w (\Delta V)} = \frac{W_s}{3 V_0 \gamma_w} = \frac{\gamma_d}{3 \gamma_w}$$

Checked By:

STIFF STRATUM
SHRINKAGE SETTLEMENT CALCULATIONS

Site 4 - Perimeter Footing Shrinkage Calculations

		initial conditions for perimeter ftg.		S_v	
depth below footing (feet)	thickness of interval (feet)	interior footing moisture content	perimeter footing moisture content	strain vs. moisture slope	lineal shrinkage (in)
0					
	0.5	29.7	25.8		
0.5				0.61	0.1757
	0.5	28.4	27.5		
1					
	0.5	25.4	23.7		
1.5				0.61	0.1391
	0.5	27.7	25.6		
2					
	0.5	32.3	29.7		
2.5				0.61	0.3367
	0.5	36.2	29.6		
3					
	0.5	38.5	33.9		
3.5				0.61	0.4776
	0.5	43.2	34.75		
4					
	0.5	44.35	35.6		
4.5				0.61	0.6314
	0.5	45.5	37		
5					
				total	1.76

$\Delta h = h_0 S_v \Delta w$

$h_0 = 12''$

$\Delta w = \frac{(29.7+28.4)}{2} - \frac{(25.8+27.5)}{2}$

$= 2.4\% = 0.024$

$\Delta h = (12'')(0.61)(0.024) = 0.1757''$

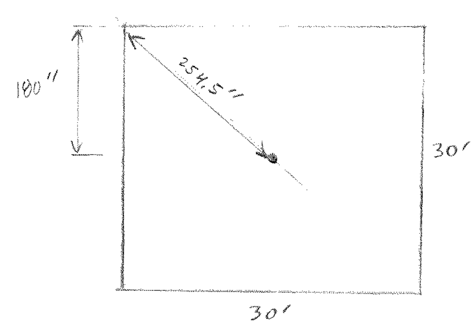
Site 7 - Perimeter Footing Shrinkage Calculations

		initial conditions for perimeter ftg.			
depth below footing (feet)	thickness of interval (feet)	interior footing moisture content	perimeter footing moisture content	strain vs. moisture slope	lineal shrinkage (in)
0					
	0.5	32.2	25.1		
0.5				0.61	0.3111
	0.5	30.2	28.8		
1					
	0.5	33.7	29.2		
1.5				0.61	0.4319
	0.5	37.6	30.3		
2					
	0.5	39.3	32.45		
2.5				0.61	0.5618
	0.5	43.1	34.6		
3					
	0.5	42.1	34.25		
3.5				0.61	0.5545
	0.5	41.2	33.9		
4					
				total	1.86

SOFT STRATUM
CONSOLIDATION SETTLEMENT CALCULATIONS

STRESS-STRAIN RESPONSE OF SOFT STRATUM FOR HYPOTHETICAL LOADING CONDITIONS

	settlement due to excavation (feet)	settlement due to filling (feet)	settlement due to water drop (feet)	combined settlement (feet)	settlement due to excavation (inches)	LOAD CASE I 1/2 the settlement due to excavation (inches)	All of the rebound occurs after construction settlement due to filling (inches)	No rebound occurs after construction settlement due to filling (inches) without rebound	one-half of the rebound occurs after construction LOAD CASE II settlement due to filling (inches) without 1/2 rebound
center	-0.056	-0.043	-0.013	0.036	-0.671	-0.335	-0.511	0.160	-0.175
edge	-0.028	-0.006	0.035	0.098	-0.336	-0.168	-0.073	0.263	0.095
corner	-0.015	0.033	0.063	0.132	-0.177	-0.088	0.396	0.573	0.484
						254.500		L1	254.500
						180.000		L2	180.000
DIFFERENTIAL SETTLEMENT BETWEEN CENTER AND CORNER						-0.247			-0.66
DIFFERENTIAL SETTLEMENT BETWEEN MIDPOINT AND CORNER						-0.080			-0.389
ANGULAR DISTORTION CENTER TO CORNER						-0.00097			-0.00259
ANGULAR DISTORTION MIDPOINT TO CORNER						-0.00044			-0.00216
ANGULAR DISTORTION CENTER TO CORNER (LENGTH PER INCH OF SETT.)						-1030 inches			-386 inches
ANGULAR DISTORTION MIDPOINT TO CORNER (LENGTH PER INCH OF SETT.)						-2260 inches			-463 inches



STRESS-STRAIN RESPONSE OF SOFT STRATUM FOR HYPOTHETICAL LOADING CONDITIONS (CONTINUED)

	100% rebound After construction settlement due to water drop (inches)	0% rebound After construction settlement due to water drop (inches) without rebound	LOAD CASE III settlement due to water drop (inches) without 1/2 rebound	combined settlement (inches)	combined settlement (inches) without rebound	LOAD CASE IV combined settlement (inches) without 1/2 rebound
Center	-0.157	0.514	0.178	0.426	1.097	0.761
Edge	0.420	0.756	0.588	1.177	1.513	1.345
Corner	0.754	0.931	0.842	1.583	1.760	1.671
		L1	254.500		L1	254.500
		L2	180.000		L2	180.000
			-0.66			-0.91
			-0.255			-0.327
			-0.00261			-0.00358
			-0.00141			-0.00182
			-383			-280
			-707			-551

STRESS-STRAIN RESPONSE UNDER CENTER OF BASEMENT EXCAVATION

excavation depth (feet)	6
excavation unit-weight (PCF)	125

perimeter fill thickness (feet)	2
fill unit-weight (PCF)	115

pressure from weight of house (PSF)	270

Interval	RR	CR	P_c
9'-21'	0.025	0.26	2600 PSF
21'-31'	0.015	0.15	1957 PSF

interval	interval thickness (feet)	initial effective stress at center of interval (PSF)	corresponding strain	change in stress due to excavation (PSF)	final stress after excavation (PSF)	corresponding strain	corresponding settlement (Feet)	corresponding settlement (inches)
9 - 21	12	1405	0.0000	-422.4	982.6	-0.00389	-0.04669	-0.56024
21-31	10	1957	0.0000	-259.2	1697.8	-0.00092	-0.00922	-0.11060
						total	$\boxed{-0.056}$ → 100% rebound	

$L = 15'$
$B = 15'$
$z_{upper} = 9'$
$z_{Lower} = 20'$
$B/z_{upper} = 1.67$
$B/z_{Lower} = 0.75$
$L/z_{upper} = 1.67$
$L/z_{Lower} = 0.75$
$I_{upper} = 0.22 \times 4 = 0.88$
$I_{Lower} = 0.135 \times 4 = 0.54$
$\Delta\sigma_{upper} = -480 PSF \times 0.88 = -422$
$\Delta\sigma_{Lower} = -480 PSF \times 0.54 = -259$

$\varepsilon_{upper} = 0.025 \times (\log 983 - \log 1405)$
$= -0.0039$
$\varepsilon_{Lower} = 0.015 \times (\log 1698 - \log 1957)$
$= 0.00092$

STRESS-STRAIN RESPONSE UNDER CENTER OF BASEMENT EXCAVATION

$982.6 + 230 - 165.6 = 1047$

	stress increase due to infinite fill (PSF)	reduction due to basement (PSF)	final stress after fill (PSF)	corresponding strain		corresponding settlement (Feet)	corresponding settlement (inches)
9'-21'	230	-165.6	1047	-0.003201	$= 0.025\,(\log 1047 - \log 1405)$	-0.038	-0.461
21'-31'	230	-92	1835.8	-0.000413	$= 0.015\,(\log 1836 - \log 1957)$	-0.004	-0.050
				total		$\boxed{-0.043}$	

-0.043 → Final net rebound due to excavation and perimeter filling

$1697.8 + 230 - 92 = 1835.8$

$L = 15'$

$B = 15'$

$Z_{upper} = 15'$

$Z_{lower} = 26'$

$B/Z_{upper} = 1$

$B/Z_{lower} = 0.58$

$L/Z_{upper} = 1$

$L/Z_{lower} = 0.58$

$I_{upper} = 0.18 \times 4 = 0.72$

$I_{lower} = 0.10 \times 4 = 0.40$

$\Delta\sigma_{upper} = -230\,PSF \times 0.72 = -166$

$\Delta\sigma_{lower} = -230\,PSF \times 0.40 = -92$

STRESS-STRAIN RESPONSE UNDER CENTER OF BASEMENT EXCAVATION

Depth	neg. Pore pressure
9	250 PSF
13	0 PSF
21	0 PSF

Average For 12' Thick Stratum = 41.6 PSF

stress increase due to drop in water table (PSF) **250** $= 4' \times \gamma_w$

$982.6 + 250 + 41.6 = 1274.2$ PSF

$1047 + 250 + 41.6 = 1338.6$ PSF

$0.025 \times (\log 1339 - \log 1405) = -0.005$

	adjustment for neg. pore pressure in upper 4' (PSF)	final stress no fill water drop (PSF)	corresponding strain	corresponding settlement (Feet)	final stress fill and water drop (PSF)	corresponding strain	corresponding settlement (Feet)
9'-21'	41.600	1274.200	-0.0011	-0.013	1338.600	-0.0005	-0.006
21'-31'		1947.800	0.0000	0.000	2085.800	0.0042	0.042
			total	$\boxed{-0.013}$			$\boxed{0.036}$

Final net compression due to excavation, Perimeter filling, and water table drop

$1697.8 + 250 = 1947.8$ PSF

$1835.8 + 250 = 2085.8$ PSF

$0.15 (\log 2086 - \log 1957) = 0.0042$

Final net rebound due to excavation & water table drop

Interval	RR	CR	P_c
9'-21'	0.025	0.26	2600 PSF
21'-31'	0.015	0.15	1957 PSF

STRESS-STRAIN RESPONSE UNDER MIDPOINT OF BASEMENT WALL

excavation depth (feet)	6
excavation unit-weight (PCF)	**125**
structural pressure (PSF)	**270**

perimeter fill thickness (feet)	**2**
fill unit-weight (PCF)	**115**

interval	interval thickness (feet)	intitial effective stress at center of interval (PSF)	corresponding strain		change in stress due to excavation (PSF)	final stress after excavation (PSF)	corresponding strain	corresponding settlement (Feet)
9 - 21	12	1405		0.0000	-220.8	1184.2	-0.0019	-0.022
21-31	10	1957		0.0000	-163.2	1793.8	-0.0006	-0.006
							total	$\boxed{-0.028}$ → 100% rebound

$L = 30'$
$B = 15'$
$Z_{upper} = 9'$
$Z_{lower} = 20'$
$B/Z_{upper} = 1.67$
$B/Z_{lower} = 0.75$
$L/Z_{upper} = 3.33$
$L/Z_{lower} = 1.5$
$I_{upper} = 0.23 \times 2 = 0.46$
$I_{lower} = 0.17 \times 2 = 0.34$
$\Delta\sigma_{upper} = -480 \times 0.46 = 221$
$\Delta\sigma_{lower} = -480 \times 0.34 = 163$

$\varepsilon_{upper} = 0.025 \times (\log 1184 - \log 1405)$
$= -0.0019$

$\varepsilon_{lower} = 0.015(\log 1794 - \log 1957)$
$= -0.0006$

STRESS-STRAIN RESPONSE UNDER MIDPOINT OF BASEMENT WALL

$1184.2 + 230 - 92 = 1322$ PSF

	stress increase due to infinite fill (PSF)	reduction due to basement (PSF)	final stress after fill (PSF)	corresponding strain		corresponding settlement (Feet)
9'-21'	230	-92	1322.2	-0.0007	$= 0.025\,(\log 1322 - \log 1405)$	-0.008
21'-31'	230	-62.1	1961.7	0.0002	$= 0.15\,(\log 1962 - \log 1957)$	0.002
				total		$\boxed{-0.006}$

Final net rebound due to excavation and perimeter filling

$1793.8 + 230 - 62 = 1962$ PSF

$L = 30'$

$B = 15'$

$Z_{upper} = 15'$

$Z_{lower} = 26'$

$B/Z_{upper} = 1$

$B/Z_{lower} = 0.58$

$L/Z_{upper} = 2$

$L/Z_{lower} = 1.15$

$I_{upper} = 0.20 \times 2 = 0.40$

$I_{lower} = 0.135 \times 2 = 0.27$

$\Delta\sigma_{upper} = -230\,PSF \times 0.40 = -92\,PSF$

$\Delta\sigma_{lower} = -230\,PSF \times 0.27 = -62\,PSF$

Depth	neg. Pore Pressure
9	250 PSF
13	0 PSF
21	0 PSF

Average for 12' Thick stratum = 41.6 PSF

STRESS-STRAIN RESPONSE UNDER MIDPOINT OF BASEMENT WALL

stress increase due to drop in water table (PSF)	**250**

$= 4' \times \gamma_w$

$1184.2 + 250 + 41.6 = 1475.8$ PSF

$1322.2 + 250 + 41.6 = 1613.8$ PSF

$0.025 (\log 1614 - \log 1405) = 0.0015$

	adjustment for neg. pore pressure in upper 4' (PSF)	final stress no fill water drop (PSF)	corresponding strain	corresponding settlement (Feet)	final stress fill and water drop (PSF)	corresponding strain	corresponding settlement (Feet)
9'-21'	41.600	1475.800	0.0005	0.006	1613.800	0.0015	0.018
21'-31'		2043.800	0.0029	0.029	2211.700	0.0080	0.080
			total	0.035			0.098

$1793.8 + 250 = 2043.8$ PSF

$0.15 \times (\log 2044 - \log 1957) = 0.0029$

$0.15 (\log 2212 - \log 1957) = 0.008$

$1961.7 + 250 = 2211.7$

0.035: Final net compression due to excavation & water table drop

0.098: Final net compression due to excavation, perimeter filling, and water table drop

Interval	RR	CR	P_c
9'-21'	0.025	0.26	2600 PSF
21'-31'	0.015	0.15	1957 PSF

STRESS-STRAIN RESPONSE UNDER CORNER

excavation depth (feet)	6
excavation unit-weight (PCF)	**125**
structural pressure (PSF)	**270**

perimeter	
fill thickness (feet)	**2**
fill unit-weight (PCF)	**115**

interval	interval thickness (feet)	intitial effective stress at center of interval (PSF)	corresponding strain	change in stress due to excavation (PSF)	final stress after excavation (PSF)	corresponding strain	corresponding settlement (Feet)
9 - 21	12	1405	0.0000	-115.2	1289.8	-0.0009	-0.011
21-31	10	1957	0.0000	-103.2	1853.8	-0.0003	-0.003
						total	$\boxed{-0.015}$ → 100% rebound

$L = 30'$

$B = 30'$

$z_{upper} = 9'$

$z_{Lower} = 20'$

$B/z_{upper} = 3.33$

$B/z_{Lower} = 1.5$

$L/z_{upper} = 3.33$

$L/z_{Lower} = 1.5$

$I_{upper} = 0.24$

$I_{Lower} = 0.215$

$\Delta\sigma_{upper} = -480(0.24) = -115$

$\Delta\sigma_{Lower} = -480(0.215) = -103$

$\varepsilon_{upper} = 0.025(\log 1290 - \log 1405)$
$= 0.0009$

$\varepsilon_{Lower} = 0.015(\log 1854 - \log 1957)$
$= 0.0003$

STRESS-STRAIN RESPONSE UNDER CORNER

$1289.8 + 230 - 52.9 = 1467\ PSF$

	stress increase due to infinite fill (PSF)	reduction due to basement (PSF)	final stress after fill (PSF)	corresponding strain		corresponding settlement (Feet)
9'-21'	230	-52.9	1466.9	0.0005	$= 0.025(\log 1467 - \log 1405)$	0.006
21'-31'	230	-43.7	2040.1	0.0027	$= 0.15(\log 2040 - \log 1957)$	0.027
				total		$\boxed{0.033}$

→ Final net settlement due to excavation and perimeter filling

$1853.8 + 230 - 43.7 = 2040\ PSF$

$L = 30'$

$B = 30'$

$Z_{upper} = 15'$

$Z_{lower} = 26'$

$B/Z_{upper} = 2$

$B/Z_{lower} = 1.15$

$L/Z_{upper} = 2$

$L/Z_{lower} = 1.15$

$I_{upper} = 0.23$

$I_{lower} = 0.19$

$\Delta\sigma_{upper} = -230PSF \times 0.23 = -53$

$\Delta\sigma_{lower} = -230PSF \times 0.19 = -44$

STRESS-STRAIN RESPONSE UNDER CORNER

stress increase due to drop in water table (PSF)	250

$1289.8 + 250 + 41.6 = 1581.4$ PSF

$1466.9 + 250 + 41.6 = 1758.5$ PSF

$\rightarrow 0.025\,(\log 1759 - \log 1405) = 0.0024$

	adjustment for neg. pore pressure in upper 4' (PSF)	final stress no fill water drop (PSF)	corresponding strain	corresponding settlement (Feet)	final stress fill and water drop (PSF)	corresponding strain	corresponding settlement (Feet)
9'-21'	41.600	1581.400	0.0013	0.015	1758.500	0.0024	0.029
21'-31'		2103.800	0.0048	0.048	2290.100	0.0103	0.103
				$\boxed{0.063}$			$\boxed{0.132}$

$1853.8 + 250 = 2103.8$ PSF

$2040.1 + 250 = 2290.1$ PSF

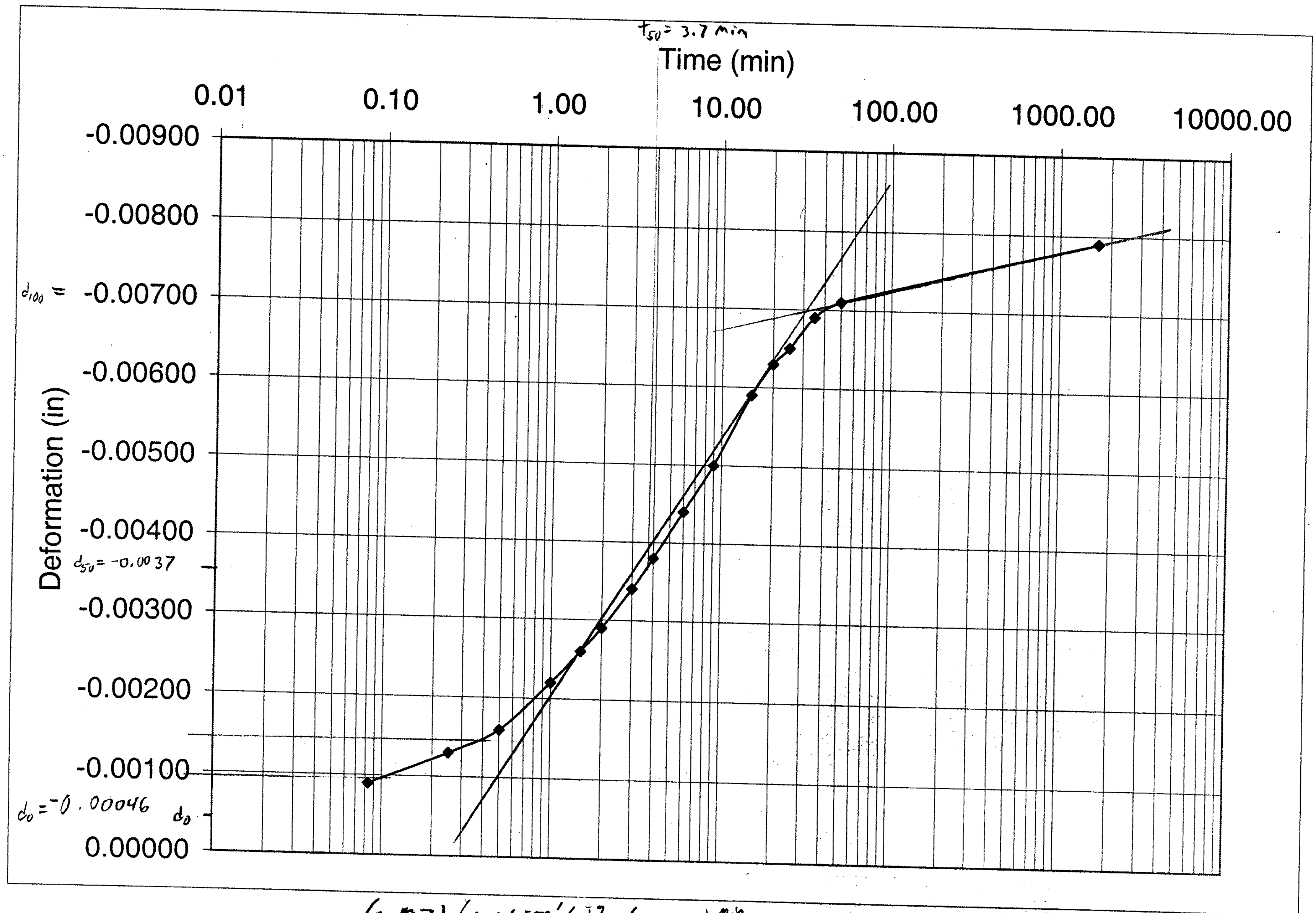
$\boxed{0.132} \rightarrow$ Final net compression due to excavation, perimeter filling, and water table drop

$0.15 \times (\log 2104 - \log 1957)$
$= 0.0048$

$0.15\,(\log 2290 - \log 1957)$
$= 0.0103$

Final net compression due to excavation & water table drop

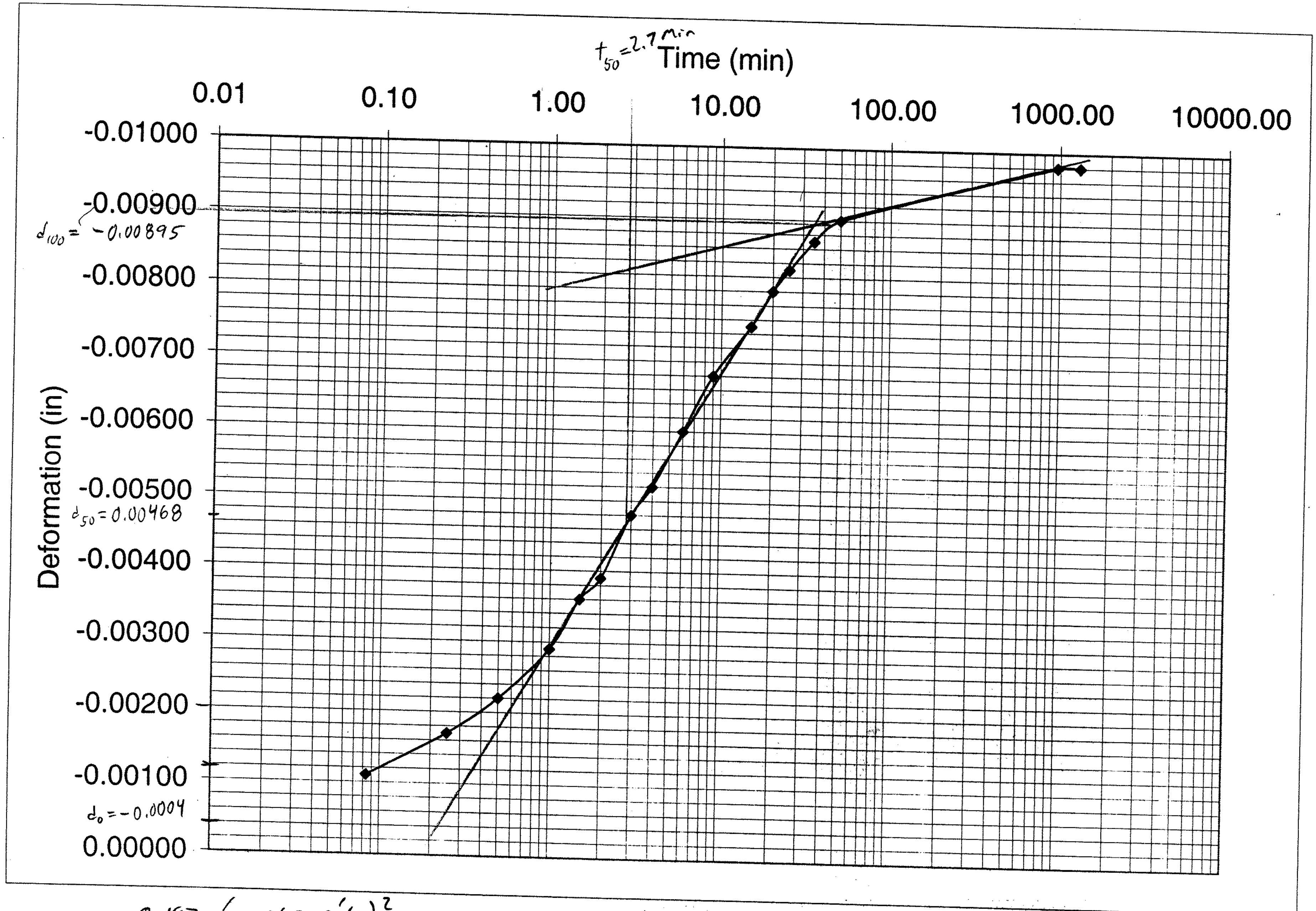
1500 PSF TO 500 PSF



$$C_v = \frac{(0.197)(0.06559'/2)^2}{3.7\text{ min}} \times (60 \times 24)\frac{\text{min}}{\text{day}} = 0.082\frac{ft^2}{day}$$

$h_0 = 0.06268$

2000 PSF TO 500 PSF



$$C_v = \frac{0.197\,(0.06268'/2)^2}{2.7\ \text{min}} \times (60 \times 24)\frac{\text{min}}{\text{day}} = 0.103\ \frac{ft^2}{\text{day}}$$

US Army Corps of Engineers
Buffalo District

Originator:
Date:
Project:
Subject:

Time - Rate consolidation

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$\boxed{\text{50\% rebound and 50\% compression}}$

$$t_{50} = \frac{(0.197)(11\,ft)^2}{0.08\,\frac{ft^2}{day}} = 298\,day$$

$$C_v = 0.08\,\frac{ft^2}{day}$$

$\boxed{\text{90 \% compression}}$

$$t_{90} = \frac{(0.848)(11\,ft)^2}{0.08\,\frac{ft^2}{day}} = 1280\,day \rightarrow 3.5\text{ years}$$

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$\boxed{\text{50 \% rebound}}$

$$t_{50} = \frac{(0.197)(11\,ft)^2}{0.10\,\frac{ft^2}{day}} = 238\,day$$

$$C_{v\,comp.} = 0.04\,\frac{ft^2}{day}$$

$$C_{v\,reb.} = 0.10\,\frac{ft^2}{day}$$

$\boxed{\text{50 \% compression}}$

$$t_{50} = \frac{(0.197)(11 ft^2)}{0.04\,\frac{ft^2}{day}} = 596\,day$$

90% compression

$$t_{90} = \frac{(0.848)(11ft)^2}{0.04\,\frac{ft^2}{day}} = 2565\,day \rightarrow 7\text{ years}$$

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Checked By:

Table GA.1 – Calculated Post-Construction Settlement/Rebound due to Strain Response of Soft Stratum at Site 29 in Amherst, NY (100% of the rebound occurs after construction)

LOCATION IN BASEMENT	CASE [V] (INCHES)	CASE [VI] (INCHES)	CASE [VII] (INCHES)	CASE [VIII] (INCHES)
Center	0.7 upward	0.5 upward	0.2 upward	0.4 downward
Wall Midpoint	0.3 upward	0.1 upward	0.4 downward	1.2 downward
Corner	0.2 upward	0.4 downward	0.8 downward	1.6 downward

[V] 100% of excavation rebound occurs after house construction

[VI] 100% of excavation rebound occurs after house construction and lot is raised with 2 feet of fill placed around the perimeter of the house after construction

[VII] 100% of excavation rebound occurs after house construction and water table drops 4 feet after construction

[VIII] 100% of excavation rebound occurs after house construction, lot is raised with 2 feet of fill placed around the perimeter of the house after construction, and water table drops 4 feet after construction

Table GA.2 – Calculated Post-Construction Settlement/Rebound due to Strain Response of Soft Stratum at Site 29 in Amherst, NY (0% of the rebound occurs after construction)

LOCATION IN BASEMENT	CASE [IX] (INCHES)	CASE [X] (INCHES)	CASE [XI] (INCHES)	CASE [XII] (INCHES)
Center	0.0	0.2 downward	0.5 downward	1.1 downward
Wall Midpoint	0.0	0.3 downward	0.8 downward	1.5 downward
Corner	0.0	0.6 downward	0.9 downward	1.8 downward

[IX] 0% of excavation rebound occurs after house construction

[X] 0% of excavation rebound occurs after house construction and lot is raised with 2 feet of fill placed around the perimeter of the house after construction

[XI] 0% of excavation rebound occurs after house construction and water table drops 4 feet after construction

[XII] 0% of excavation rebound occurs after house construction, lot is raised with 2 feet of fill placed around the perimeter of the house after construction, and water table drops 4 feet after construction

6.7 GUIDELINES FOR DESIGN/CONSTRUCTION

The problems of lateral pressure and excessive settlement can be overcome if the subsurface conditions and their interaction with a proposed structure are thoroughly understood and considered in the planning, design, and construction phases.

Typically, houses in Amherst are supported by traditional shallow foundation systems consisting of strip footings that support exterior walls, and spread footings, which support interior columns (Figure 8, Photo 14). Even though footings below basements may be placed up to 10 feet below the ground surface, they are still classified as shallow footings.

The assessment of settlement of shallow foundations is not usually performed for routine house design in Amherst. It is assumed that when a footing is designed for a contact pressure considered allowable, the differential settlements will be within the allowable range. As discussed above, many traditional shallow foundation systems supporting houses in Amherst have not performed as expected. Significant differential settlements across house foundations in Amherst were observed during site inspections. Factors other than footing contact pressure can contribute to problematic settlements of traditional shallow foundation systems in Amherst. Therefore, simply limiting footing bearing pressure to an allowable contact pressure may not be sufficient to limit settlements to tolerable magnitudes.

Two major soil conditions are suspected to be contributing to damaging differential settlements across house foundations in Amherst. These factors include (1) differential shrink/swell of relatively stiff clay soils directly beneath foundations, and (2) laterally variable strain response of underlying soft soil strata due to changes in effective stress caused by basement excavation, placement of fill around the perimeter of houses, and/or changes in water table elevation. Selection and design of shallow foundation systems should consider the potential for long-term differential settlements.

Significant cracking and displacements of basement walls induced by lateral pressures were observed during site inspections (Photo 6). As discussed in Section 3, four sources are suspected to be contributing to lateral pressures on basement walls in Amherst. These four sources include: (1) pressure from soil weight, (2) pressure from soil swell, (3) hydrostatic pressure, and (4) pressure from frost. Design of basement walls should consider these potential sources of lateral pressure and account for them.

The *Residential Code of New York State* (NYSDOS, 2003) includes requirements for house foundation design and construction. Considering the potentially problematic subsurface conditions in Amherst, practical application of these requirements may not ensure acceptable long-term performance of residential foundations.

We recommend that the new guidelines for residential foundation design/construction be applied at sites meeting any one of the following criteria.

- Sites with soils having a plasticity index greater than or equal to 15.

- Sites with very soft, soft, or firm fine-grained soils exhibiting standard penetration test (ASTM D 1586) N-values less than or equal to 8.

- Sites with fill material extending below proposed footing elevation.

Based on the laboratory testing conducted for this study, only till-derived soil samples (Sites 13 and 27) did not have plasticity indices greater or equal to 15 (Table 9).

In general, the new guidelines should facilitate design and construction of engineered foundations based on a site-specific geotechnical engineering evaluation (Phase I). Using the findings of the geotechnical engineering evaluation, foundation design (Phase II) should be performed by a licensed engineer. The licensed engineer who designs the foundation should be considered the "engineer of record," and she/he may or may not be the engineer performing the geotechnical engineering evaluation. The final requirement for an engineered foundation is that foundation construction should be observed and documented (Phase III) to ensure that the foundation is constructed in accordance with the provisions of the foundation design.

6.7.1 Phase I - Geotechnical Evaluation

Prior to foundation design, a site-specific geotechnical engineering evaluation should be conducted by a geotechnical engineer who is a Professional Civil Engineer (PE) registered in the State of New York. The scope of the geotechnical engineering evaluation should be sufficient to identify subsurface conditions relevant to long-term performance of a foundation system and basement walls, and to facilitate their design.

The specific scope of a geotechnical exploration and laboratory testing program should be coordinated with the engineer of record, and it should be sufficient to facilitate the geotechnical engineering evaluation. The specific scope of a geotechnical exploration and laboratory testing program depends on many factors including but not limited to the type of house to be constructed, available information regarding subsurface conditions at or near the site, the type of foundation system to be used at the site, and the level of conservatism to be used in design. Therefore, the specific scope of a geotechnical exploration and laboratory testing program should be determined by the geotechnical engineer to facilitate her/his geotechnical evaluation. The findings of the geotechnical evaluation should be presented to the engineer of record in a geotechnical report. The geotechnical report should include recommendations to facilitate design and construction of a foundation system and basement walls that will perform satisfactorily over the design life of the house.

6.7.2 Phase II – Foundation Design

The foundation design engineer should be the engineer of record and should be a Registered Professional Engineer (PE) in New York State. The engineer of record may or may not be the same individual who performed the geotechnical evaluation. If the geotechnical and foundation design engineering are not performed by the same individual, close collaboration between the engineer of record and the geotechnical engineer is essential. Foundation design includes design of the foundation system, design of basement walls, and preparation of plans and specifications. These three components of the foundation design are discussed below.

6.7.3 Design of Foundation System

A house foundation system needs to be capable of supporting the house without undergoing movements that cause structural damage or functional impairment. Potential for *long-term* differential foundation settlement is the primary design consideration.

A rational approach for designing shallow foundation systems considering potential long-term settlements involves a two-step process. The first step is to predict the long-term support offered by foundation soils across the foundation footprint. Long-term support offered by soils beneath house foundations in Amherst can be influenced by moisture content changes in the stiff stratum as well as consolidation of the firm/soft stratum. Therefore, accurately predicting long-term support offered by foundation soils is very difficult. Considering the long-term support offered by foundation soils across the foundation footprint, the second step is to design a foundation system capable of supporting the house without undergoing movements that cause structural damage or functional impairment.

One approach for dealing with potential differential foundation settlement is to prevent settlement/uplift with deep foundation systems. Deep foundation systems utilize piles or piers to transfer foundation loads down to competent bearing strata located well below the bottom of the structure. In Amherst, the use of deep foundation systems is uncommon for new house construction, but it is commonly used for foundation repair. Deep foundations are not typically used for new house construction in Amherst due to their relatively higher cost.

6.7.4 Design of Basement Walls

Section 3 discussed the four sources suspected to be contributing to lateral pressures. Pressure from soil swell, hydrostatic pressure, and pressure from frost can be significantly reduced or eliminated by specifying coarse-grained backfill soils classified as SW, SP, GW, or GP in accordance with ASTM D2487. Such coarse-grained soils consist of sands and gravels containing less than 5% by weight finer than the #200 sieve. In order to minimize pressure from soil swell, the width of coarse-grained backfill material needs to be wide enough to buffer basement walls from expansive native soils. Therefore, the zone of coarse-grained backfill soils placed against the wall should extend out to a line extending from the outside edge of wall footings up to the finished ground surface at a 45-degree angle. Unless the backfill material will be supporting overlying foundations, heavy compaction of the backfill is not recommended to avoid elevated at-rest lateral earth pressures induced by compaction. The coarse-grained backfill should be capped with 12 inches of compacted clay to minimize surface water infiltration. The clay cap should be compacted with relatively light hand-held equipment. If fine-grained soils are used for backfill, basement walls should be designed to resist potential lateral hydrostatic, soil swell, and frost pressures. The ground surface adjacent to basement walls should be sloped away from walls at a minimum grade of 5% to minimize surface water infiltration. If settlement of the backfill occurs over time, fill should be added as necessary to maintain the minimum 5% slope away from walls. Roof gutters and downspouts should be maintained to ensure diversion of water away from basement walls. A geotextile filter fabric should be used between fine-grained soils and coarse-grained backfill soils to prevent migration of fine-grained soils into coarse-grained backfill. The geotextile filter fabric should have permittivity sufficient to ensure cross-plane flow of groundwater. A drainage system at the bottom of basement walls should be used to collect and remove water from backfill material.

Where basement walls are laterally supported at the top, deflection of basement walls may not be sufficient to fully mobilize active earth pressures. Therefore, at-rest earth pressures, which are greater than active earth pressures, can be assumed. The at-rest earth pressure distribution with depth can be estimated by multiplying the vertical effective stress within the retained soil by an at-rest earth pressure coefficient. At-rest earth pressure coefficients for SW, SP, GW, and GP soils placed without mechanical compaction can be estimated using the following equation:

$$K_o = 1 - \sin \phi$$

where,

K_o = Coefficient of at-rest earth pressure;

ϕ = Angle of internal friction of retained soil.

The following table lists typical soil parameters for lightly-compacted SW, SP, GW, and GP soils.

SOIL TYPE	ϕ - ANGLE OF INTERNAL FRICTION	MOIST UNIT-WEIGHT (PCF)
SW	32	120
SP	31	115
GW	35	120
GP	33	115

Walls should be supported at the top in accordance with the wall design assumptions prior to backfilling. Surcharge loading of the ground surface from porches or other structures adjacent to basement walls should be considered when estimating lateral earth pressures.

The potential impact of the backfill material on the long-term moisture regime beneath foundations should be considered during selection and design of the foundation system. The use of coarse-grained backfill soils could potentially increase the amount of water available to foundation soils relative to fine-grained backfill soils.

6.7.5 Structural Design Considerations

The foundation design engineer should consider the following:

- Shallow individual footings or continuous footings shall not be used in areas with expansive soils unless for the foundation and superstructure are designed to account for the potential movement generated in this type soil.

- Foundation wall thickness should be calculated for each home to assure that the wall thickness and any necessary reinforcement steel can withstand the forces placed upon it.

- Compensate for concentrated loads such as fireplaces, columns and heavy interior line loads.

6.7.6 Preparation of Plans and Specifications

The foundation design engineer should prepare the plans and specifications for the foundation system and basement walls. Plans should be signed and stamped by the engineer of record for each site or lot location. Plans should identify the client's name and engineer's name, address and telephone number; and the source and description of the geotechnical data. At a minimum, the signed and stamped engineer's drawings should include:

- A plan view of the foundation locating all major structural components and reinforcement;

- Sufficient information to show details of beams, piers, basement walls, drainage details including landscaping and tree locations near the foundation walls, etc., if such features are integral to the foundation; and

- Sufficient information for the proper construction and observation by field personnel.

In addition, the engineer's specifications should include:

- Concrete specifications including compressive strengths;

- Site preparation requirements;

- Reinforcement specification including locations, sizes, types, numbers, and strengths;
- Fill material and placement requirements; and
- The schedule of required construction observations, testing, and the submission of this information back to the engineer of record.

6.7.7 Phase III - Observation and Documentation of Foundation Construction

The foundation should be built in accordance with the design. The engineer of record should *approve any design modifications*. The engineer of record or a qualified delegate should perform observation and documentation of foundation construction. The qualified delegate should be a staff member under his/her direct supervision, or an outside agent approved by the engineer of record. The observation reports should be provided to the engineer of record. The engineer of record should *issue a compliance letter* indicating that construction of the foundation was in conformance with the engineer's plans and specifications including any modifications or alterations authorized. Additionally, non-compliance letter shall be issued if any part of the foundation construction fails to meet the requirements put forth by the engineer of record

6.8 GUIDELINES FOR EVALUATION/REPAIR

Homeowners should employ a Professional Civil Engineer (PE) registered in the State of New York to evaluate foundation damage. The engineer should personally visit the site and recommend an appropriate scope for the evaluation. The scope of the evaluation should be sufficient to identify causative factors and provide recommendations regarding remediation. The scope of services to be provided by the engineer shall be jointly established and agreed to by both the homeowner and engineer.

The findings of the evaluation should be presented in a report signed and sealed by the engineer. The engineer should represent the homeowner and provide objective, confidential, and honest advice regarding maintenance and remedial options. The engineer should consider the cost effectiveness and practicality of the recommendations, the projected performance, and the needs of the homeowner. For example, periodic cosmetic repairs and door adjustments may be more feasible than comprehensive foundation repair. At a minimum, the report should include the following information.

1. Authorization and Scope
2. Property Location and Description
3. Sources of Information
4. Data
5. Assumptions

6. Conclusions
7. Recommendations
8. Limiting Conditions
9. Warranties

Recommendations for remedial measures should include a clear description of what the remedial measures are intended to accomplish. Perfection is not attainable by remedial measures. Recommendations for remedial measures should identify important or significant limitations of the measures, and should comment on reasonable expectations of the remedial measures. Design of remedial measures should be based upon generally accepted engineering practice. If proposed remediation involves installation/construction of repair components, the report should include applicable engineering calculations and site-specific plans and specifications to facilitate installation/construction of the components in accordance with the engineer's design. At a minimum, the plans and specifications should include:

1. The site address
2. The engineer's name and the firm's name, address, and telephone number
3. The client's name and address
4. The purpose and limitations of the repair components
5. Available geotechnical information and source
6. A plan view of the existing foundation locating known relevant structural components
7. Details to show how to construct repair components
8. Specifications to identify appropriate materials and methods
9. Requirements for construction observation or testing by the engineer or others
10. Existing floor elevation information, if applicable
11. Post-repair floor elevation survey requirements, if applicable
12. Site restoration requirements

Installation/construction of repair components should be observed and documented to ensure that the components are installed/constructed in accordance with the design. The engineer should *approve any design modifications*. The engineer or a qualified delegate should perform observation and documentation of installation/construction of repair components. The qualified delegate should be a staff member under the engineer's direct supervision, or an outside agent approved by the engineer. The observation reports should be provided to the engineer. Upon completion of installation/construction of repair components, the engineer should *issue a compliance letter* to the homeowner indicating that installation/construction of the repair components was in conformance with the engineer's plans and specifications including any modifications or alterations authorized.

6.9 HOMEOWNER INSPECTION

Homeowner Inspection and Maintenance

The expert on daily and seasonal behavior of a house is generally the homeowner. A homeowner inspection is probably the most important and economical assessment tool, and it is certainly the first step in determining distress or unusual behavior. Forensic and anecdotal evidence provided by homeowners during house inspections was very useful for the Corps inspection team and is a key component of foundation damage evaluations.

We subdivided this discussion into *Basic Inspection* and *Basic Maintenance*. The *Basic Inspection* takes about an hour, is observational, and can generally be done by most homeowners without any specialized training. The *Basic Maintenance* can generally be performed by the "do-it-yourselfer" who routinely performs home maintenance and landscaping.

Basic Inspection

Every homeowner whose house is located North of Main Street should consider, at a minimum, a bi-annual walk-around inspection of the house exterior and interior during late spring and late fall. On the exterior walk around you should:

- Walk the perimeter of your house (safety permitting) and note any locations and sources of ponded water near your basement/foundation walls. Determine the source of standing water (snowmelt, disconnected downspouts, gutters, sump pump outfall, surface runoff from adjacent properties, etc). Note other low areas in yard and their proximity to basement walls.

- Note the slope of soils near basement/foundation walls and whether they have settled.

- Note new fractures and movement (direction and displacement) of basement/foundation walls, driveway slabs, porches, steps, etc. (use a permanent marker to make a reference mark). When do the gaps widen and close? Has the movement stopped?

- Ensure downspouts are properly connected to underground piping. Check the flow of downspouts during a rain/melting event. Follow the flow to the curbside bubbler and see if the water is discharging into the storm sewer/ditch. Does flow back up anywhere?

On the interior of the basement perform/observe the following:

- First, you may want to sketch of your basement (use your blueprint as a guide).

- Visually inspect and note the condition of interior basement wall surfaces (sometimes walls are covered with materials). Note bowing and all cracks along with their orientation, length, width, and any relative movement along the cracks (when you drag your hand across the crack, which side sticks out, and is the crack wider at the top or bottom). Again, mark reference points directly on the wall.

- Note evidence of water build-up behind the wall, such as leaking, dampness, discoloration, salts, and other staining.

- Inspect the basement slab for cracking and sloping, especially near foundation walls (older cracks often are filled with floor dust). Note location of water leaks if present.

- Ensure that your sump pump is in good working order. Note the time between pump cycles during wet and dry periods and observe the flow into the sump pit (is inflow from one or both pipes). With a flashlight, observe any sediment/roots in the base of sump pit. Note frequency of sump pump replacement.

- Periodically inspect the house during unusual events such droughts, floods, significant rainfall, construction, tree removal, etc.

In short, be a curious homeowner and record your observations. This information is useful for diagnosing specific causes of foundation damage.

Basic Maintenance

Common basic maintenance steps include the following:

- Promote positive drainage away from basement/foundation walls. Landscape the soil near your basement/foundation walls to slope away from the wall. Ponded water near basement/foundation walls can promote foundation damage.

- Additional fill can be brought in to replace settling fill. One reference suggests a minimum of 5% slope away from the home for the first ten feet around the foundation walls (USACE, 1983). A minimum 2% slope should be established for lawn areas greater than ten feet away from the home. These “rules-of-thumb” may not be feasible for some lots, and homeowners should contact their local building department for additional guidance.

- When surface drainage cannot be improved by grading, subsurface water drains can be used to control surface water runoff. The minimum slope of the pipe should be 0.5 percent (approximately 0.6 inches per ten feet) toward a surface outfall. Homeowners should contact their local building department for additional guidance.

- Uncontrolled roof runoff from downspouts can cause erosion and ponding of water near the structure. Downspouts should be extended well past the edge of the foundation and past the edge of abutting planting beds or into well-drained areas.

- Trees or large shrubs near a foundation may cause soil shrinkage near the foundation. Tree removal can, however, have adverse effects such as soil heave. Reasonable pruning is used to control soil moisture content for shallow footings in England (Freeman et al., 1994).