TOWN OF AMHERST
SOILS AND RESIDENTIAL FOUNDATION STUDY

Prepared For:

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EXECUTIVE SUMMARY

The U.S. Army Corps of Engineers and the Town of Amherst conducted a one-year cooperative investigation of residential foundation damages in Amherst, New York (2004-05). The study purpose was to determine (1) the extent and scope, (2) causative factors, and (3) provide recommendations to Town officials and homeowners. The study methods included a literature review, house inspections (43 sites), soil sampling (32 sites), field inspections (bimonthly), and a phone survey (52 homeowners).

Nearly 1,100 foundation repair permits and foundation inquiries have been received by the Town since 1987. Seventy-five percent of the permits and inquiries are located north of Main Street on lowlands with fine-grained lacustrine (geologic lake) soils. The town-wide foundation damage rate on lacustrine soils is about 3 percent, but in several affected areas the rate is an order of magnitude greater. The cost of some foundation-related repairs exceeded $100,000, but most homeowners have spent less than $20,000.

The damages generally result from lateral pressures and/or differential settlement. Lateral pressures are caused by soil weight, frost, hydrostatic pressure, and shrink/swell. The backfill and the underlying foundation soils are classified as moderate to highly expansive and undergo volumetric change as their moisture content varies. A non-uniform change in soil moisture content across the foundation footprint is a primary causative factor for differential settlement. A second primary causative factor for differential settlement involves the soft stratum that underlies the stiff stratum, where many residential footings are placed. This soft clay stratum is susceptible to consolidation with a drop in groundwater elevation and/or the addition of perimeter fill material. In addition to problematic soil conditions, foundation inspections revealed that houses were designed and constructed without fully considering lateral pressures and potential settlement.

The primary recommendation to the Town is to develop and adopt new guidelines for the design/ construction and assessment/repair of residential foundations to supplement existing building codes. Recommendations for homeowners include conducting an annual foundation inspection and retaining a licensed, qualified engineer when appropriate.

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1.1 PROBLEM STATEMENT

Homeowner reports of foundation-related problems and structural damage in Amherst, New York, increased in the late 1990s and peaked in 2003. Of the estimated 31,000 residential structures with basements, about 1,095 homeowners have contacted the Town of Amherst (Town) to request a foundation repair permit, make a foundation inquiry, or have their property value assessments lowered because of foundation-related damages. However, many homeowners are concerned about stigmatizing their neighborhood or adversely affecting their property value, thus the actual number of repaired and damaged houses, their age, and repair cost is somewhat uncertain and disputed.

The majority of damaged houses are located north of Main Street, in the lowland, and on fine-grained lacustrine soils. The geographic pattern and severity of the damage is irregular, akin to earthquake damage, and can affect none, one, or a cluster of houses in close proximity.

Damage symptoms commonly include cracked and bowed basement walls and slabs, and/or uneven settlement across the foundation. Lateral pressures and differential settlement have been recognized as causative factors. However, there is much speculation about specific hydrologic, geotechnical, and structural factors behind these causes.

Most homeowners with damages are seeking simple, immediate, and economical solutions and, in some cases, restitution and adjudication. To be sure, the financial and emotional distress for some homeowners is substantial. Property owners without damages are seeking unambiguous advice about preventative maintenance, monitoring, and even predictions about the future occurrence and location of problems. Town officials seek simple engineering remedies that translate into policy and/or ordinances. Practicing engineers seek new data for design and repair of houses. Builders and contractors seek confirmation of their methods and consistent planning and policy-making.

These expectations, however well intentioned, must be subordinated to an initial investigation into the (1) scope and extent of the problem and (2) its causative factors. This preliminary, one-year, cooperative investigation between the U.S. Army Corps of Engineers (Corps) and the Town, represents this first step toward developing a basic foundation of understanding. These findings will be applied by engineers and Town officials and assist homeowners and building professionals in the future design/construction and evaluation/repair of Amherst houses. This study, then, is the beginning of a process rather than the end, and additional work will be needed to verify and advance these preliminary findings.

1.2 STUDY OBJECTIVES

The study objectives were negotiated in a Letter of Agreement and the Project Management Plan (PMP, 2004) between the Town and Corps and include the following:

- Better define the <u>extent</u> and <u>scope</u> of the foundation-related damages;
- Determine potential <u>causative factors;</u>
- Provide <u>recommendations</u> to the Town and homeowners regarding new construction and existing residential structures.

1.3 APPROACH

The study approach is a synthesis of existing literature and local investigations. The literature sources include Town databases, government and consultant reports, and peer-reviewed articles. The local investigations include a phone survey, home inspections, and field inspections. Four Town departments, the United States Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS), and the geology department at the State University of New York at Buffalo (University at Buffalo or UB) contributed to this report.

1.4 LITERATURE REVIEW

This literature review is an introductory bibliography for homeowners, Town officials and building professionals. The topics are wide ranging and reference general to technical publications.

There are three excellent guides for homeowners or homebuyers: (1) *Has your House got Cracks* (Freeman et al.,1994); (2) *A Guide to Swelling Soils for Colorado Homebuyers and Homeowners* (Noe et al., 1997), and (3) *So Your Home is Built on Expansive Soils* (ASCE, 1995). In addition, online guidance from Virginia (DPWES, 2002) and Canada (CHMC, 2004) is informative. It must be stressed that all of these guides reflect their geographic region and climate; therefore, some problems and recommendations are not applicable to Amherst.

Erie County's soils have been mapped and their characteristics described by the National Resource Conservation Service (NRCS, formerly Soil Conservation Service, or SCS) since the early 1970's (USDA, 1986). An excellent summary entitled, *Soil Inventory and Interpretive Study for Town of Amherst, Erie County, New York*, was prepared for the Town regarding the suitability of soils for such uses as foundations, utilities, streets, etc. (ECSWCD, 1972).

Regional hydrogeology and hydrology is the focus of several government reports. A comprehensive analysis of groundwater resources in the Erie-Niagara basin was written by La Sala (1968). Bedrock aquifers in Erie and Niagara County have been described by the U.S. Geological Survey (Kappel and Miller, 1996; Miller and Staubitz, 1985; Staubitz and Miller, 1987). The Corps did an extensive geotechnical investigation along Ellicott Creek for a flood control project (USACE, 1979).

Several reports present geologic and geotechnical data about the unconsolidated deposits in Amherst or the surrounding region. For example, Ward (1971, 1973) conducted sequential geotechnical investigations of soils at the District 16 Sewage Treatment Plant, located in northwest Amherst. Hodge et al. (1973) used geophysical surveys to describe the geologic and hydrogeologic zones surrounding the University at Buffalo (north campus). The Corps (USACE, 1979) developed a geologic cross section from about 35 boreholes along the 1.7 mile corridor of Ellicott Creek, between Niagara Falls and Maple Road. Two technical papers describe the extensive subsoil investigation and embankment instability along the Lockport Expressway (McGuffey et al., 1981; Kyfor and Gemme, 1994). The Corps (USACE, 1973) investigated subsoil conditions in North Tonawanda (also glacio-lacustrine sediments) as part of the Lake Erie-Lake Ontario Waterway project.

Clays are associated with the subsidence of many major cities around the world (Waltham, 2002). Expansive soils have been mapped in many parts of the world and the United States (FHA, 1975). Classification of expansive soils is reviewed by Sridharan and Prakash (2000). Meehan and Karp (1994) summarize 30 years of lessons learned with expansive soils and housing damage in California; they recommend, that buildings on expansive soils must be engineered, a comprehensive pre-building design and geotechnical investigation of the site is needed, and the construction should be performed with architect's observation and engineer's inspection. Simons (1991) describes damage caused by expansive soils as probably the least publicized of natural hazards, but it ties with hurricanes for second place amongst economic loss to buildings. Simons summarizes that due to the uniqueness of each type of structure, one set of assumptions, design criteria and subsequent repairs, cannot provide a solution for every problem encountered. Numerous authors cite the Jones and Holtz (1973) article entitled, *Expansive soils – the hidden disaster*. A Corps technical manual for engineers entitled, *Foundations in Expansive Soils*, is published online (USDOA, 1983). The Texas Section of the American Society of Civil Engineers (Texas ASCE) promulgated two useful expansive soils-related guidelines entitled, *Recommended Practice for the Design of Residential Foundations* and *Guidelines for the Evaluation and Repair of Residential Foundations* (Texas ASCE, 2002, 2002a).

Numerous articles and reports describe the causes/damages of settlement and lateral pressures on residential structures. Wahls (1981) reviews the current concepts and practices for establishing tolerable settlements for buildings. Settlement caused by groundwater withdrawal is discussed by Preene (2000). Damage caused by tree root extraction of moisture is discussed by Day (1992), Silvestri and Tabib (1994), Vipulanandan et al. (2001). Methods of engineering for settlement cases, often on marginal lands, is described by Adid and Paratore (1994), Moore and Chryssafopoulos (1972), and Whitlock and Moosa (1996). Diaz et al. (1994) describes basement failure as one of the most common problems in residential buildings in Ohio.

Anumba and Scott (2001) described a knowledge-based system intended to provide guidance for engineers dealing with subsidence cases. Their system covers diagnosis to remedial measures and was in response to the large number of buildings in

England being subjected to remedial underpinning (foundation damage in the UK is covered by insurance); they concluded the underpinning was not always justified.

Alternative methods of design and construction are described by Senapathy et al. (2000) and Sealy and Brandimere (1987).

Literature on the geotechnical and mineralogical properties of various clays is discussed in Murray and Quirk (1980) and Ghably (1998). Lytton (1994) presents an in-depth discussion of the mechanics and theory of moisture-related volume change in expansive soils.

1.5 SITE DESCRIPTION

1.5.1 General

The town of Amherst is located northeast of the city of Buffalo in extreme western New York State, USA (Figure 1). The Town incorporates 54 square miles and had a population of 116,510 in the 2000 census (Amherst IDA, 2005). Amherst includes the village of Williamsville with a population of 5,573. Approximately 45 percent of the land area is developed in residential uses. In January 2000, there were an estimated 33,000 single-family residences and 14,000 multi-family residences. Approximately 13% of the land area is currently designated for recreation and open-spaces. Most of the town is sewered, with the exception of 43,000 acres in the northern area near Millersport Highway and I-990. Where sewer is not available, soil conditions generally constrain development and limit residential development to one unit per ¾ acre. Nearly 50 percent of Amherst is in the 500-year flood plain, with approximately 24 percent in the regulated 100-year floodplain.

1.5.2 Physiography/Topography

Erie County is located in the western portion of the Erie-Ontario physiographic province of New York, which is in the northeastern portion of the Central Lowlands physiographic province of the Interior Plains physiographic division (USACE, 1979). The region is bordered on the north by Lake Ontario, on the west by Lake Erie and the Niagara River, and on the south by the Allegheny Plateau. Within the region are three plains, Ontario, Huron, and Erie, separated by the east-west striking Niagara, Onondaga, and Portage escarpments. Amherst is located within the Salina Lowland of the east-west trending Huron plain. This lowland area is bounded by the Onondaga and Niagara Escarpments, which are composed of more resistant rock.

A digital elevation model (DEM) of the town shows the major topographic features in Amherst (Figure 2). Topographic relief in Amherst is due to pre-glacial erosion of the bedrock and subsequent topographic modification by glaciation (La Sala, 1968). The Town generally slopes north-northwest, which promotes surface and subsurface drainage toward Tonawanda Creek and the Niagara River. Between the major drainages of Ellicott and Ransom Creeks, the topography is nearly flat, with Tonawanda Creek dropping only three feet per mile across northern Amherst.

1.5.3 Geologic History

Lake Tonawanda is a remnant of Lake Dana-Lundy resulting from the final northward retreat of the Ontario ice lobe of the Wisconsin glacial ice sheet when the ice front was located between the Niagara escarpment and Lake Ontario (section after D'Agostino, 1958). The lobes position near the Niagara escarpment prevented the northward drainage of Lake Dana-Lundy, forcing the lake to drain eastward through the Marcellus Spillway. However, when the ice lobe had retreated further northward toward the present Lake Ontario, lower outlets of eastern Lake Dana-Lundy, now uncovered by glacial ice, caused a rapid lowering of the lake level (Figure 3). When the level of Lake Dana-Lundy was lowered to the elevation of the Niagara Escarpment, the water sought various drainage outlets over the Niagara Escarpment. These drainage outlets terminated the existence of Lake Dana-Lundy and brought into existence Lake Ontario, Lake Erie, and the large river-lake called Lake Tonawanda. From the descriptions provided by D'Agostino (1958), it is the older lacustrine deposits, more so than the Lake Tonawanda deposits, that has cohesive and varved clays (Photo 1). More detailed accounts about the evolution of Lake Tonawanda are found in NYSGA (1966), NYSGA (1982), and D'Agostino (1958).

1.5.4 Bedrock Geology

The Erie-Niagara basin is underlain by bedrock that is largely covered by unconsolidated deposits (section after La Sala, 1968). The bedrock consists mainly of shale, limestone, and dolomite. All bedrock units were built up from fine-grained sediments deposited in ancient seas during the Silurian and Devonian periods and, therefore, are bedded or layered. The dip of the rocks (inclination of the bedding planes) is gently southward at 20 to 60 feet per mile, but the dip is so gentle that it is hardly perceptible in outcrops.

The Camillus Shale underlies most of Amherst (Figure 4). The Camillus contains a large amount of interbedded gypsum with beds up to 5 feet thick, but most gypsum occurs as thin lenses and veins. Underground gypsum mining in east Amherst is described briefly below. Groundwater in shale is likely to follow fractures and openings formed by dissolution of gypsum. Groundwater that enters the formation discharges mainly to Tonawanda Creek (La Sala, 1968). In Clarence, Staubitz and Miller (1987) describe the top 3 to 10 feet of the Camillus as moderately weathered and fractured, and, where overlain by dolomite, extremely fractured and weathered, such that it resembles coarse gravel. Boring logs reviewed in this study and reports by the Corps (USACE, 1979) and Kappel and Miller (1996) also suggest the upper surface can be weathered and have degraded rock quality.

Further south but stratigraphically above the shale is the "limestone unit" that includes the Bertie Limestone at the base, the Akron Dolomite, and the Onondaga Limestone at the top (La Sala, 1968; Kappel and Miller, 1996). Both limestone and gypsifferous shale can be dissolved by circulating groundwater, thus, dissolution of rock may be a causative factor for settling houses on or near the escarpment.

1.5.4.1 Gypsum Mining

Underground gypsum mining in Amherst occurred beneath a one mile-square area near the southeastern boundary of the Town, approximately between Klein, Maple, Ayer and Transit roads. As development of that area progressed, the Town acquired additional information that further defined the mining area. The refined information was based upon borings into bedrock and electrical resistivity analysis. The vast majority of the mined-out area is now known to be located east of Covent Garden Lane, south of Renaissance Drive, north of Maple Road, and west of Transit Road.

The mining operation commenced in 1925 and ended in 1976. The impure gypsum was removed (mined) from a layer approximately 70 feet below ground level. The bedrock is overlain by 10 to 40 feet of unconsolidated glacial deposits. The layer of gypsum varied in thickness but was typically 36 to 42 inches in height. The room and pillar method used in mining leaves intact pillars of bedrock within excavated "rooms" of gypsum and rock. According to various reports, the size of the pillars ranged from 10' by 10' to 24' by 24'. Measurements of the pillar spacing have been described as 33 feet from center to center with a roof span of approximately 24 feet. The haulage ways were excavated to a height of 6 feet with a width of 18 feet. Main haulage ways may have had some spans of 40 feet and a height of 8 feet.

With the cessation of mining in 1976, the dewatering operation (pumping) discontinued and the water table returned to its natural level. While in operation the pumping rate was estimated to be 500 to 1,000 gallons per minute (gpm, or 1.44 million gallons per day) during the summer and up to 5,000 gpm during the spring.

Details of the mining operation can be found in various reports contained in the Town's rezoning files associated with the DRT, Tesmer, and Cimato parcels. The master file for these reports is obtainable from the Planning Department. It is the policy of the Town Building Department to assume that all properties in the Maple-Ayer-Klein-Transit roads area are located above a mine unless determined otherwise through appropriate engineering or scientific analysis.

1.5.5 Surficial Geology

The overlying unconsolidated deposits are mostly glacial or glacial lacustrine in origin and were formed during the Pleistocene time about 10,000 to 15,000 years ago, when the ice sheet covered the region (section after La Sala, 1968). The glacial deposits consist of: (1) till, which is a non-sorted mixture of clay, silt, and, stones deposited directly from the ice sheet; (2) lake deposits, which are bedded clay, silt, and sand that settled out in lakes fed by the melting ice; and (3) sand and gravel deposits, which were laid down in glacial streams. In Amherst, the typical thickness of unconsolidated material is about 40 feet but ranges from less than one to more than 70 feet. The lacustrine or lake deposits are a primary focus in this report and will be discussed below. Other unconsolidated deposits are alluvium formed by streams in recent times and swamp deposits created by the accumulation of decayed plant matter in poorly drained areas.

The surficial geology map of western New York shows lacustrine deposits in northern Amherst and till in the south (Figure 4). This mapping scale does not adequately identify lacustrine deposits in the south-central portions of Amherst (see below).

Appendix 6.1 shows a geologic cross section prepared by the Corps for the Ellicott Creek Flood Control Project (USACE, 1979). The section illustrates several characteristics about the overburden which are consistent with our findings. The cross section is composed of several segments that parallel Ellicott Creek from Niagara Falls Boulevard (Plate D5) to Maple Road (Plate D6). The dominant geologic units in descending stratigraphic order are: (1) Lacustrine (Qlt) – stratified, sorted, sandy silt and sandy clay of low plasticity, associated with Lake Tonawanda; (2) Lacustrine (Ql) – well sorted, thin bedded to massive, red-brown to gray clayey silt of high plasticity, associated with proglacial Great Lakes, and (3) Glacial Till (Qt) – compact, non-stratified, red-brown to gray, pebble, clay silt till of low plasticity. The cross section suggests:

- The overburden thickness above the shale ranges from 10 to 75 feet;
- The northern section (1) has a "layer-cake" stratigraphy;
- The southern section (2) shows lacustrine deposits lapping onto till;
- All three units (Qlt, Ql and Qt) are exposed at the ground surface;
- Qlt is more heterogeneous (SM, SC, ML, CL, OL, CH) than Ql (CH, CL);
- Stratified gravel deposits occasionally overlie or are embedded in the till.

1.5.6 Soils

Soil texture is an expression of the proportion of sand, silt and clay in the soil. Common regional descriptions of soils as "clay" are often simplifications or misinterpretations of the true soil texture. South of the escarpment, soils are mainly silt loam in texture, meaning the soils consist of roughly equal proportions of sand, silt and clay within the surface layer or horizon. North of the escarpment, while often described as "clay," surface textures would more accurately be described as silty clay or silty clay loam, although there are still large areas of silt loam and smaller pockets and bands of sandy loam and other textural groups.

Silt (0.074 to 0.002 mm) and clay (less than 0.002 mm) are described as "fine particles" (meaning smaller). Sand-size particles represent a range of particle sizes from fine sand (0.25 mm to 0.1 mm) to very course sand (2.0 to 1.0 mm). The properties and characteristics of soil are heavily influenced by soil texture. Drainage, permeability, infiltration and percolation, rooting depth, moisture holding capacity, and seasonal high water table or zone of saturation are some of the properties most influenced by soil texture. Textural variation with the soil profile can be significant. Particularly in lacustrine soils, it is not uncommon to find layers of fine to coarse sand bounded by silty clay or clay layers.

Deeper horizons often do not have the same texture as the surface layer. Below the surface soils, including the formal subsoil or "B" horizon, the deep, unconsolidated subsoil can be similar to or may vary significantly from the overlying surface soil. In this region, subsoil may be associated with older geological events and time periods. This can result in some unexpected conditions with somewhat unpredictable consequences when relying solely on mapped soil data for planning and engineering decisions. The inaccuracy of National Cooperative Soil Survey maps has been evaluated in several studies (see Brevik et al., 2003).

1.5.6.1 Regional Soils

The Soil Survey of Erie County, New York, and prior published soil maps describe the surface soils generally to a depth of 60 inches (USDA, 1986). The characteristics of the deep subsoil can sometimes be inferred from the mapped soil data but, typically, *on-site site investigations* would be required to properly characterize the unmapped subsoil. Surface geology maps are not detailed enough for site-specific evaluations.

Soil properties are an important planning, design and engineering consideration. The fine-grained lacustrine soils which dominate the landscape of the study area are recognized has having serious limitations for a variety of engineering activities and land uses (see ECSWCD, 1972; USDA, 1986). Agricultural uses are limited because of generally poor drainage characteristics without artificial drainage, usually surface drainage. Large areas of hydric soils, often indicative of historic or current wetlands, are common. Seasonal high water tables (zones of saturation) are recognized as serious limitations. High potential frost action, low permeability (except in sandy soils and sandy layers within lacustrine deposits), high plasticity indices and high liquid limits are common limitations for most urban uses of these soils. North of the escarpment, slow permeability combines with the flat slopes to contribute to ponding and localized drainage problems. These problems are exacerbated in areas prone to localized or regional flooding.

The Soil Survey describes the stratified, fine-grained deposits common throughout the study area as "difficult to use for engineering works" and suggests that "sites proposed for embankments and heavy structures or buildings must be investigated for soil strength, settlement characteristics, and the effects of ground water" (USDA, 1986). The same section of the Soil Survey reads:

> Because of their fine texture and high moisture content, these deposits have relatively low strength. They are generally highly compressible and tend to settle over long periods.

Other deposits including stratified, coarse-grained deposits formed in lacustrine sands, shallow-to-rock deposits along the escarpment and small areas of organic deposits, occur in the study area. Coarse-grained materials generally have high strength but may settle when vibrated (USDA, 1986). Long-term settlement is also of concern if organic soils are filled over.

1.5.6.2 Amherst Soils

The Onondaga Escarpment, which parallels Route 5 through the Town, marks the approximate boundary between surface soils which are predominantly lacustrine in origin (to the north) and predominantly glacial till soils (south). Soils are more typically shallow to bedrock along and just south of the escarpment. North of the escarpment, soils are generally deeper, with depth to bedrock greater than 10 to 20 feet in most areas.

There are approximately 55 mapped soil units within the town of Amherst (ECSWCD, 1972). Five soil units are described as fine-grained lacustrine soils and include Cheektowaga, Cosad, Lakemont, Niagara, and Odessa. These soils cover about 42 percent of Amherst and account for 48 percent of the foundations (Figure 5). These cohesive soils generally show high porosities, low permeabilities, and a natural moisture content associated with low strength, low bearing capacity, and high settlement characteristics. Often increasing values of moisture content tend to be associated with decreasingly favorable foundation conditions (Watson and Burnett, 1995).

1.5.6.3 Soil Boring Data

To investigate subsurface conditions across Amherst, data from 371 boring logs were entered into the Town's Geographic Information System (GIS). A boring log is a geotechnical/geologic description of the subsurface materials encountered by a driller, and a GIS is specialized computer system capable of analyzing and displaying layers of spatial data. The majority of the boring logs came from recent building department permits, but about one-third are from the installation of intercepting sewers in the early 1970s.

The primary purpose for analyzing these data is to determine the extent and depth of an exceptionally soft silty clay layer, locally described as the "peanut butter" or "gumbo" layer (Dolan, 2004). Similar soils have been implicated in bank failures along creeks, ponds, and roadways in other parts of Erie County.

The data entered into the GIS included surface elevation (if available) and the depths (below ground) to the bottom of fill, top of soft layer (if present), bottom of soft layer (if present), bottom of bore hole, reason for termination (end-of-bore, refusal, rock), and groundwater depth. The consistency of the soft layer was classified as "*soft*" if the N-value (sum of middle blow counts) was less than 4, or "*semi-soft*" if the N-value was greater than or equal to 4 but less than 8. Some boreholes did not encounter a soft stratum (N = 8) and are termed "*not soft*." The soft stratum of interest consists of silty clay, however, other soft horizons were sometimes present elsewhere in the profile (e.g. organics or wet sand). The water level measurement from the open borehole was entered for the longest interval after borehole completion, which ranged from 0 to 72 hours.

Figure 6 shows the location of borings and whether a soft, semi-soft, or not soft horizon was encountered. The soft stratum is present in most boreholes in central and northern Amherst. However, semi-soft and not soft strata are also found in these areas. Conversely, the subdivision near Transit Road and Maple Road is underlain by till that is mostly not soft, except for one boring that has soft strata. Figure 6 illustrates the heterogeneity in a soft stratum area at the parcel-scale (see inset). These data suggest that in most areas (1) site specific borings are generally needed to determine subsurface conditions, but (2) in some areas, that requirement could be excessive.

1.5.6.4 Geotechnical/Geologic Cross Section

Figure 7 is geotechnical/geologic cross section across central Amherst that follows the 5.8 mile former New York Central Railroad, known locally as the "Peanut Line." The section shows the depth to the soft (lighter/red stippling), semi-soft (darker/blue stippling), or not soft stratum. (The geotechnical significance of the soft stratum is discussed in Section 3.4.3.2). These data were collected in May to June, 1973, from 61 equally spaced soil borings (record 55 missing). The section line is located entirely within lacustrine surficial geologic units (Figure 4). Figure 7 also illustrates (and exaggerates) the extremely shallow grade from east to west (0.06%).

Most borings intersect the soft (78%) or semi-soft (12%) stratum. The soft stratum appears to gradually pinch out toward the east, which may explain, in part, the relatively low occurrence of foundation-related damages in neighboring Clarence. The surface of the bedrock when intercepted varies from 571 to 545 feet above mean sea level (AMSL). Figure 7 suggests that bedrock-controlled topographically higher areas have stiffer soils, perhaps because micro-topography influences runoff, ponding, infiltration, and groundwater hydrology.

Figure 8 shows a generalized stratigraphic/soil profile for central and northern Amherst. The generally coarser sandy silt soil transitions downward to a moderately stiff silty clay that grades to a plastic soft clay (USCS classification CL/CH). The clay consistency decreases from stiff/hard to soft/very soft. The depth of the transition to soft clay varies from 3 to 35 feet across Amherst, but along the Peanut Line it averages 12.3 ± 2.0 (1s) feet below the ground surface. The transition is marked by a gradual to sudden drop in blow counts (sometimes weight of rods), an increase in natural water content (Figure 9), and a general increase in plasticity of the clay. Above the transition is the stiff stratum and below the soft stratum. Many residential footings rest on this stiff stratum, that is, the foundation footings are only a few feet above the soft stratum. In other locations, the footings rest directly on the soft stratum or till (see Appendix 6.1). The upper till boundary sometimes can include dense, wet, compacted fine sand with a little silt or coarse gravel. The dense glacial till rests on shale.

1.5.6.5 Expansive Soils

Lacustrine soils in Amherst are moderate to highly expansive (Section 3.2.3). This section provides some general information about expansive soils. Swelling or expansive soils are found in 40 of 50 United States and in all the world's continents except the polar ones (Steinberg, 1999). The first conference on expansive soils was held at Texas A&M in 1965. The need for proper construction of buildings on expansive soil was identified at least 35 years ago, was mandated by the State of California, and is required by the UBC (Meehan and Karp, 1994). Studies spanning decades have determined that the change in swelling soils' moisture content results in damaging volumetric changes. These soils are described in relation to the prevailing climate, that is, in arid climates they are known as "swelling soils" and "heaving soils," and in the temperate climate of the United Kingdom, these soils are known as "shrinkable" soils (ASCE, 1995; Freeman et al., 1994). Previous predictive mapping of expansive soils did not recognize the lacustrine deposits in western New York as having expansive soils (Figure 10).

Most clay soils swell, to varying degrees, with increased moisture and shrink with drying. There are many factors that control how much a soil can swell, including the type of and concentration of minerals, soil density, the capacity for moisture change, and the restraining pressure of the surrounding soil (Noe, 1997). The degree of shrink/swell is often related to its clay mineralogy. Kaolinite, illite, and smectite are the most common clay minerals. In rain-soaked western New York, the initial concern is for removing soil moisture. Desiccation of clay soils causes them to be hard and cracked (Photo 2).

Two commonly used indexes to characterize an expansive soil are the plasticity index (PI) and the expansion index (EI). PI is a geotechnical engineering term that is the difference between the soil's plastic limit and liquid limit, two common soil tests performed in a laboratory. If the soil's PI is between 20 and 40, the soil is considered to have moderate expansive properties (see Freeman et al., 1994), although Sridharan and Prakash (2000) suggest PI and related properties cannot satisfactorily predict a soil's expansivity. A soil with an EI of 50 or less is considered to have low expansion potential, moderate potential between 51 to 90, and an EI of 91 or greater indicates a soil with high or very high (> 121) potential.

Expansion Index testing has been required by the Amherst Building Department since January 2003. We reviewed approximately 15 Amherst projects with geotechnical reports (in 2004) that had EI values considered moderate to high. One local laboratory that has performed 75 or more EI tests reports that nearly all Amherst soils have been in the range from 60 to 120 (pers. Comm., Jeanne Asquith, 3rd Rock LLC, 2004).

Prior to EI testing, McGuffey et al. (1981) showed the average PI of 66 samples along the Lockport Expressway was 22.2 ± 3.0 (1σ). The Corps (USACE, 1979) had numerous samples along Ellicott Creek with an average PI ranging from 26.3 to 29.8 (Table 1). The Corps (USACE, 1973) collected 15 clayey soil samples from two boreholes north of Tonawanda Creek, but within the lacustrine sediments, that yielded PI's of 22.0 ± 3.3 and 24.4 ± 3.3 (Table 1). Ward (1973) soil samples form north Amherst had a PI of 24.1 ± 4.3.

1.5.6.5.1 Soil Moisture Variation

House and field inspections reveal that a fairly constant but large number of factors at various scales, both natural and man-made, are potentially affecting the soil moisture conditions in the active soil zone around a typical Amherst foundation (Table 2). Many man-made factors often result from the conversion from undeveloped to developed land (suburbanization). The net effect of these factors on soil moisture can be determined with careful measurement but is not easily anticipated by homeowners. The term “active zone” has taken on several different meanings over the past two or three decades (Nelson et al., 2001). In this study, it refers to the zone of soil that is contributing to heave and settlement at any particular time and includes material below the elevation of the foundation footing, not just simply material surrounding the basement.

1.5.6.5.2 Quantitative Mineralogy

The Corps contracted the University of Buffalo’s Geology Department to investigate the amount and nature of soil minerals, particularly clays, present in the samples gathered for geotechnical analysis. All samples were analyzed using X-ray diffraction and quantitative mineralogical analysis software (Giese and Juul, 2005). When analyzed, specially prepared samples reveal characteristic diffraction patterns that are matched with an internal standard to identify and quantify the minerals in the sample.

Appendix 6.2 shows the results for three sample strata -- backfill, stiff stratum, and upper soft stratum (see Figure 8). Two samples of till (Corps No. 13 and 27) are not used in the statistical summaries because they differ significantly from the lacustrine samples. The total of clay minerals average 31.8%, 35.9% and 36.6% by weight for the three sample stratum, respectively (i.e., the clay content increases with depth). The dominant minerals are illite and quartz, followed by calcite, chlorite (a clay mineral), and feldspar.

The dominance of illite (with chlorite), both non-swelling clays, suggests the mechanism of soil swell is *not* the classically understood interlayer swelling, which occurs with smectite clays. Interlayer swelling is the process where water enters directly into the clay structure and can expand the mineral volume by 100% or more. Smectite clays are found in western states with well known expansive soil problems. In Amherst, the lacustrine soils are swelling by another mechanism, perhaps intra-layer swelling and/or by a process involving an organic coating on quartz and other mineral grains. This conclusion is interesting and not simply academic because potential cutting edge remedial options involving soil amendments will be predicated on our understanding of this swelling behavior.

1.5.7 Hydrology

Amherst hydrography is shown in Figure 11. Ellicott Creek is the largest tributary of Tonawanda Creek. Ellicott Creek drops precipitously some 60 feet in Williamsville, then flows northwesterly before discharging into the channelized section of Tonawanda Creek at an elevation of approximately 564 feet (USACE, 1979). The slope in the flatlands is about two feet per mile. Before the Ellicott Creek flood control project, peak discharge during flood events was considerably less downstream at Niagara Falls Boulevard than upstream in Williamsville, indicating abundant overbank storage. The Corps’ project was designed to keep flow in the channel and lower the 100-year flood stage by an average 1.5 feet between Maple Road and Niagara Falls Boulevard. It is plausible that areas near the creek receive less groundwater recharge as a result of the project.

Amherst's growth and development (Section 1.6.1) converted farmland to residential subdivisions. Development generally alters the hydrologic budget of an area and leads to less infiltration and more surface water runoff (NJDEP, 1999). Downspout collection systems, yard drainage, footing drain tiles, sump pumps, maturing trees, positively sloped yards, and impervious roofs, walks, patios, and driveways route water away from a parcel and reduce recharge to the water table (see Table 2). A typical ½-acre house lot, for example, has 25 percent impervious cover. Groundwater and diverted surface water runoff enters Amherst's stormwater system comprised of underground storm sewer pipes, ditches, retention ponds, and dry wells. Sometimes this drying trend is offset by landscaping, leaky plumbing (e.g., sprinkler, sewer, water, pool), snow storage and retention ponds. In either case, net soil moisture changes can be incremental and may take several years. We inspected some houses whose settlement might be caused or aggravated by long-term localized desiccation.

The New York State Department of Environmental Conservation (NYSDEC) has identified approximately 1,565 acres of regulated wetlands in Amherst, and an additional 250 acres of wetlands are protected by Federal wetland protection. Wetlands provide many important functions, not the least of which is temporary storage for flood waters.

Consulting engineers sometimes provide water balances with proposals for residential development. For comparison, an estimated average water budget for nearby Ohio is provided in Appendix 6.3. Calculating an actual water balance at the house-lot scale, however, is challenging because many of the inflows and outflows listed above are simply unknown or have significant temporal and spatial variability (e.g., canopy cover, infiltration rate, sump pump withdrawal). In addition, homeowners landscape to route surface water from their yards in what we termed as Amherst's "topography war." Newer subdivisions are often elevated and flow into older neighborhoods. In short, careful site inspection and measurement may be needed to produce an accurate water balance.

Minor flooding occurs periodically in many Amherst neighborhoods and affects soil moisture conditions. Historically, much of Ransom Oaks, Audubon and SUNY-Buffalo experienced flooding and were even declared flood hazard areas in the 1970s (MacClennan, 1974). During inspections, many homeowners recounted minor flooding in their neighborhood. Local flooding observed during this study appeared to be caused by overflow of storm water conveyances, which are sized for the 10-year storm event, rather than the overtopping of streams banks.

1.5.7.1 Climate/Precipitation

The Niagara Frontier, including Buffalo and vicinity, experiences a fairly humid, continental-type climate, but with a definite "maritime" flavor due to strong modification from the Great Lakes (NWS, 2005). The average annual temperature is about 48 degrees

Fahrenheit (Table 3). The average annual precipitation at the first-order Buffalo Airport (1971-2000) station is about 40.5 inches, which is uniformly distributed throughout the year. The average annual snowfall for Buffalo is 97.0 inches. During the summer growing season, the potential evaporation is about 24.4 inches, thus evapotranspiration exceeds precipitation and a deficiency of soil moisture generally develops (La Sala, 1968).

Figure 12 shows historical trends in precipitation in the Buffalo area as inferred from the Palmer Drought Index (PDI) and precipitation record. Many homeowners first noticed foundation damage during drier years. The Palmer Drought index uses precipitation and temperature information in a formula to determine dryness, where 0 represents normal and drought is shown in terms of negative numbers: for example –2 is moderate drought, –3 is severe drought, and –4 is extreme drought (NRCC, 2005). The PDI suggests that 1988-89, 1991, 1995, 1998, and 2001 were dry years. Less than average annual precipitation occurred in 1980-81, 1983-84, 1986, 1988, 1994-95, 1998-99, and 2001-03. Prolonged summer dry periods occurred during 1982-86, 1989, 1991, 1994-95, 1998-99, and 2001-02. Periods with two or more months of severe or extreme drought last occurred in the Great Lakes Climate Division in January 1961 (NRCC, 2005)

1.5.8 Hydrogeology

Groundwater geology, or hydrogeology, investigates the origin, occurrence and movement of groundwater, and is potentially associated with foundation damage

Unfortunately, long-term groundwater monitoring data from overburden wells in Amherst are generally absent. Therefore, we present available data that is incomplete and preliminary. These data generally suggest that groundwater levels fluctuate periodically at the footing level and less frequently in the soft stratum. The amplitude of these fluctuations diminishes with depth and there is vertical gradient through the soft stratum. For this discussion, the overburden is subdivided into three zones – upper, middle, and lower.

1.5.8.1 Upper Soil Zone

The upper soil zone extends from the ground surface to about six to eight feet below ground surface, or approximately the depth of a typical foundation footing (Figure 8). Under natural conditions, shallow soils beneath the surface of the ground alternately become wetter and drier as a result of seasonal moisture and temperature changes. That is, groundwater storage normally undergoes seasonal changes because the rates of recharge and discharge are rarely equal. Geophysical, soil boring and monitoring well data provide evidence of these seasonal fluctuations.

Hodge et al. (1973) used geophysics (seismic and resistivity) and hand-auger borings to investigate the overburden near the University of Buffalo's north campus. The study area was bounded by the major roads of French, Sweet Home, North Forest and Campbell Boulevard. The late fall seismic survey identified three and sometimes four distinct layers. Universally, there was a top layer that represented unconsolidated sediment composed of either *unsaturated* sand or clay. Beneath the top layer in most locations was a second layer that represented the water interface. The depth to the *saturated zone* varied considerably throughout the area, but ranged from one to eight feet below the ground surface. Hand-auger borings revealed saturated sand was overlying a sand-clay interface. At several seismic sites, however, the second layer was *stiff clay* described as "unsaturated."

A perched water table typically exists where a more permeable stratum (e.g., sandy loam) overlies a less permeable stratum (e.g., silty clay). About half of the 60 soil borings along the Peanut Line (Figure 7) identified a "wet" sandy horizon overlying a clayey horizon, with some logs explicitly noting "water encountered." Many water level measurements on Figure 7 are shallow and postulated to represent the elevation of the perched water table.

Desiccation cracks and mottling also indicate a fluctuating water content. Both features are commonly recorded in boring logs to depths of 7 to 10 feet below ground surface. Soil mottling (discoloration) reflects the natural depth to a seasonally high water table (Earth Dimensions, 1981). Oily contaminants have migrated along vertical cracks in the Tonawanda landfill to depths of 20 to 25 feet (pers. comm., Glen May, NYSDEC). Contractors who repair foundations have observed "bone dry" conditions at the footing level at many repair sites (pers. comm., ABS, 2003). Desiccation cracks near foundations (Photo 2) greatly increase infiltration rates and the vertical movement of groundwater. Desiccated soils that pitch toward the foundation wall explain, in part, why many homeowners report sump pump cycling shortly after a rainstorm.

The seasonal fluctuations of the water table in some soils may mimic the hydrograph (water level vs. time) of a Ransomville well, located northeast of Buffalo, NY (Figure 13). Approximately weekly water level readings have been gathered by the U.S. Geologic Survey (USGS) from 1972-95 at a farmer's dug well estimated to be 25 feet deep. Two subsurface borings (ARC-66509, AUC-67511) about one mile west of the site show a fairly homogenous sandy clay over a sandy clay with some gravel that is underlain by bedrock at 30 to 50 feet (USACE, 1973). Well use could not be determined. The 1990 to 1994 period was selected to illustrate a contrasting dry (1991) and wet year (1992), with the latter being comparable to the 2004 study year.

Figure 14 illustrates qualitatively the annual change in groundwater storage between February and September, which averages about 4.6 ft. If the upper groundwater zone in parts of Amherst fluctuates to the extent of the Ransomville well, then the active soil zone could experience some dramatic soil moisture changes in some years.

There are approximately 30 shallow to deep monitoring wells located around the former Tonawanda landfill and Spaulding sites (Appendix 6.4). The subsurface material is a uniform, red-brown, silt and clay, with some sand and fine gravel, with faint bedding that transitions to gray-brown clay in deeper wells. This monotonous sequence overlies sand with trace gravel, which overlies weathered clay-filled shale. The depth to rock is typically 60 to 95 feet below ground surface. The blow counts are generally between 10 and 30. This material is different from the typical lacustrine deposits in north-central Amherst and is presumed a till sequence; nonetheless, it may mimic groundwater behavior in south-central Amherst or, more broadly, a impervious formation overlying fractured bedrock.

Figure 15 shows the depth to groundwater at several observation wells along the perimeter of the Spaulding site. The 10-foot well screens have sand packs with a midpoint that is uniformly about 14 feet below the ground surface. The ground elevation at the wells varies from about 591 to 603 feet AMSL. These data show the temporal and spatial variability of the water table across a comparatively small site. For example, the water table fluctuated nearly nine feet at OW-1, and the range of concurrent measurements was nearly 4 to 6 feet.

In summary, geophysical readings, desiccation cracks, mottling, and groundwater measurements from across the region suggest that groundwater in the upper soil zone fluctuates seasonally and likely affects soil moisture conditions near the footing.

1.5.8.2 Middle Soil Zone

The middle zone, unlike the upper zone, appears to be less affected by seasonal fluctuations. In lacustrine deposits, the middle zone corresponds to the soft stratum. The middle zone may be within the capillary fringe during certain periods. Only a few wells have been constructed to monitor groundwater levels in the soft clay stratum, and most of these have extremely short records.

Daigler (2004a) determined the hydraulic head in the soft stratum in northern Amherst for two weeks during August 2004. He placed vibrating wire piezometers in a single borehole at 13, 18, and 25 feet below the ground surface. The soft stratum overlies till (27') and bedrock (33'). These data showed a downward vertical hydraulic gradient (I) of about 0.26 ft/ft (I = ? H/L = 3 ft/11.8 ft). That is, the 13 and 25 foot probes registered heads equivalent to about 7 and 10 feet below ground surface. This snapshot suggests some recharge to the soft silty clay comes from the upper soil zone. The rate of recharge can be estimated using laboratory vertical permeability data (two samples) on the silty clay. Considering groundwater flow through a $1 ft^2$ area, the flux can be computed as Q = K x I x A: where Q is discharge (gals/year), K is hydraulic conductivity (2.7 $gals/ft^2/yr$ or 6.9×10^{-7} ft/min), I is the gradient of 0.26 ft/ft, and A is area 1 ft^2. The discharge is equal to 0.7 gals/year (= 1.1 inches = 2.8 % of total precipitation). This suggests that a meager 1.1 inches is available for bedrock recharge per year (c.f., Kappel and Miller (1996) chose 10 in/yr rate).

Daigler (2004a) showed that the hydraulic head being measured 18 feet below ground actually rose about *three feet* while the head in the till actually dropped about two feet during a two week period. The golf course maintenance department does irrigate from a pond and a bedrock well, but both sources are located several hundred yards away on the first hole. Daigler concluded, "groundwater conditions at the undeveloped site were not steady state and appear to vary from one location to the next."

Ward (1973) installed a *piezometer* (B-9) in soft varved clay near the Amherst Sewage Treatment Plant and measured the head for about four weeks during March 1973. The piezometer opening was at 22.4 feet and the hydraulic head was 9.6 feet below ground surface. As in many boreholes, the hydraulic head in the soft stratum about coincides with the top of the stratum.

A third well was installed at the Amherst Senior Center (Barron and Associates, 1999). The bottom 18 feet of the 24 foot well was constructed with slotted screen. The well casing was sealed with bentonite in the soft lacustrine brown clay. During the initial boring, they encountered water at silt seam about 20 feet below grade. Three days after installing the well, the water level was 20 feet below grade (elevation 558.1 ft msl). Bailing the well dry three days later, they returned after 11 days and the elevation was similarly 20.5 feet below ground surface (elevation 557.6 msl). Grain size analysis from the 18 to 20 foot depth showed the sample that was 81% clay and 17% silt. One plausible explanation for the well behavior may be dewatering of a silt seam and extremely slow recharge due to low hydraulic conductivity.

The Tonawanda landfill has two pair of side-by-side wells (BM-13 & BM-14) comprised of a shallow (S) and moderately deep (D) well (Appendix 6.4). The sandpack midpoints are approximately 15 and 40 feet below the ground surface, respectively. Figure 16 shows there is clearly some hydraulic relationship at BM-13 as well as a fairly constant downward component to the gradient most of the time. Note, these data are not continuous (as illustrated) and span several years. Also, the head is not static but varies about 35 feet in BM-13D. Figure 17 shows similar behavior at BM-14. These well pairs also demonstrate a lateral flow component.

In summary, these few middle zone data from lacustrine and till deposits suggest that (1) the hydraulic head in the zone is not static and (2) that a downward gradient is normally present. We speculate that these groundwater level changes are felt by the upper soil zone and underlying compressible clays.

1.5.8.3 Deep Soil Zone

The deep soil zone occurs from the upper till to bedrock interface. Several wells have been completed in this zone.

Ward (1971, 1973) installed eight *piezometers* during a soils investigation for the Amherst sewage treatment plant. The site is nearly flat and located 1,500 feet south of Tonawanda Creek. All borings encountered the typical lacustrine stratigraphy (Fig. 8) except B-17, which was predominantly till. PS-2 cored through 27 feet of shale and noted 100% water loss. The piezometer openings were completed at the till/rock interface and ranged in depth from 53 to 17 feet below ground. Figure 18 shows the hydraulic heads in 1971 and 1973 (depth annotated on legend). The 1971 data may reflect summer conditions or the somewhat deeper construction. The 1973 data may represent spring conditions; B-17 and B-12 appear to be influenced by the rising river stage in Tonawanda Creek. The discharge in the creek, as measured at Batavia, increased significantly during that period. B-17 would likely have an upward gradient during this period. These data reveal some dynamic conditions in the deeper stratum.

Earth Dimensions collected groundwater data in the fall 1982 from two observation wells (1B-30 and 1B-40) at two sites along Young's Road, north of Sheridan Drive. Both wells bored through lacustrine deposits but eventually placed sandpacks that intercept groundwater from the wet compact sand/till. Figure 19 shows the fluctuation observed in those wells.

Tonawanda landfill has several monitoring wells both north and south of the landfill that are screened in till or at the bedrock interface. Figure 20 shows groundwater elevations during several measurement periods. With the exception of DW-1, most wells show comparatively modest or steady change.

1.5.8.4 Bedrock Aquifers

Groundwater in bedrock in western New York is described in detail by Kappel and Miller (1996), La Sala (1968), Staubitz and Miller (1987), and in less detail in USACE (1979), USACE (1973), Hodge et al. (1973), Ward (1971, 1973), Earth Dimensions (1981), Barron (1999), and Daigler (2004).

The Onondaga Limestone and Camillus Shale are generally regarded as high-yield aquifers (La Sala, 1968; Kappel and Miller, 1996). In Amherst, artesian conditions exist along the base of the escarpment. North of the escarpment, groundwater movement likely mimics the topography and moves from higher to lower parts of the basin. Boreholes that cored several feet into the underlying shale generally did not intercept groundwater but rather lost water. The upper weathered bedrock probably acts variably as an aquitard or groundwater sink; that is, the bedrock surface may channel groundwater flow along uneven topography or allow it to percolate deeper into the rock.

1.5.8.5 Summary

Long-term groundwater data from clustered wells in Amherst was not available for this report, nonetheless, the cited studies provide a starting point for future investigations.

In general, the upper soil zone undergoes seasonal changes in groundwater storage that are typical of the northeastern United States (La Sala, 1968). Flat terrain, relatively impervious soils, and few incised features suggest lateral movement of groundwater could be quite limited. The well data suggests groundwater moves vertically downward with an accompanying head loss. The amplitude of fluctuations in the groundwater levels becomes more subdued with depth. However, a few till wells respond quickly to phenomenon such as river stage or possibly groundwater pumping. The fate of groundwater reaching the till or bedrock is unknown, but may follow the bedrock topography or enter the deeper bedrock aquifer. Importantly, fluctuating groundwater levels in the upper and middle soil zones suggest that soil moisture conditions could change periodically in the stiff stratum and perhaps less frequently in the soft stratum.

1.6 HISTORICAL PERSPECTIVE

1.6.1 Amherst Growth and Land-Use

During the past fifty years, the town of Amherst has experienced significant growth, increasing from a population of 33,744 in 1950 to 116,510 in 2000 (Table 4). Amherst's share of Erie County's total population has also increased, from less than 4% in 1950 to over 12% in 2000. The Town saw its greatest growth and largest percentage increase, 46%, in the 1950's and 1960's. The growth rate in the 1990's was approximately 4%. According to population growth estimates, the Town can expect to grow by about 11,000 to 22,000 people over the next 20 years, to a total of 127,264 to 138,839.

By 2020, approximately 5,000 to 10,000 additional housing units could be built in Amherst to accommodate new residents and future growth. The number of building permits for single-family dwellings (multi-family not included) during 1990–2004 averaged 194 ± 68 (1s).

Significant land use changes have occurred since 1972. Approximately 55% of vacant and agricultural land in the Town has converted to other uses (Table 5). During the 1980's and 1990's, the non-residential uses increased, and Amherst has become a major employment center.

1.6.2 Special Flood Hazard Areas

Property owners within the town of Amherst became eligible to purchase flood insurance through the National Flood Insurance Program (NFIP) on August 9, 1974. From 1974 to December 17, 1984, the flood insurance program was regulated under the provisions of the emergency program of the NFIP. On December 18, 1984, the regular program of the NFIP became effective and continues to the present date.

Under the emergency program, the Town adopted its first official floodplain map on February 27, 1978. The floodplain maps depict the Special Flood Hazard Area (SFHA) which is commonly known as the 100-year floodplain (Figure 21). Updated Flood Insurance Rate Maps (FIRM's) were adopted in association with the regular program in 1984. Since then, there have been two major revisions to the FIRM's on September 28, 1990 and October 16, 1992. The FIRM's from 1992 are still in effect. The current 100-year floodplain covers approximately 24% of the town of Amherst.

Within the regulated SFHA, certain structures, including dwellings, must be built in accordance with floodplain regulations so that flood insurance can be obtained. The conventional floodplain regulations require the lowest floor of a structure, including the basement floor, to be constructed above the 100-year flood elevation (also known as the base flood elevation - BFE). However, the town of Amherst obtained an exception to the conventional regulations in 1978. The Federal Insurance Administrator of the Department of Housing and Urban Development approved the exception on November 20, 1978.

The so-called basement exception allows for the construction of residential basements where the basement floor is located at a lower elevation than the base flood elevation. The basement exception permits the basement floor elevation to be no more than five feet below the BFE and the building must comply with other structural and elevation requirements. When using the basement exception option, the first floor elevation (not including the basement floor) must be elevated to at least one foot above the BFE and the structure must be flood-proofed to one foot above the BFE.

1.6.3 Building Codes for NYS and Amherst

Table 6 provides a chronology of important building codes that have been adopted in the town of Amherst. From 1936 through end of 2002, the building codes for one- and two-family dwellings under 40-feet in height allowed the bearing capacity of soil to be based upon the presumptive bearing value for that soil. The presumptive bearing value is determined by identifying the soil type and then obtaining a bearing value as listed in a table contained in various design manuals, building codes, or engineering books. This methodology does not involve any geotechnical analysis.

For one- and two-family dwellings, the relevant code sections for soil bearing values associated with the building codes are listed in chronological order below:

1) Building Code of the Town of Amherst (1936-77, known as Building Ordinance):

Subdivision 8.4 – Bearing Value of Soils – The bearing power of the soil on the maximum of live and dead loads combined, in tons per square foot of bearing surface on the ground, shall not exceed the following:

Soft Clay	1
Wet Sand	2
Ordinary clay and sand together in wet and springy layers	2
Loam, clay or fine sand; firm, clean and dry	3
Hard, dry clay	4
Very firm, coarse sand or stiff gravel	5

Where the bearing power of the soil is doubtful or undetermined, the Inspector may direct that borings or soil tests be made. Such tests shall be made under his supervision and he shall keep a record of their results.

2) The "State Building Construction Code" (1977-83):

Section A 302-2(a) – For buildings 40 feet or less in height, the allowable bearing value of the soil upon which the building rests shall be the presumptive bearing value or shall be determined by field loading tests in conformity with generally accepted standards.

3) State Uniform Fire Prevention and Building Code (1984-02):

Section 801.2(a)(1) - For buildings 40 feet or less in height, the allowable bearing value of the soil upon which the building rests shall be the presumptive bearing value, or shall be determined by field loading tests in conformity with generally accepted good engineering practice.

4) Residential Code of New York State (2003 to present):

Section R401.4 Soil tests. In areas likely to have *expansive, compressible, shifting or other unknown soil characteristics*, a soil test shall be performed to determine the soil's characteristics at a particular location. This test shall be made by an approved agency using an approved method.

Until the Residential Code of New York State became effective in 2003, soil analysis was based on the “presumptive bearing value” method. Furthermore, architects and professional engineers were relied upon to exercise good engineering practice in the preparation of construction plans.

It is the current practice of the town of Amherst Building Department to require soil testing and geotechnical analysis prior to the issuance of a building permit for any proposed dwelling located north of the Onondaga Escarpment. At the discretion of the Commissioner of Building, soil testing may be required for proposed buildings located on the Onondaga Escarpment.

The design and construction of a dwelling must take into account the soil conditions for each specific site. The construction drawings and specifications must include the design recommendations of the geotechnical engineer.

In addition to the more detailed soil analysis, the Building Department has adopted a policy to require compressive strength tests for concrete that is used for footings and foundation walls. The compressive strength test results must be submitted to the Building Department prior to the issuance of a Certificate of Occupancy. Minimum compressive strength requirements are specified in the Residential Code of New York State. This testing addresses defective concrete concerns.

In accordance with New York State Education Law, plans, specifications and reports relating to residence buildings of gross floor area of fifteen hundred square feet or less, not including garages, carports, porches, cellars, or uninhabitable basements or attics are not required to be prepared by a licensed architect or professional engineer. Unless exempt from NYS Education Law, all other buildings, plans, specifications, and reports relating to the construction of buildings shall be stamped and signed by a professional engineer, architect or land surveyor. Regardless of any exemption from the NYS Education Law, the Building Department currently requires reasonable soil testing and geotechnical analysis prior to the issuance of a building permit.

Since the adoption of the first building code in 1936, the Town has conducted plan reviews in association with each building permit application. Plan reviews were based upon the adopted building code at the time of the permit application. Subsequent to the issuance of a building permit, town of Amherst Building Inspectors conduct appropriate construction inspections. From 1936 through 1951, the town's Building Inspectors were assigned to the Engineering Department. The Building Department was created in 1951 and now employs approximately 27 inspectors in various job titles.

Upon the adoption of the State Uniform Fire Prevention and Building Code on January 1, 1984, the state also established minimum standards for the administration and enforcement of that code. The minimum standards included (among other items) provisions for construction inspections, such as observation of the foundation, structural elements, electrical systems, plumbing systems, heating, ventilation and air conditioning systems, fire protection systems and exit features. The Town conducted inspections and continues to conduct inspections as established by state standards.

The information contained in this section is just a brief overview of the codes, inspections, and building permit process. Further detailed information regarding these topics can be obtained from the Town of Amherst Building Department.

1.7 Figures, Table, Photos

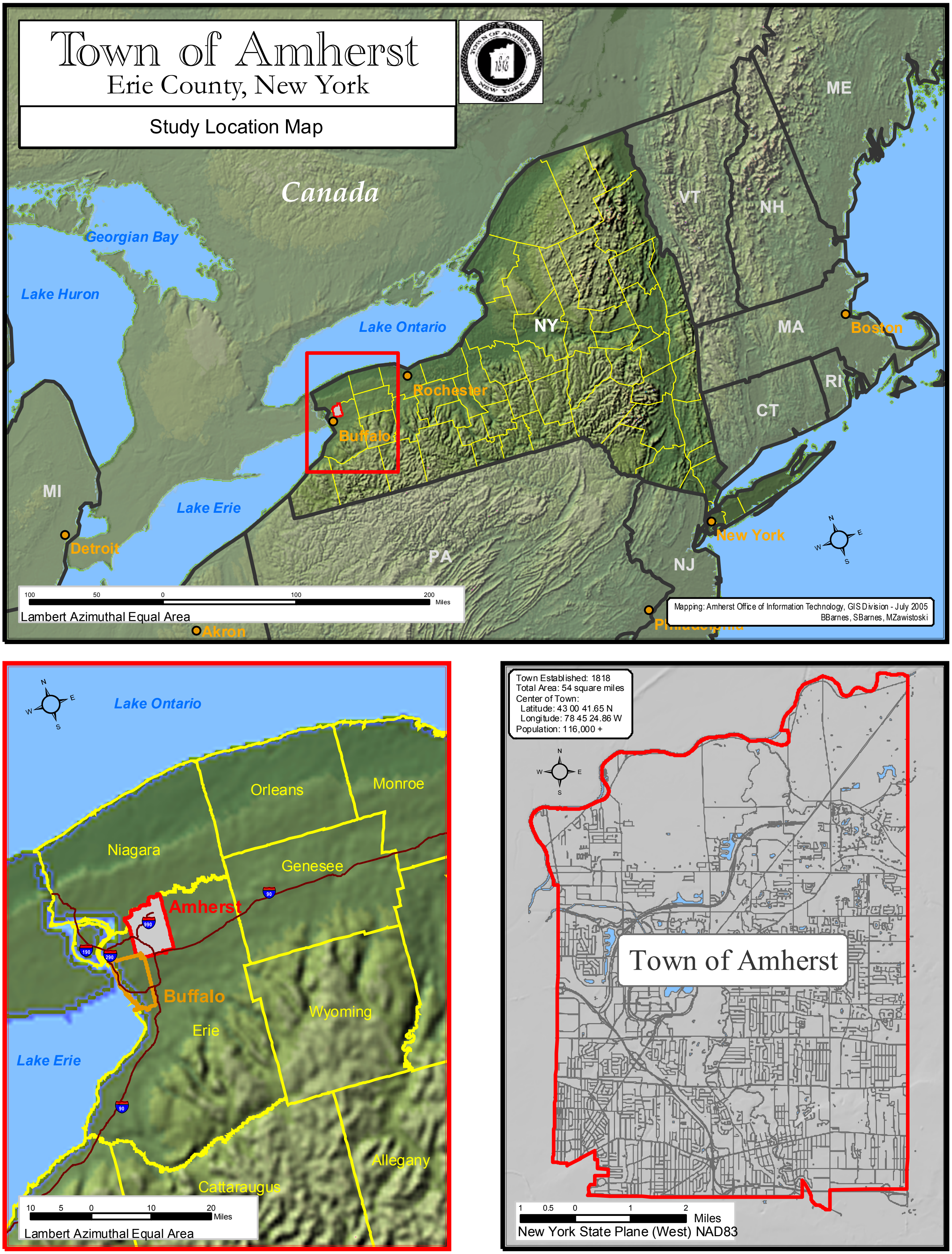

Figure 1. Study location maps of Town of Amherst, Erie County, and western New York.

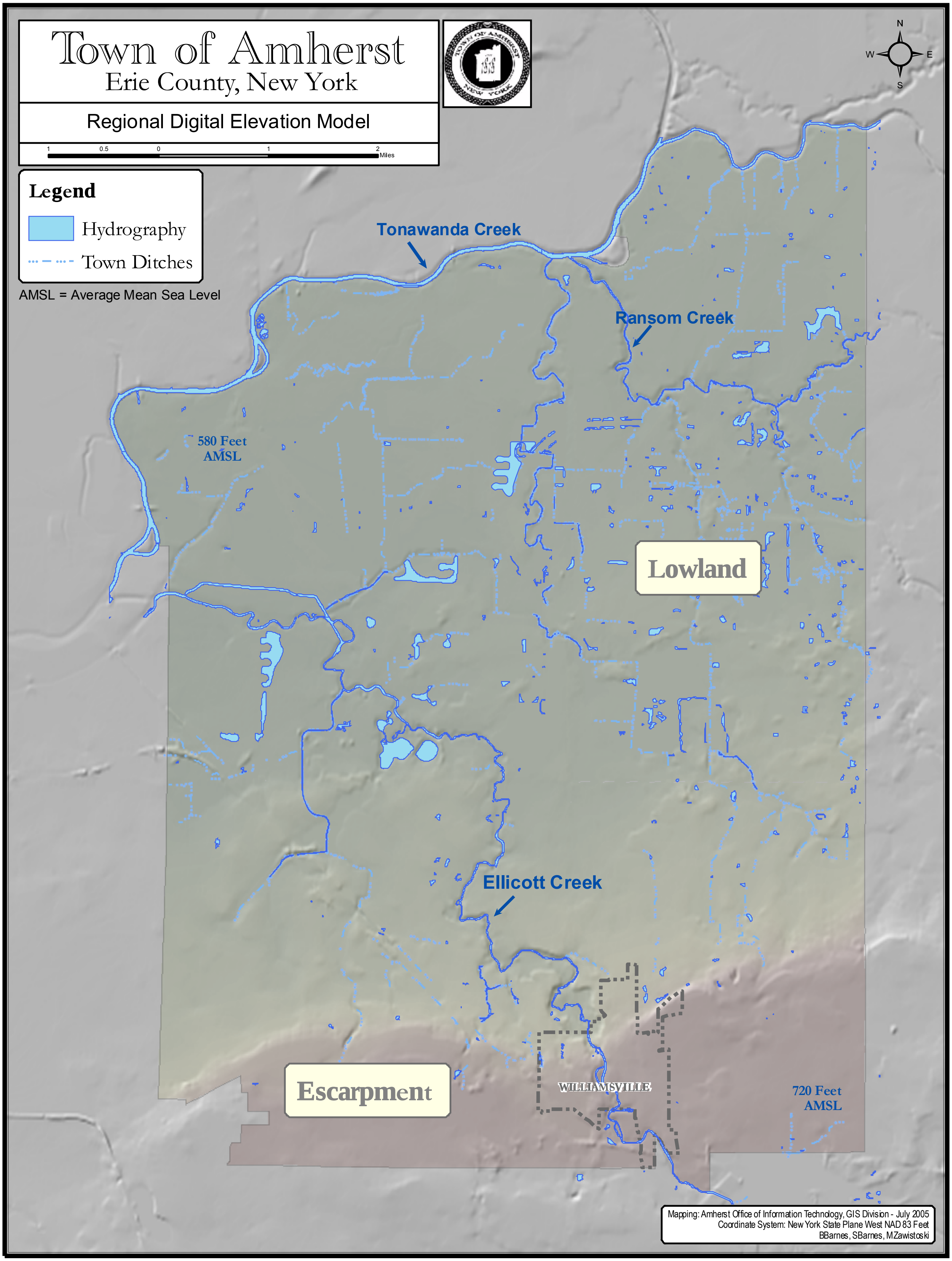

Figure 2: Digital elevation model (DEM) of Amherst, NY, showing change in relief from escarpment to lowland areas.

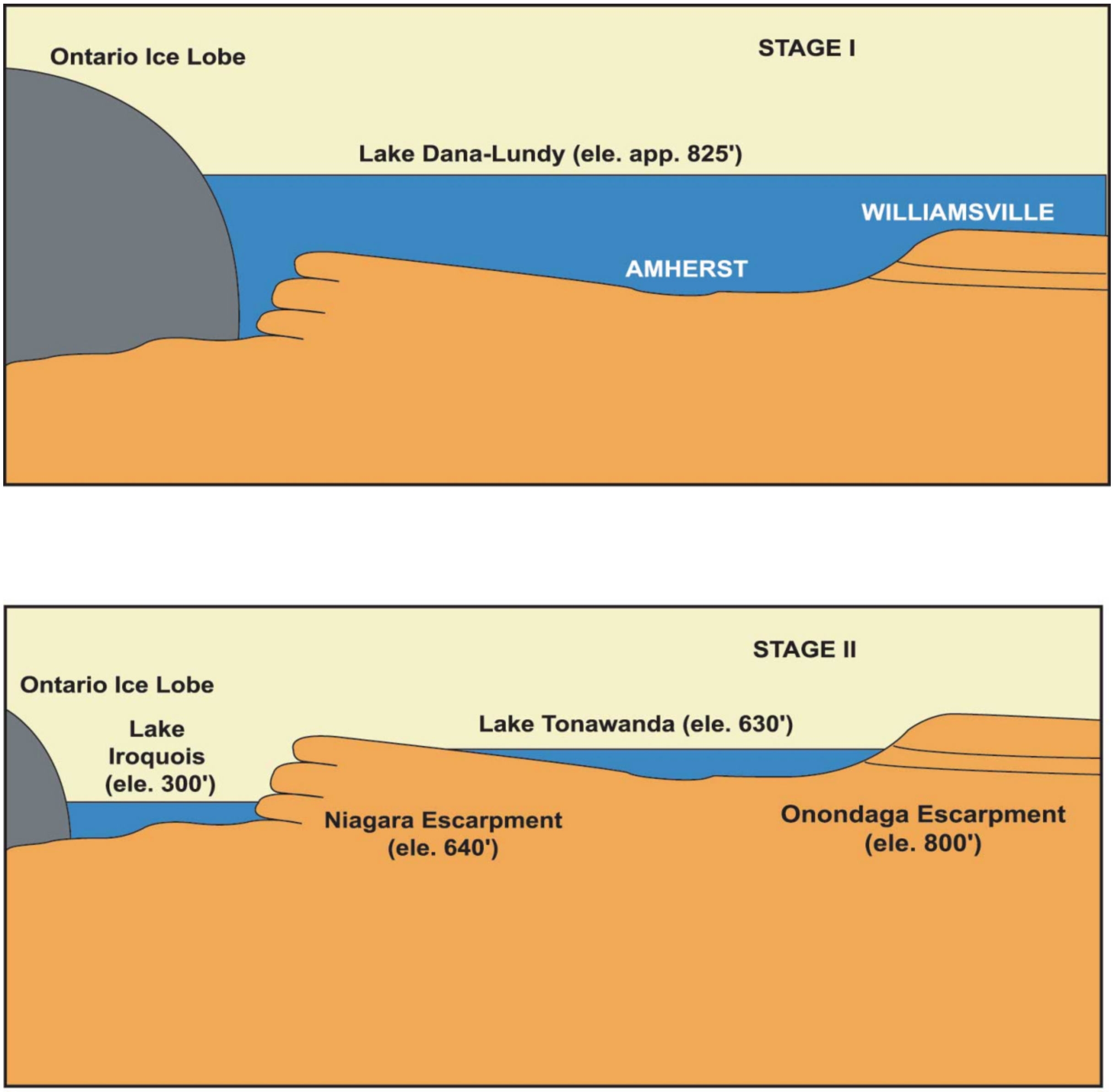

Figure 3. Lake Tonawanda is a remnant of Lake Dana-Lundy resulting from the final northward retreat of the Ontario ice lobe of the Wisconsin glacial ice sheet when the ice front was located between the Niagara escarpment and Lake Ontario (from D'Agostino, 1958).

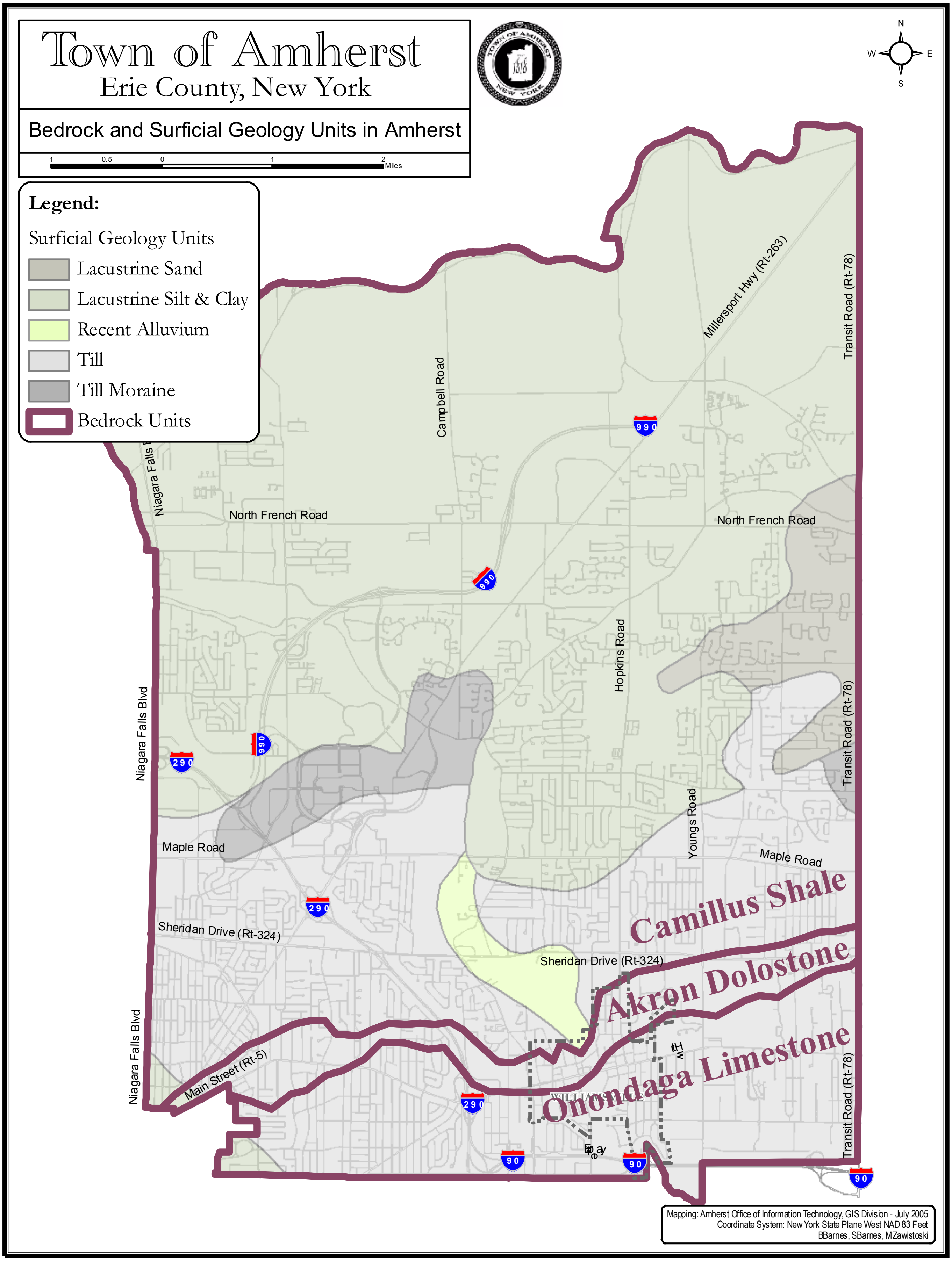

Figure 4: Surficial and bedrock geology of Amherst, NY. Lacustrine deposits (north) and till (south) overly gypsiferrous shale and calcarious rocks (dolostone and limestone) respectively (Source: NYSGS, 2005).

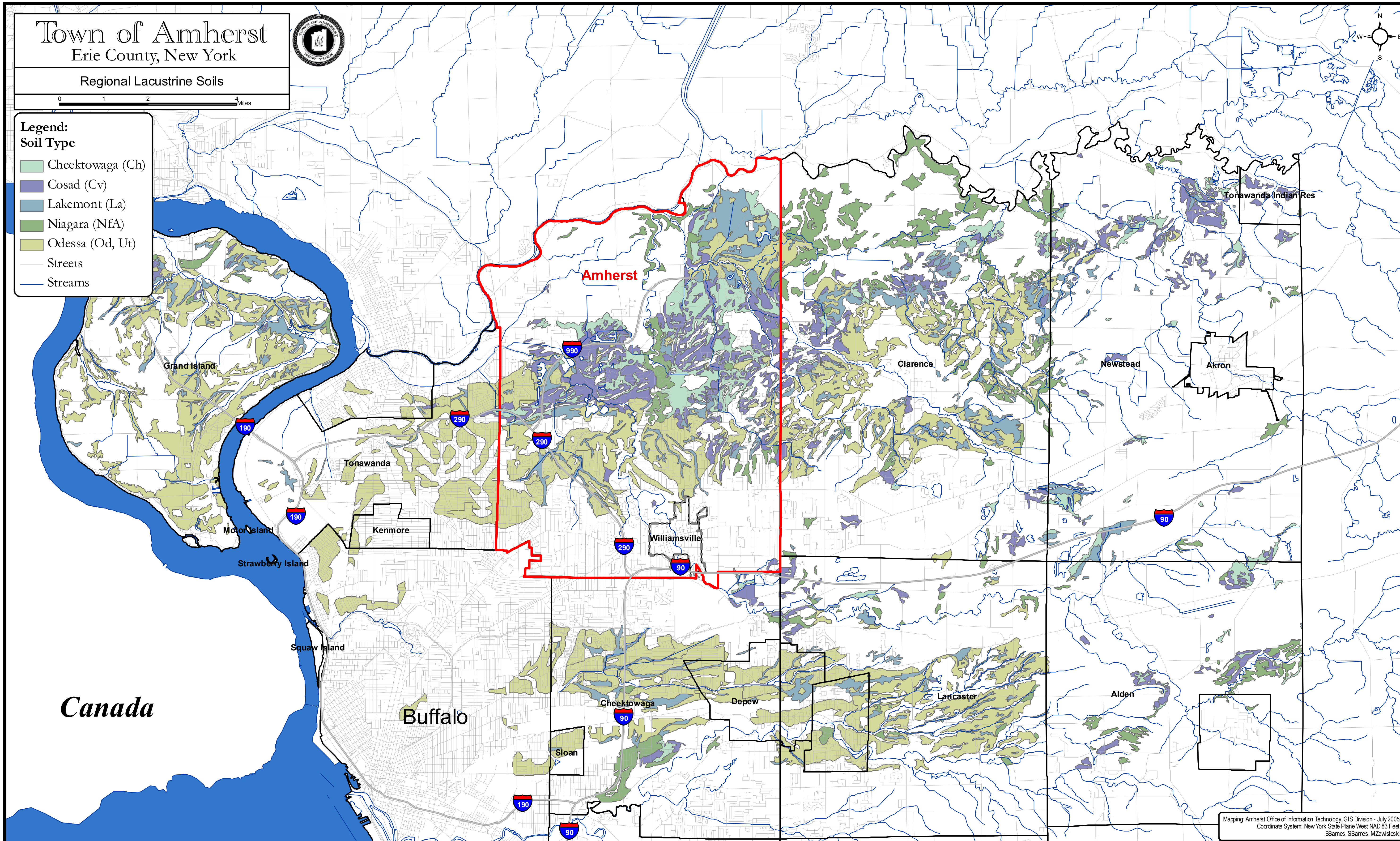

Figure 5: Distribution of five lacustrine surface soil types in Amherst, NY.

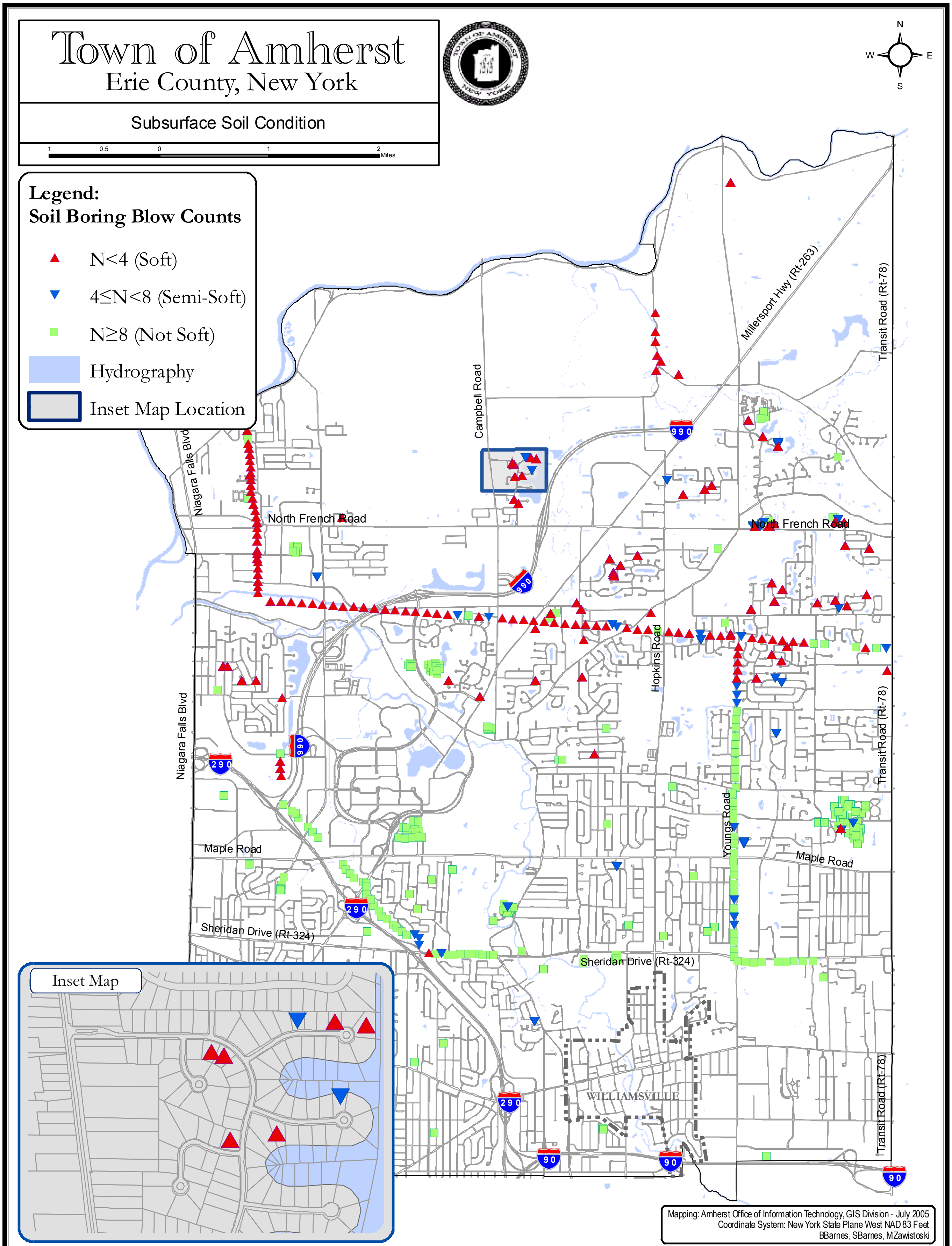

Figure 6: Location of boreholes and the presence of "soft," "semi-soft," or "not soft" strata. These designations are based on the N-values (blow counts). The soft stratum generally consists of silty clay. In general, the northern portion of Amherst is underlain by the soft stratum, but the inset shows variability at parcel level.

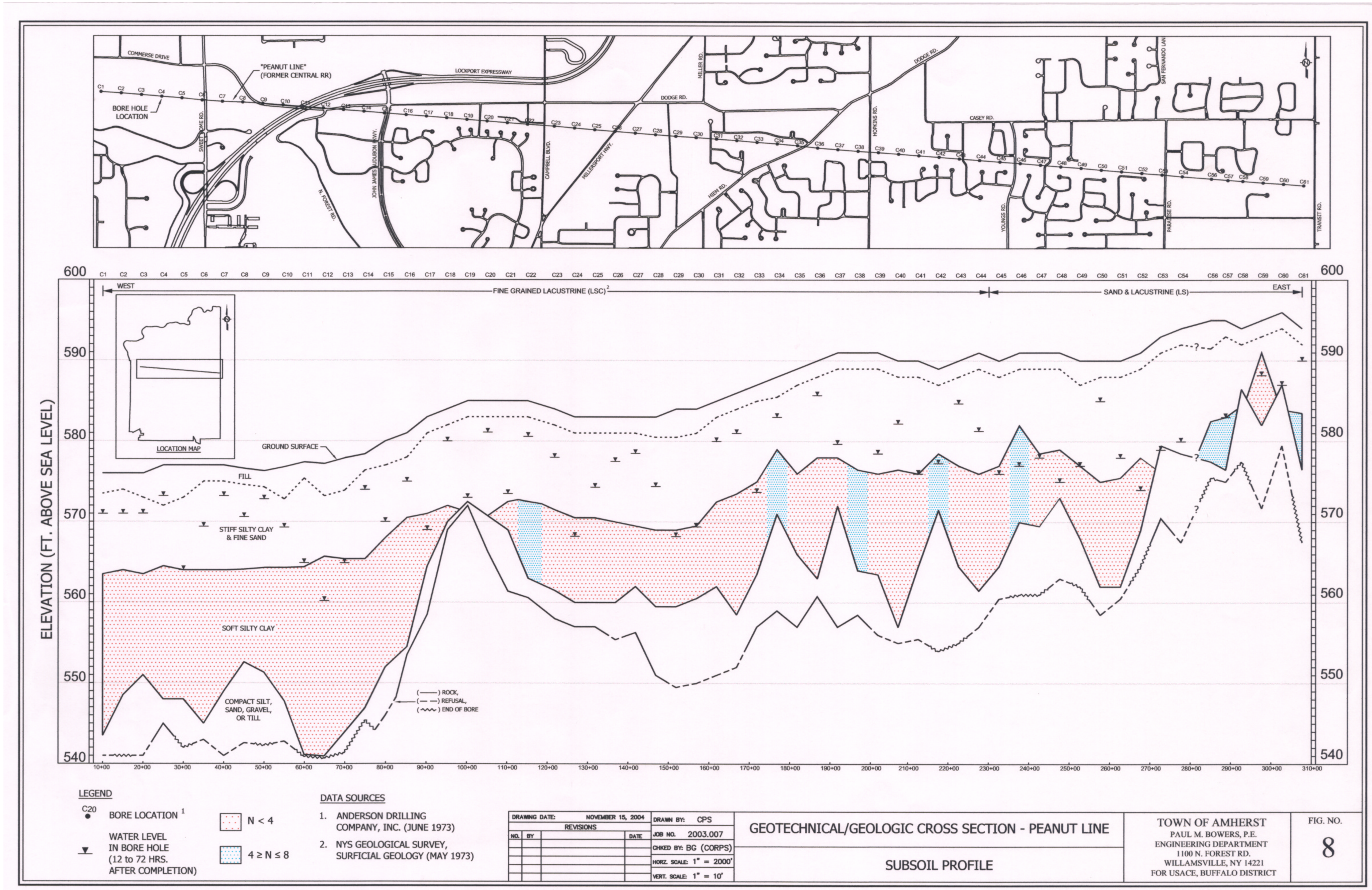

Figure 7. Geotechnical/geologic cross-section through central Amherst along former New York Railroad, or "Peanut Line." Upper section shows aerial view of borehole locations. Cross-section shows elevation of ground surface and depths of fill, soft stratum (if present), till, and borehole termination (end-of-borehole, refusal, or rock). Stipling shows thickness of soft stratum (red) and semi-soft stratum (blue), generally silty clay. Micro-topography of the bedrock and surface appear to influence the location of softer stratum. Subsurface condition becomes more heterogeneous in east towards Clarence.

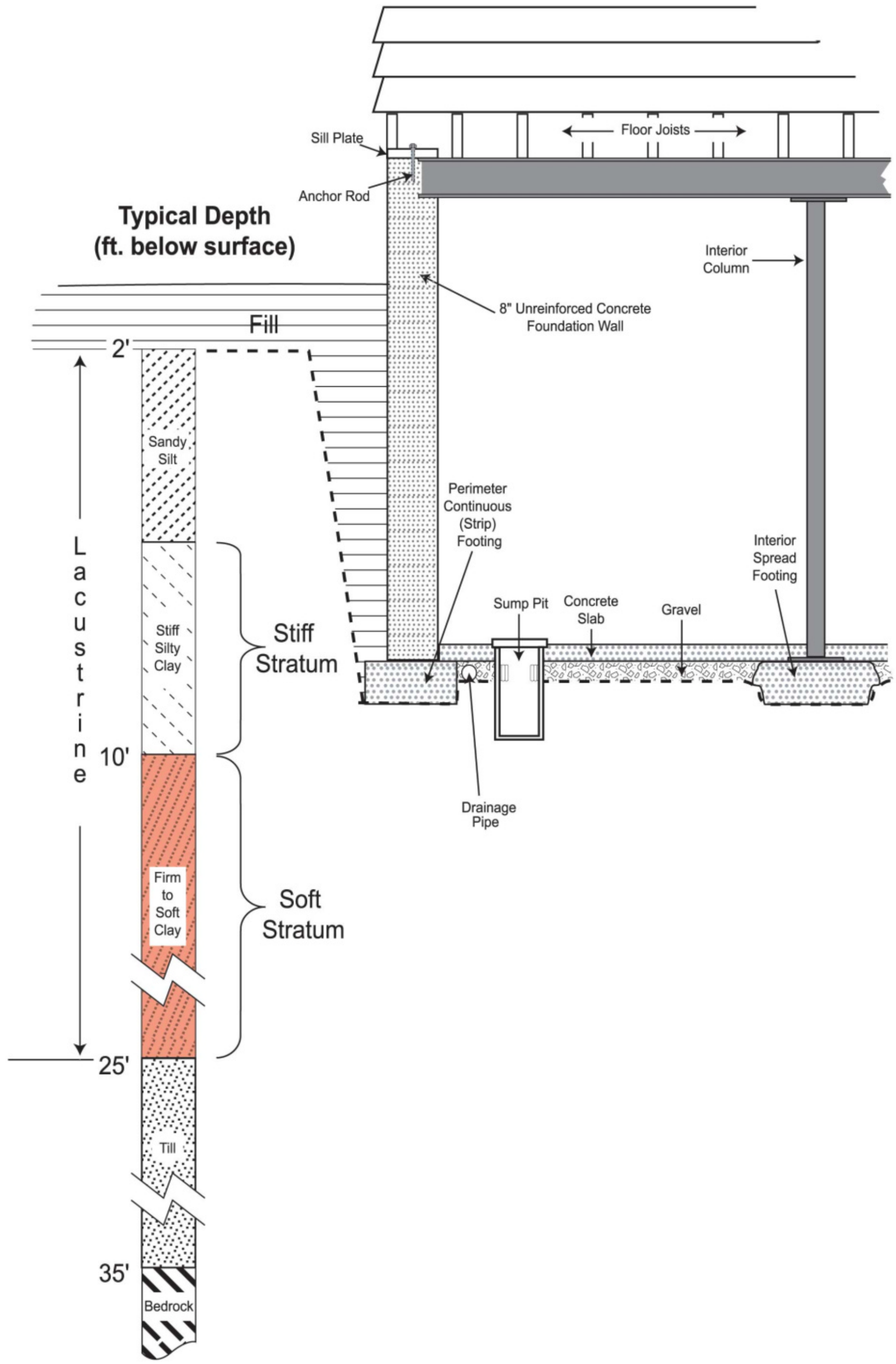

Figure 8. Schematic stratigraphic soil profile in central and northern Amherst showing typical construction of older houses. Many footings rest on a stiff stratum that overlies a soft stratum. Fill consists of remolded native material.

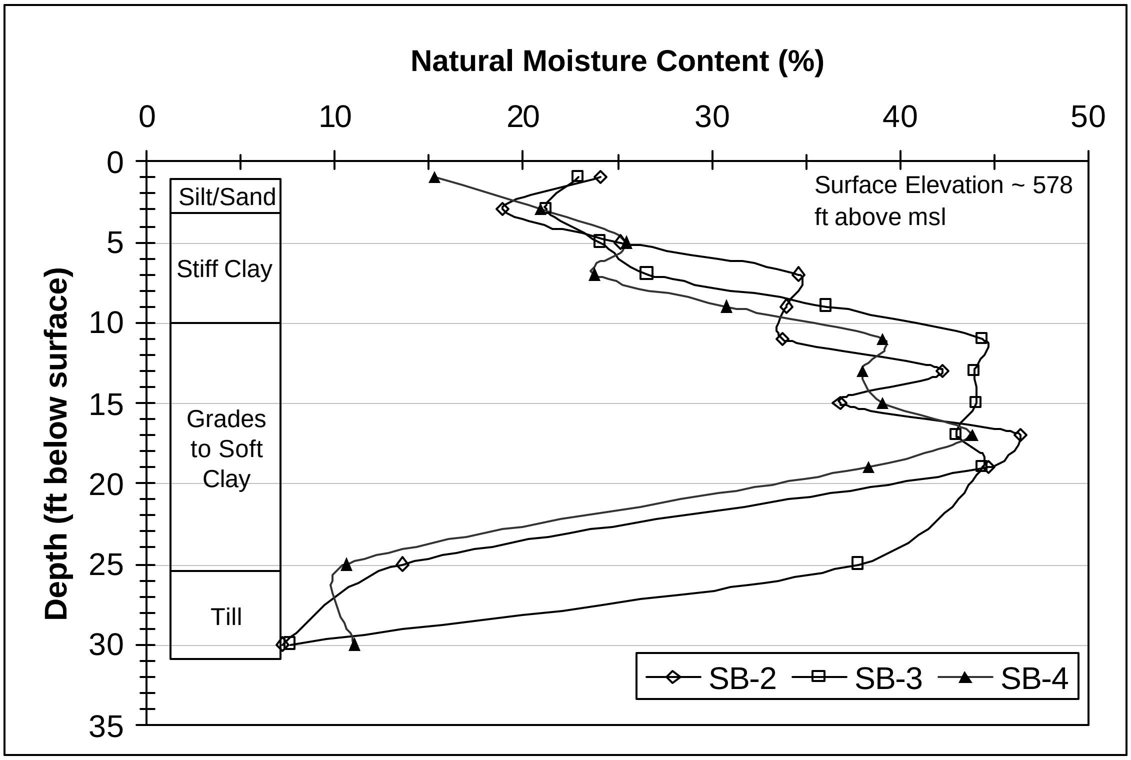

Figure 9. Typical soil moisture content profile from three soil borings at sewage treatment plant in northwestern Amherst. Data from Ward (1973).

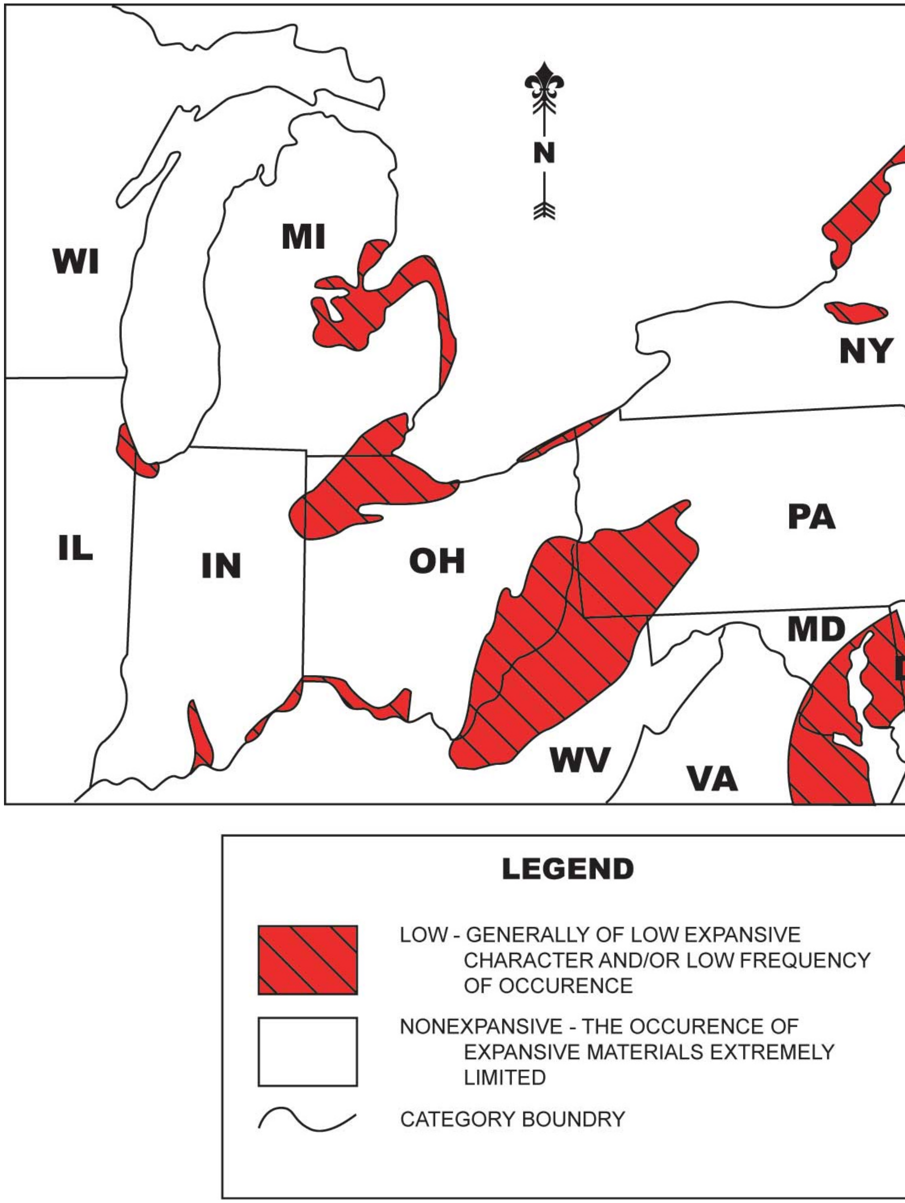

Figure 10. Distribution of potentially expansive soils in the Great Lakes and Northeastern United States (modified from FHWA, 1975).

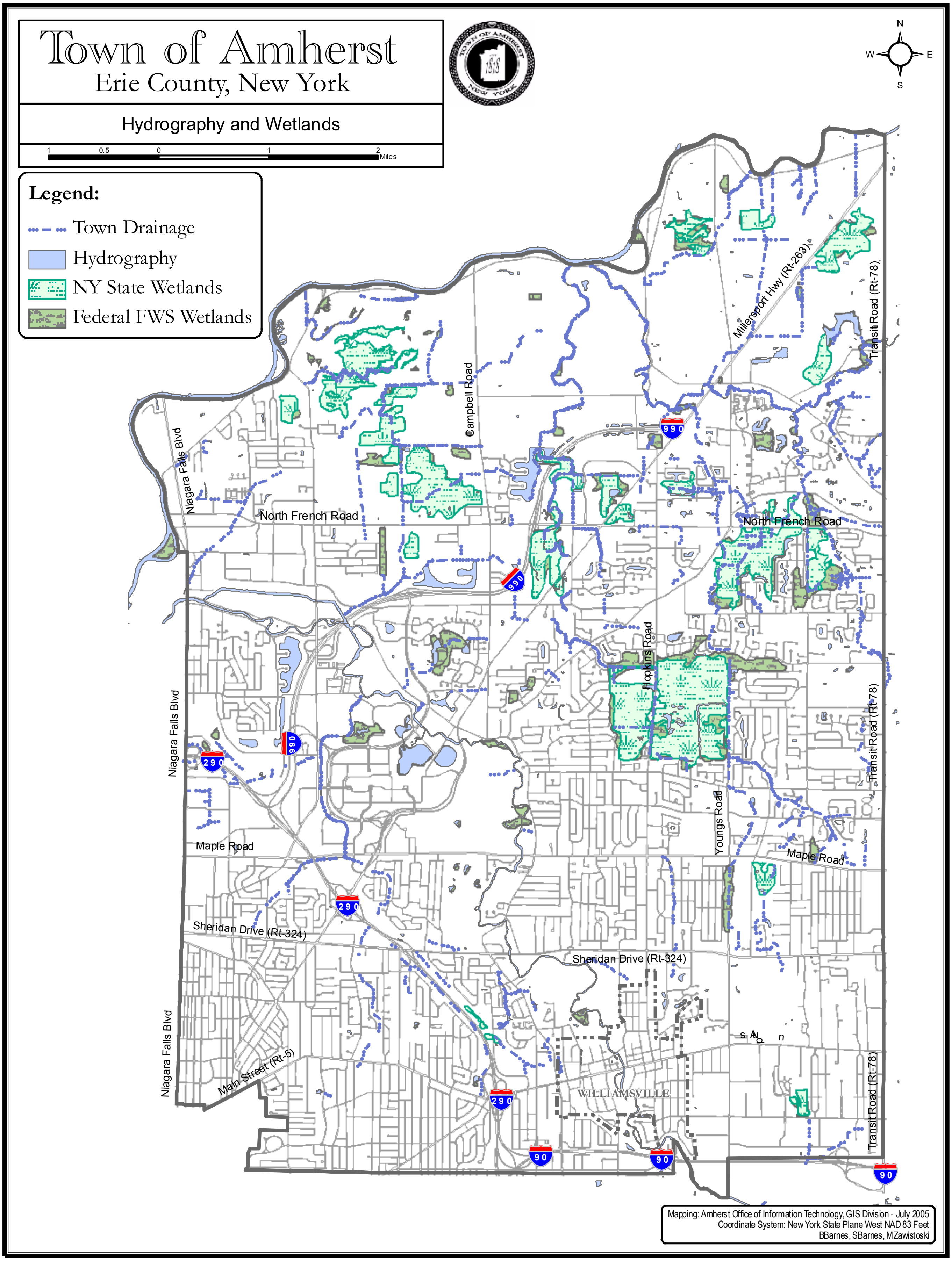

Figure 11: Hydrography and wetlands of Amherst, NY.

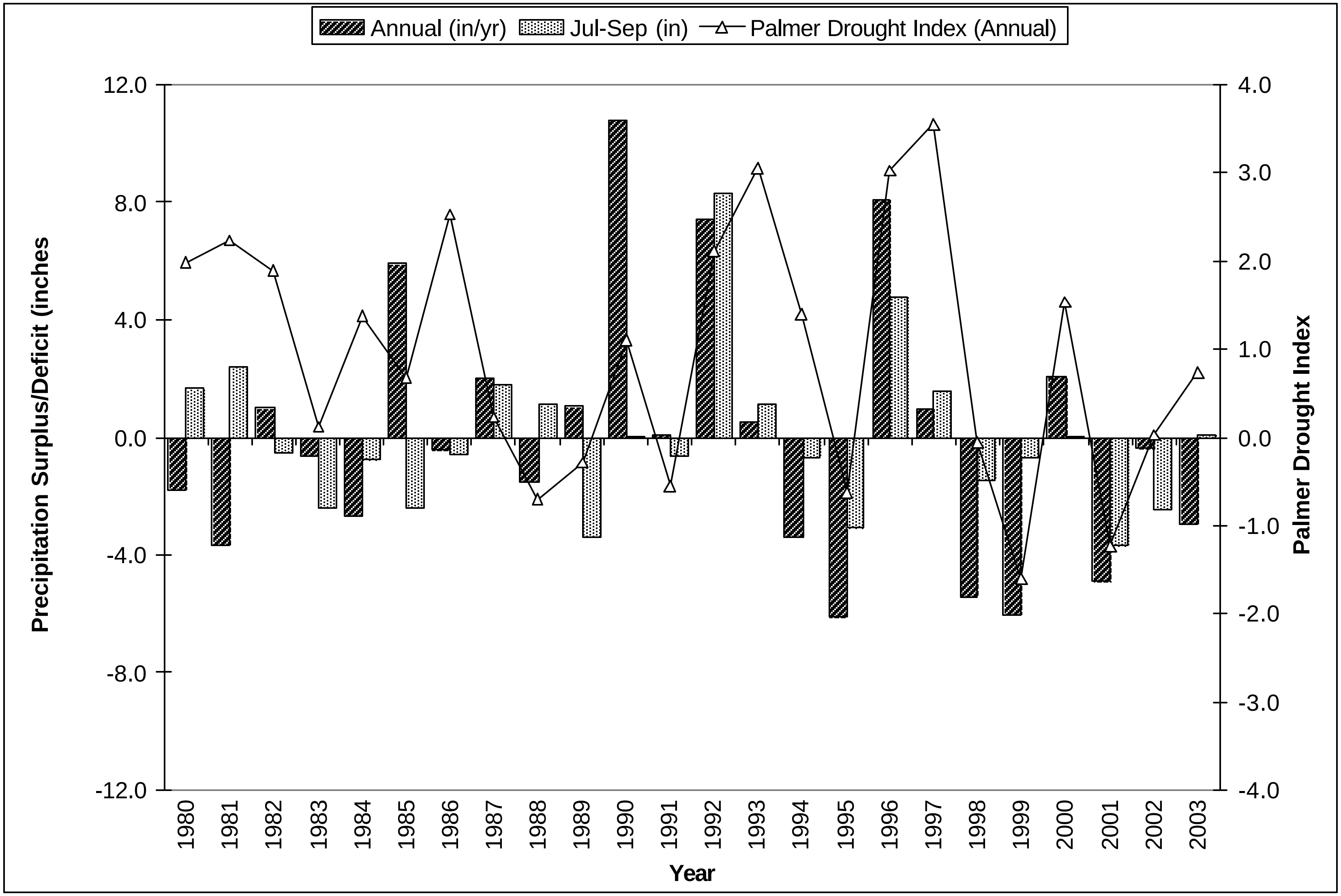

Figure 12. Annual and summer precipitation deficit and surplus (inches from normal) and the annual Palmer Drought Index. For example, in 1996 the summer and year was wetter than normal, while 2001 was drier than normal.

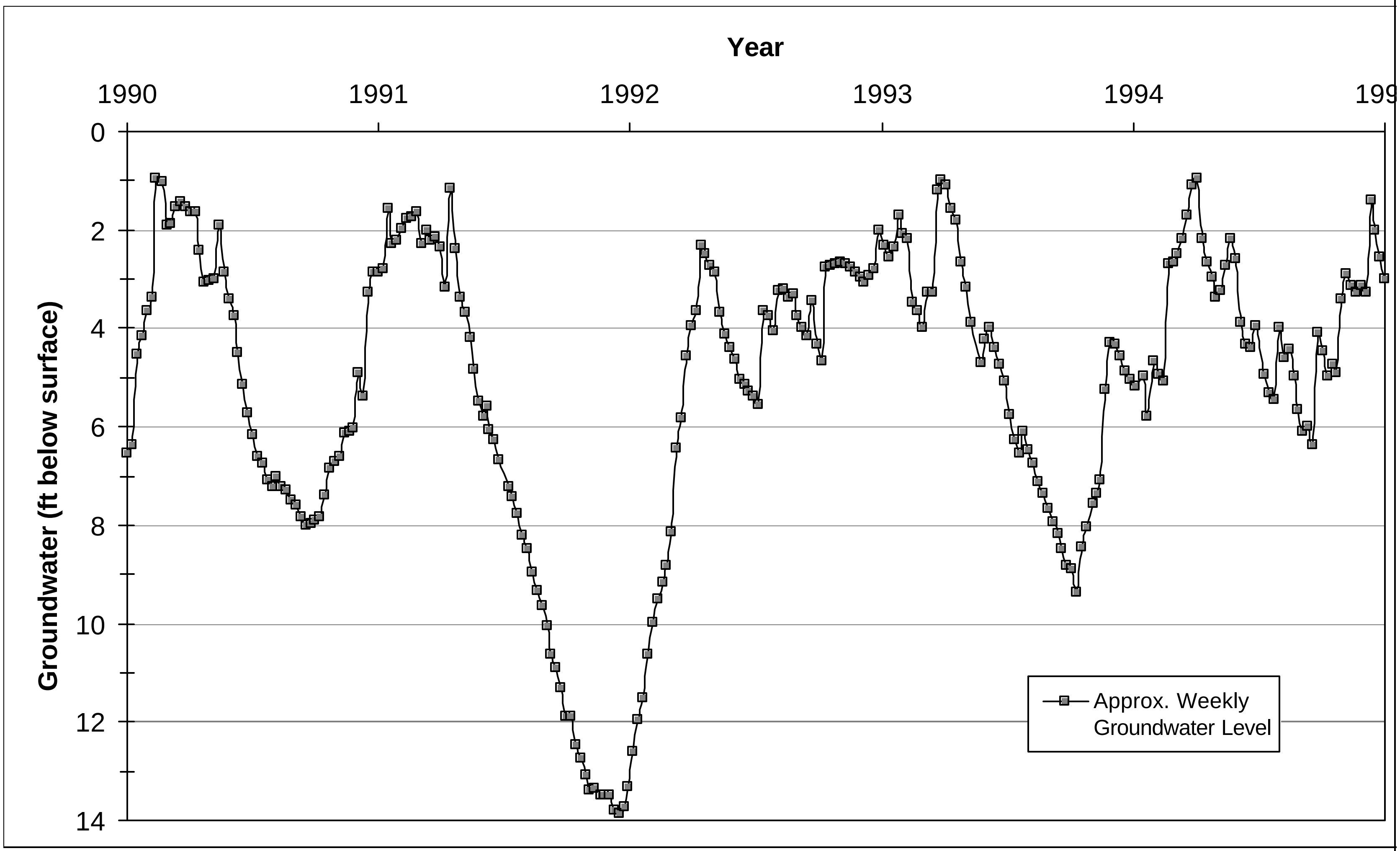

Figure 13. Hydrograph of Ransomville well, near Buffalo, NY, showing weekly groundwater levels in dug well (~25 ft deep) in possible sandy clay stratum. Dry and wet years correspond to 1991 and 1992. Data collected by the USGS (Well No. NI-70).

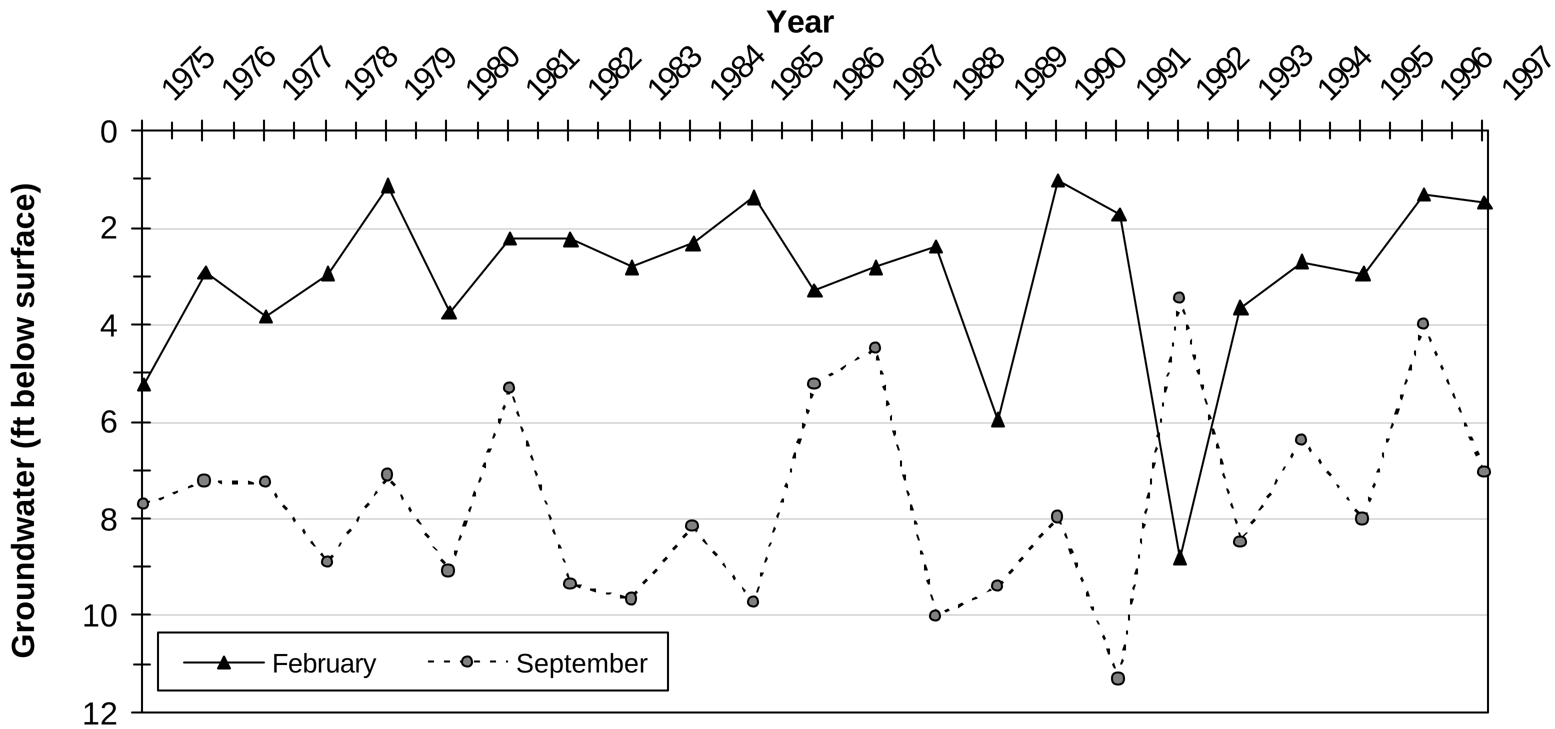

Figure 14. Hydrograph of Ransomville well, near Buffalo, NY, shows qualitatively the annual change in storage from late-February to late-September (1975-96). The dug well is 25 foot-deep and the subsurface may be sandy clay. Data collected by the US GS (Well No. NI-70).

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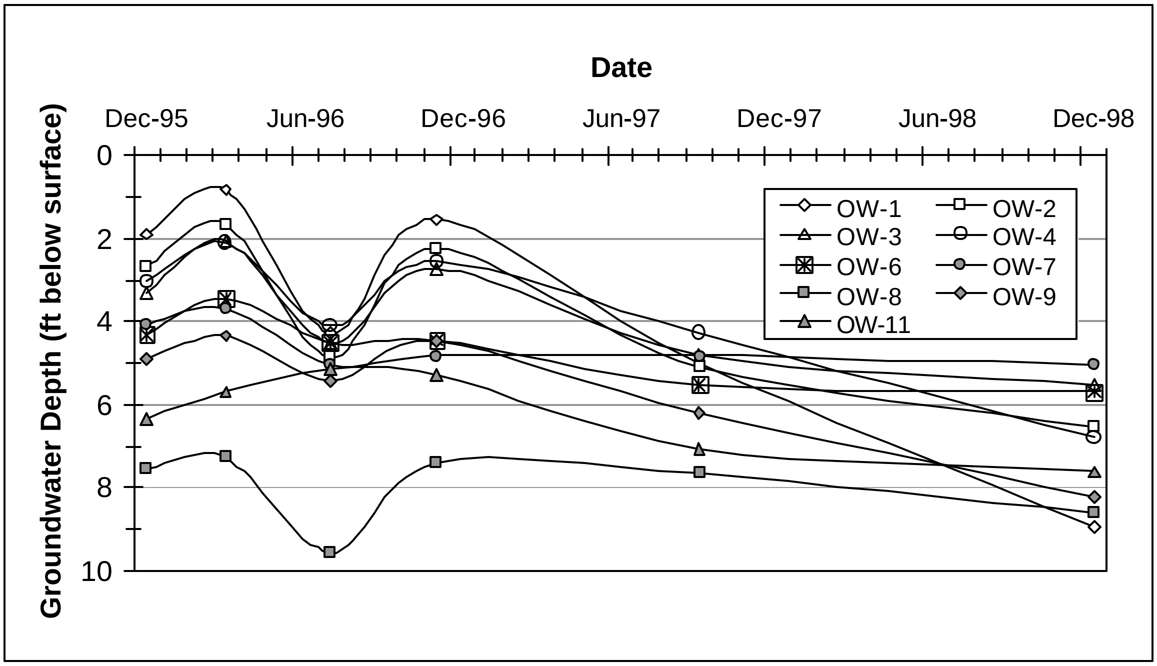

Figure 15. Groundwater levels in shallow wells at Spaulding Site, north of former Tonawanda Landfill (see Appendix 6.1.4). The midpoint of the sandpack for all wells is about 14 feet below ground surface in silty clay (probably till). Data shows spatial and temporal variability in shallow groundwater level. (Data source: Glen May, NYSDEC, 2005)

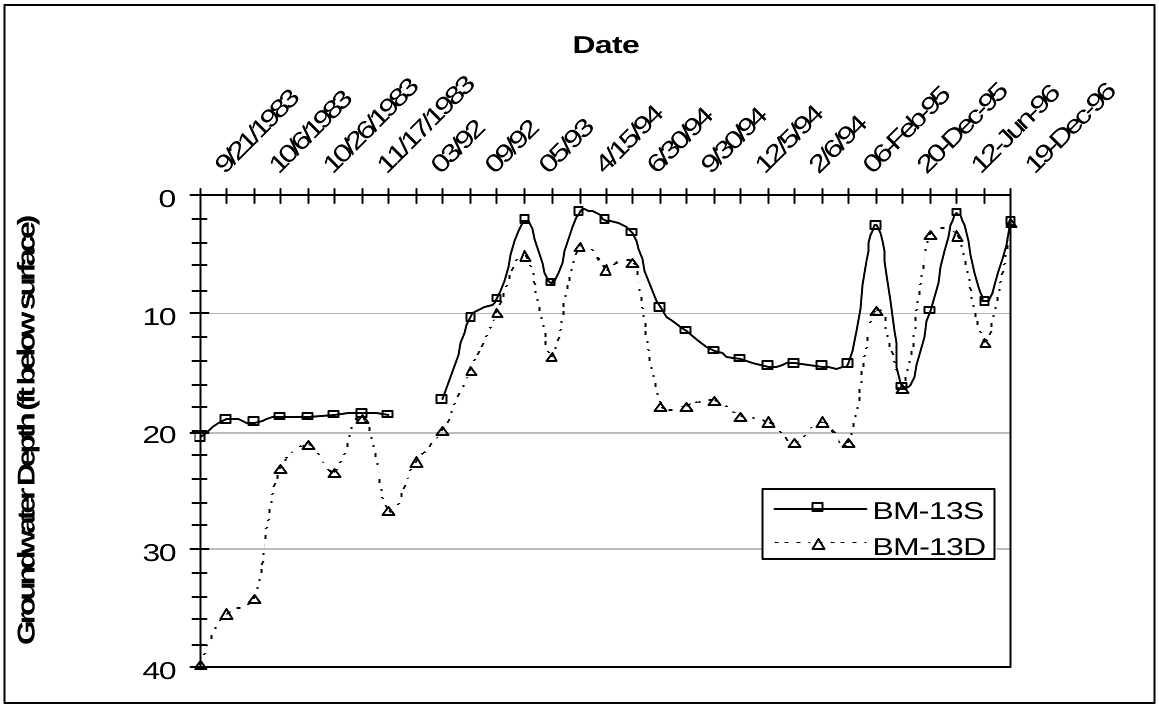

Figure 16. Tonawanda landfill wells BM-13 shallow (S) and deep (D). Sandpack midpoints are approximately 15 and 40 feet below ground surface. Note discontinuous dates. (Data source: Glen May, NYSDEC, 2005).

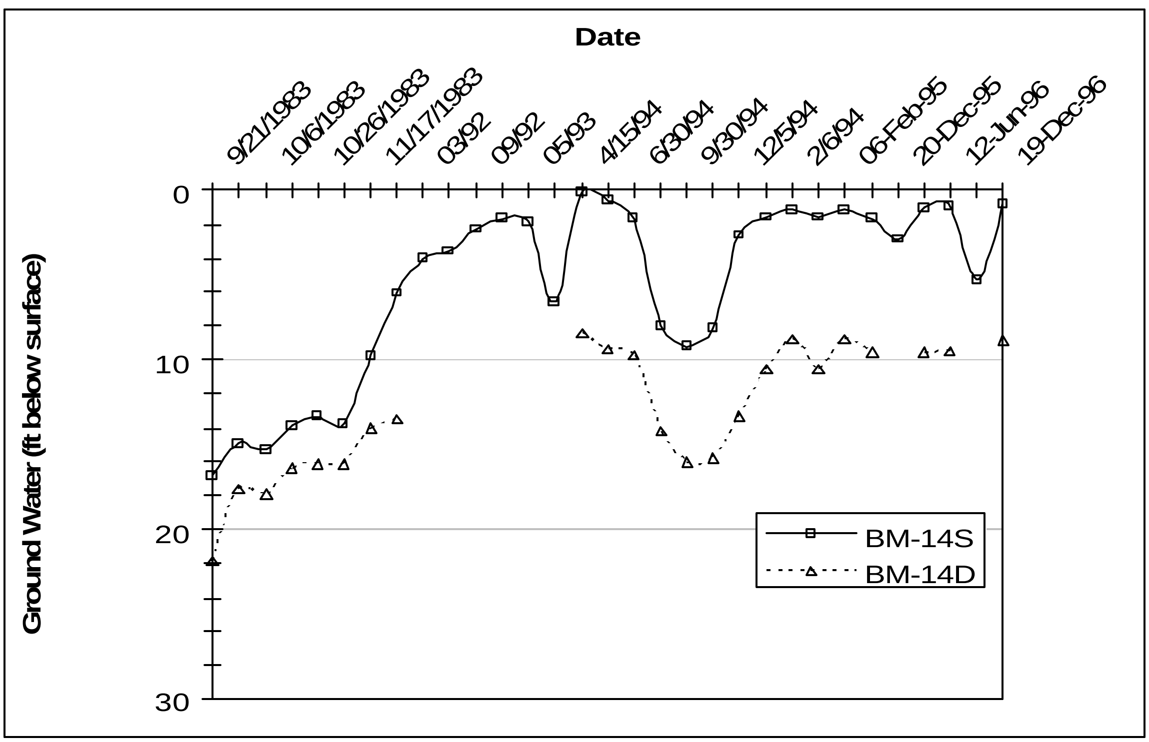

Figure 17. Tonawanda landfill wells BM-14 shallow (S) and deep (D). Sandpack midpoints are approximately 15 and 40 feet below ground surface. Note discontinuous dates. (Data source: Glen May, NYSDEC, 2005).

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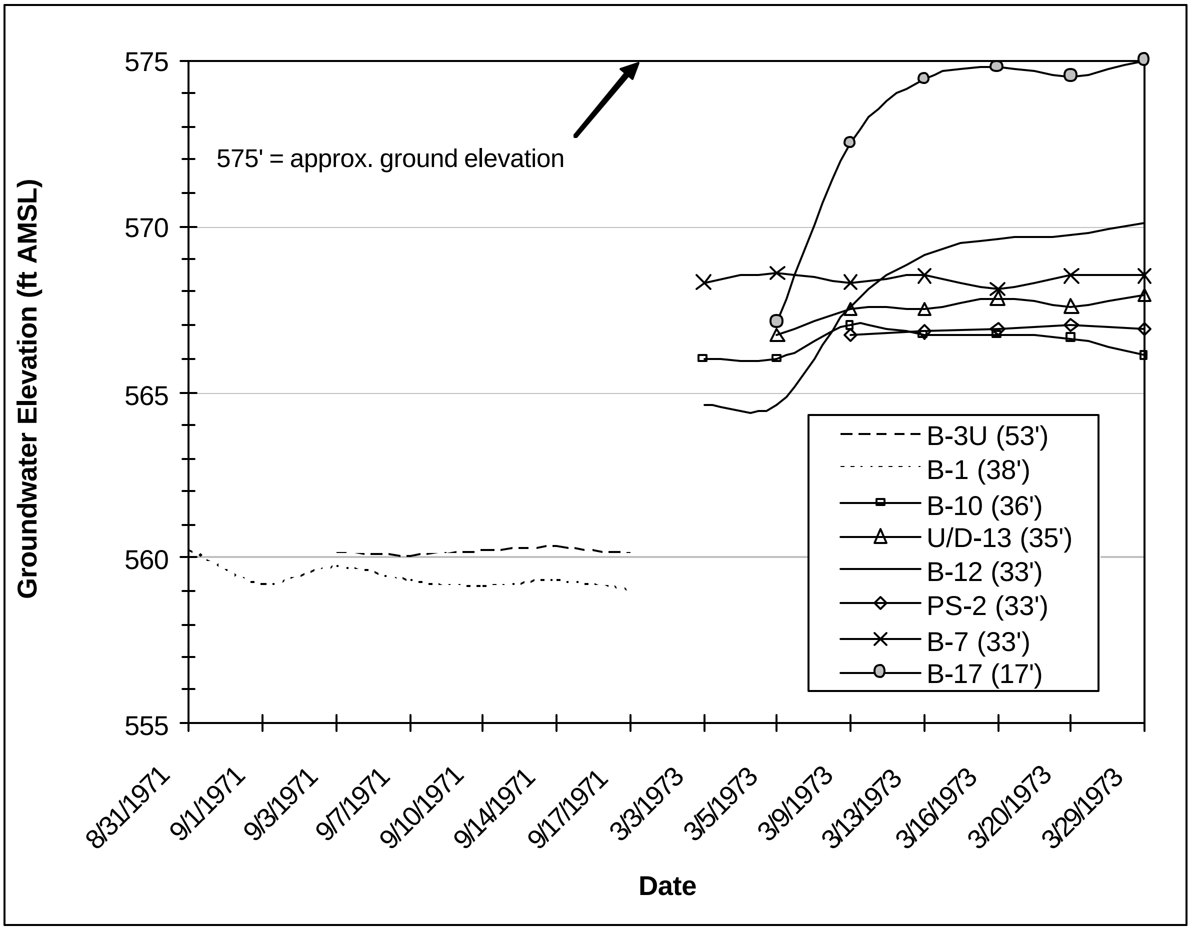

Figure 18. Groundwater elevation at piezometers in till or rock in northwest Amherst, near sewage treatment plant. Note hiatus in dates. Legend shows depth of opening below ground surface. Site is nearly flat with ground elevation ~ 575 ft AMSL. (Data source: Ward, 1971 & 1973).

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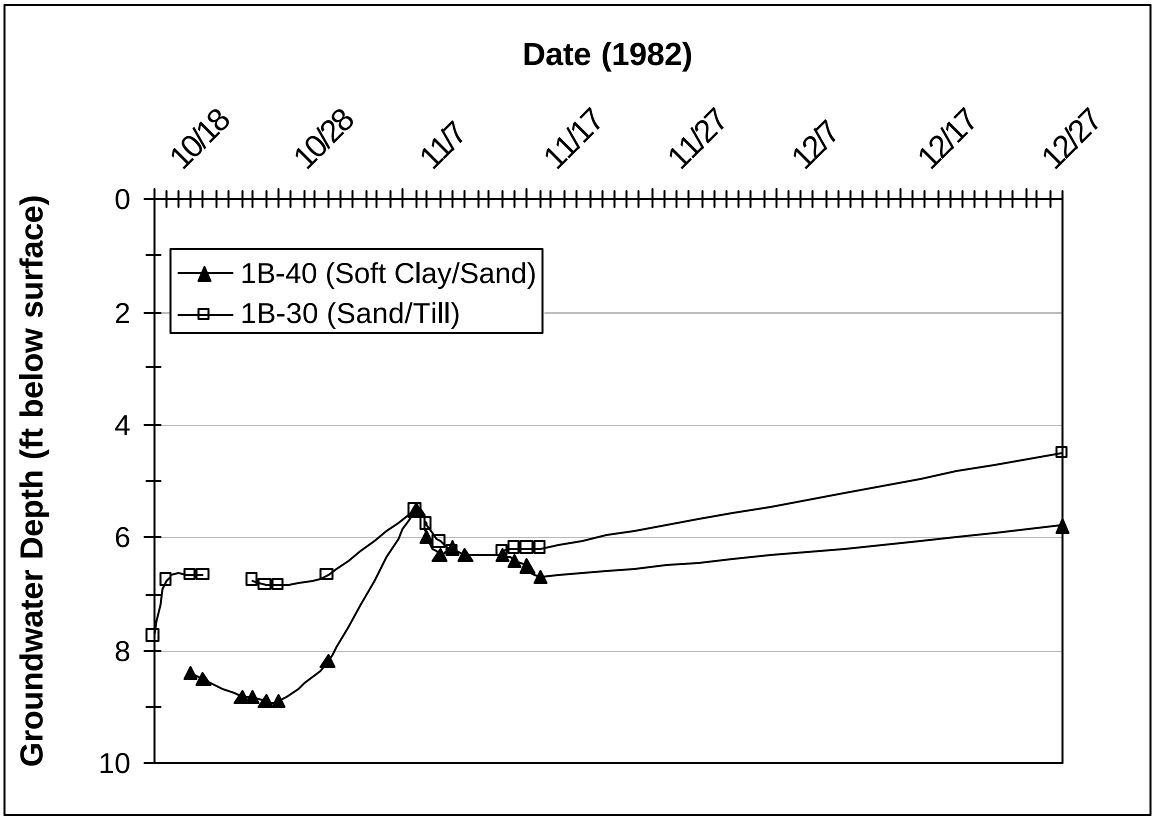


Figure 19. Groundwater elevations at two observation wells along Youngs Road, north of Sheridan Drive (1982). 1B-40 is screened in soft clay stratum transitioning to sand (12 to 20'), and 1B-30 is screened in sand/till (12-20'); both have a total depth of 20 ft. (Data source: Earth Dimensions, 1982).

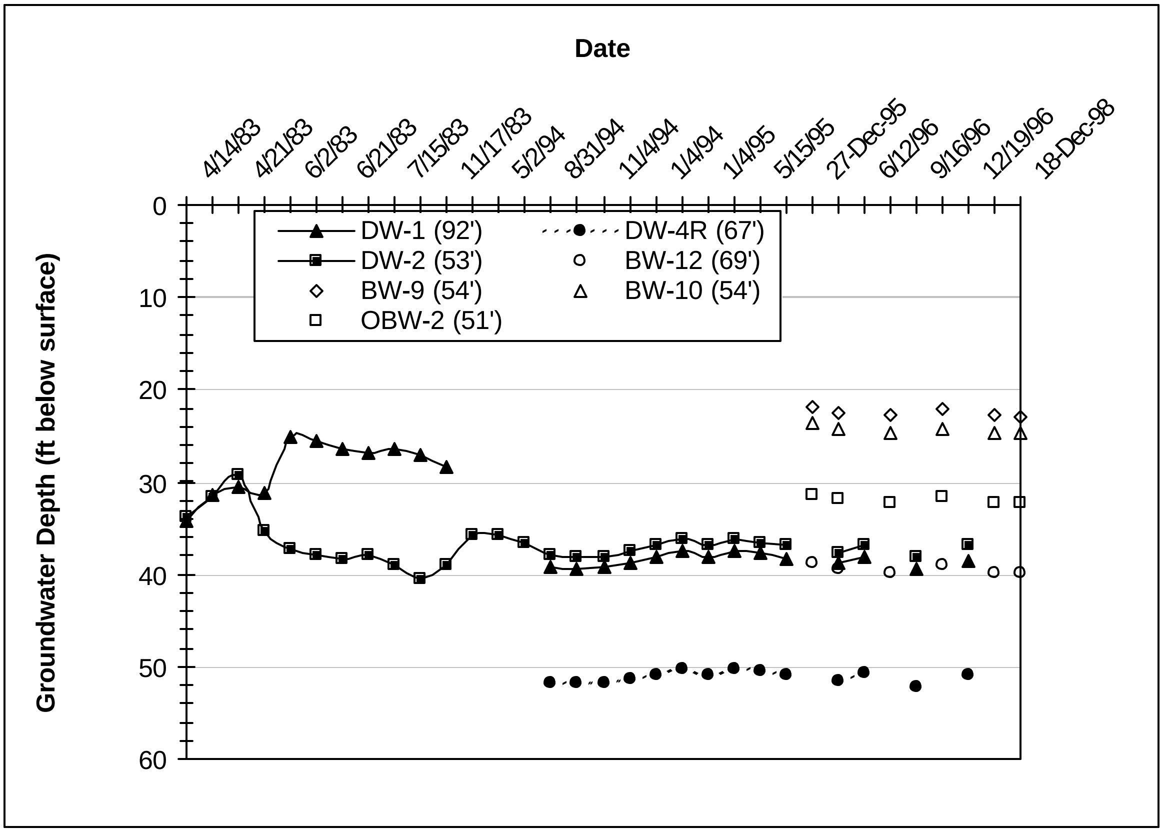


Figure 20. Hydrograph of till (open) and shale (solid) monitoring wells at the Tonawanda landfill. NY. Legend indicates midpoint depth of screened interval (see Appendix 6.4). Note hiatus in dates. (Data source: Glen May, NYSDEC, 2004)

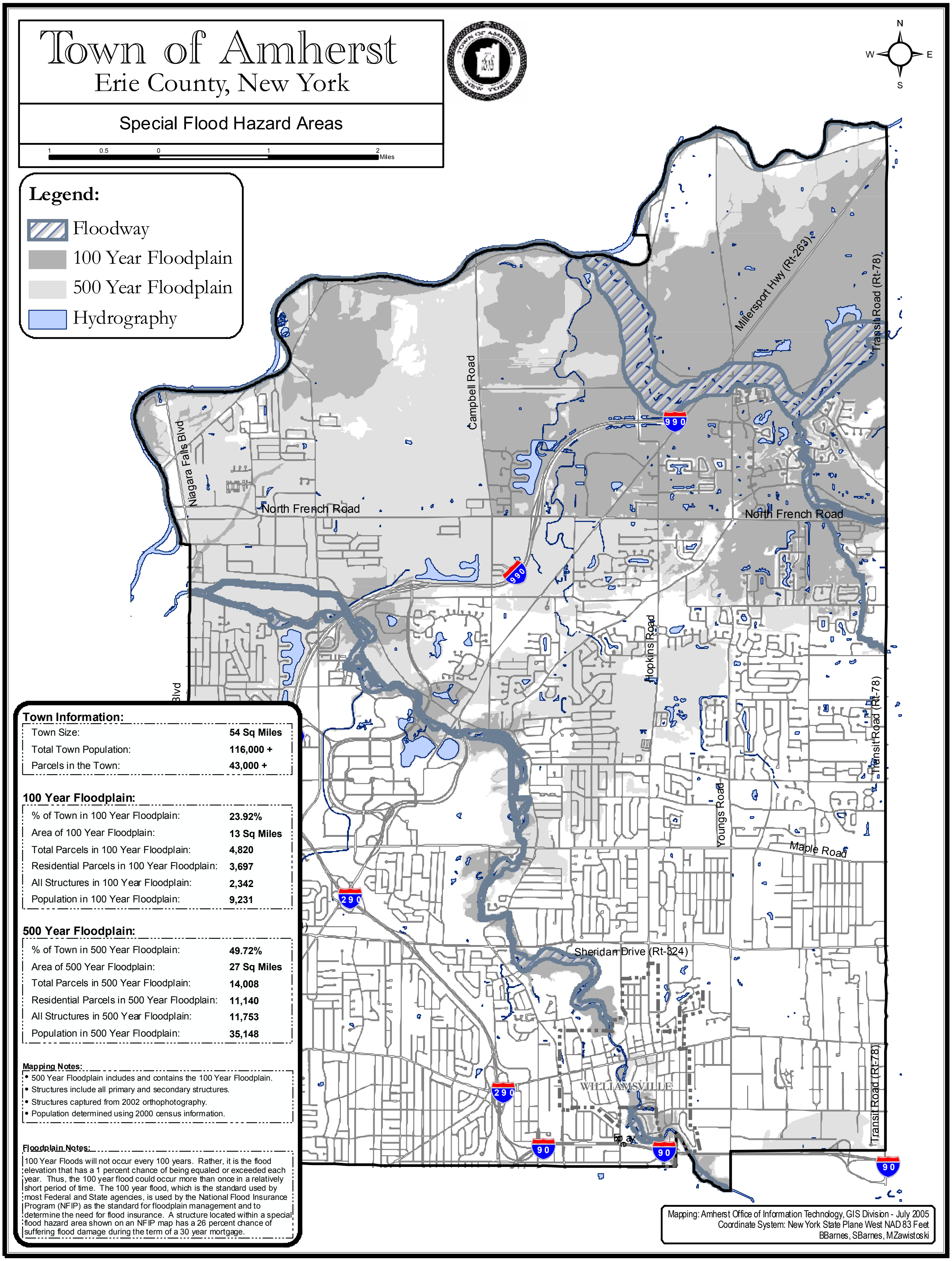


Figure 21: Special flood hazard areas in Amherst, NY (2004).

Table 1. Historical plasticity index data for Amherst and surrounding area.

Location	Data Source	Geologic Material	Sample Depth (feet below surface)	No. of Samples	Average PI	Standard Deviation	Max. PI	Min. PI
Lockport Expressway	McGuffey et al. (1982)	Firm-Soft Silty Clay	---	66	22.2	± 3.0	---	---
Ellicott Creek Flood Control Project	USACE (1979)	Lacustrine Deposit (Ql)[1]	0 – 9	13	27.2	± 6.6	36	13
			10 – 19	33	29.8	± 3.3	33	20
			20 – 25	10	27.8	± 3.9	34	21
		Lacustrine Deposit (Qlt)[2]	0 – 20	10	26.3	± 6.5	34	11
Tonawanda Creek area North	USACE (1973)	---	---	5	22.0	± 3.3	25	17
		---	---	10	24.4	± 3.3	33	20
Amherst Sewage Treatment Plant	Ward & Associates (1971,73)	Silt and Clay (A & B)[3]	5 – 17	16	24.1	± 4.3	29	17

[1] Ql = Well sorted, thin-bedded to massive, red-brown to gray clayey-silt of high plasticity, associated with post-glacio Great Lake (includes CH or CL)

[2] Qlt = Stratified, sorted, sandy silt and sandy clay of low plasticity, associated with Lake Tonawanda (includes CH or CL)

[3] Samples represented are predominantly varved gray brown silty clay or clay and silt, less than 20 ft in depth, and have blow counts between 0-4 (A Clays) or 5-8 (B Clay).

Table 2. Natural and man-made factors affecting soil moisture content

Factors Affecting Soil Moisture Variation in the Subsurface		
Natural/Undeveloped	Man-made/Developed	
Scale of Observation		
Regional	Neighborhood	House-lot
Preciptation Evapotranspiration Temperature Wind Soils Wetlands Floodplain Flood control	Impervious Surfaces (roofs, roads, parking lots) Evapotranspiration Stormwater conveyances (storm drains, ditches) Ponds (detention, retention, recreational) Sewer main (infiltration or leakage) Water main Snow storage Mining Groundwater (withdrawal & injection)	Impervious Surfaces (roofs, driveway, patios, walkway) Trees and schrubs Sump pump Downspouts/storm drain Footing drain Leaky plumbing (water, sewer, garden hose) Yard drainage Lawn irrigation Landscaping/ground slope Dehumidifier Neighbor's runoff Pools French Drain Utility trenches Storage under slab Desiccation cracks Backfill settlement Sun exposure (N-S)

Table 3. Climate data for Buffalo, New York

	Period	Month												Annual Average
		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	
Temperature[1] (°F)	1971-00	25	26	34	45	57	66	71	69	62	51	40	30	48
ETo[2] (inches)	1987	0.46	0.43	0.89	1.69	3.13	4.26	5.31	4.32	2.55	1.51	0.71	0.48	25.7
Precipitation[1] (inches)	1971-00	3.2	2.4	3.0	3.0	3.4	3.8	3.1	3.9	3.8	3.2	3.9	3.8	40.5 ± 5.4
Snowfall[1] (inches)	1884-04	26.1	17.8	12.4	3.6	0.3	---	---	---	0.0	0.3	11.0	25.5	97.0

Data Sources:[1] National Weather Service Buffalo; temperature and precipitation data available for 1940-2004; temperature value rounded to nearest whole number; precipitation values rounded to nearest one-tenth; snowfall maximum 199.4 inches (1976-77) and minimum 22.4 inches (1889-90). [2] ETo (potential evapotranspiration) values for Clarence from Staubitz and Miller (1987), but similar to La Sala (1968) = 24.4 inches for Buffalo.

Table 4. Population growth in Town of Amherst (1950–2000)

	Total Population	10-Year Change (%)	Erie County Population (%)
1950	33,744	--	3.8%
1960	62,837	+46.3%	5.9%
1970	93,929	+33.1%	8.4%
1980	108,706	+13.6%	10.7%
1990	111,711	+2.7%	11.5%
2000	116,510	+4.1%	12.3%

Source: United States Census Bureau

Table 5. Land use changes in Town of Amherst (1972– 2000)

Land-Use Category	1972		1985		2000	
	Acreage	Percentage	Acreage	Percentage	Acreage	Percentage
Residential	7,229	21.2%	8,840	25.9%	12,492	36.6%
Commercial	885	2.6%	1,160	3.4%	1,367	4.0%
Office	65	0.2%	224	0.7%	818	2.4%
Industrial	127	0.4%	453	1.3%	335	1.0%
Public and Semi-public	2,390	7.0%	2,533	7.4%	2,578	7.6%
Recreation and Open Space	2,146	6.3%	2,319	6.8%	3,678	10.8%
Transportation, Utilities, Communications	4,149	12.2%	5,012	14.7%	5,112	15%
Vacant and Agricultural	17,017	49.9%	13,559	39.8%		
Agricultural					1,226	3.6%
Vacant					6,484	19.0%

Source: Town of Amherst, Planning Department

Table 6. Chronology of important building code changes in Amherst, NY.

CODE NAME	EFFECTIVE DATES OF CODE	NOTES
None	Before 5/11/36	
Building Code of the Town of Amherst (also known as the Building Ordinance)	5/11/36 to 7/4/77	This Building Code was supplemented by "The Fifth Edition of the Building Codes recommended by the Board of Fire Underwriters". The Fire Underwriter's code was deemed to be the generally accepted good practice for conditions, details and subjects not covered in the Building Ordinance.
The "State Building Construction Code"	7/5/77 to 12/31/83	This was a non-mandated New York State Building Code. The adoption of the code was voluntary.
State Uniform Fire Prevention and Building Code	1/1/84 to 12/31/02	The adoption of this code was mandatory for all municipalities throughout New York State with the exception of New York City
1) Building Code of New York State (NYS); 2) Residential Code of NYS; 3) Fire Code of NYS; 4) Plumbing Code of NYS; 5) Mechanical Code of NYS; 6) Fuel Gas Code of NYS; and 7) Property Maintenance Code of NYS.	1/1/03 to present	The adoption of these codes was mandatory. They are based upon the International Codes and are modified with New York State Enhancements.

Photo 1. Varved clay from excavation near Millersport and Transit Roads. Sample characteristic of lacustrine material in Lake Dana-Lundy (D'Agostino, 1958). Laminated bands thought to indicate annual cycles of deposition from summer (pink) to winter (gray).

Photo 1. Desiccation cracks in backfill near basement wall in central Amherst, NY, in July 2004.

SECTION 2 – SCOPE AND EXTENT

2.1 OVERVIEW

The town of Amherst building department estimates the total number of damaged houses to be the sum of the foundation-related repair permits (501) and foundation inquiries (594), which totals 1,095 in March 2005. The total number of foundations in the Town is estimated to be between 45,000 and 31,000, depending on certain assumptions, thus the town-wide minimum and maximum rate of occurrence is about 2.4 to 3.5 percent. The average damaged home was built in 1964 (41 years old). The average repair cost was approximately $7,900.

These data and approach have several recognized limitations that will be examined. We augment these estimates with data from a phone survey, home inspections, and field inspections. We also present some related findings from a remote sensing project, interviews, and we briefly discuss associative damages, foundation repairs, and multi-family structures.

Note, for privacy considerations we do not provide the names and addresses of participants in this report.

2.1.1 Phone Survey

More than 150 homeowners volunteered for a home inspection following our solicitation to certain neighborhood groups through the media. From these, we had 70 or more screening conversations and eventually conducted 15-minute phone surveys with about 52 homeowners. We requested information about residency, location, age, style, wall construction, onset of problems, utility problems, drainage, damage characteristics, crawl spaces, leaking, door and window problems, sump pump operations, repair estimates/cost, and related topics. Most homeowners could not answer every question because, for example, they are recent owners or certain details were handled by a spouse, etc. These data were primarily used to select potential home inspection sites, however, we use some statistical summaries as supporting information. Note, the phone survey and home inspection data have common participants and all respondents had damaged homes; therefore, it is not a random sample.

2.1.2 Home Inspections

The Corps' Inspection Team, consisting of a hydrologist, geotechnical engineer, structural engineer, inspected more than 43 single- and multi-family structures during the summer of 2004. The homes were selected to represent a range in geographic areas, ages, construction types, failure modes, and repair histories.

The Corps Team inspected the interior basement, exterior perimeter, and relevant historical records such as repair estimates, photos, etc. Blueprints of most homes were provided in advance by the Town's Building Department. The inspections ranged from reconnaissance-level surveys to detailed inspections, depending on basement conditions (e.g., wall visibility, access to crawl spaces). Detailed inspections lasted two to three hours. We recorded 40 or more observations, used a laser level to determine differential foundation movements, and usually took a soil sample. For their participation, homeowners received a verbal summary of the inspection results.

The majority of the inspected homes were two-story structures (79%), with an attached garage (56%). Basement wall construction was either cast-in-place concrete (70%) or concrete masonry units (CMU, 30%). Problems associated with detached and peripheral features such as a stand-alone garage, patio, decks, driveways walkway, and gazebo received less attention than basement problems. Inspection results are discussed in relevant sections.

2.1.3 Field Inspections

Team members made bi-monthly field visits to Amherst neighborhoods throughout most of 2004 for such purposes as inspecting new house construction, observing stormwater drainage, soil sampling, and to interact with homeowners and contractors.

2.2 Town Data

Prior to March 2005, the Town Building Department had two reporting categories that indicated foundation-related problems; these were foundation repair permits and complaints. After March 2005 and during the writing of this report, we added a third category called "assessment reviews," which includes houses whose assessed value was reduced because of foundation related damages. We now combine complaints and assessment reviews into one category called "foundation inquiries," in part, because some complaints were actually concerns or inquiries. Some parts of this report use the older terminology.

Figure 22 shows the number of foundation repair permits and complaints recorded by the Amherst Building Department through January 2004. The number of repair permits increased sharply in the early 1990's, sometimes catalyzed by dry conditions and increased media coverage. The Building Department established an inspection and tracking system for foundation-related complaints in 2003. Figure 23 shows the spatial distribution of these sites (maps available from Building Department). The majority of permits and inquiries are located north of Sheridan Drive, with the exception of houses in southwestern Amherst.

These data are imperfect but are the best available. The clustering of data on Figure 23 is influenced by several intangible factors that include:

- Social culture -- some neighborhoods openly publicize and discuss their foundation problems;
- House density – some areas have many times more foundations per acre than other neighborhoods (e.g., condo);

- Non-residential development – many areas of the town are zoned for uses other than residential – i.e., industrial, commercial, open space, wetlands, etc.;
- New development – new development generally has few reported problems;
- Soils – non-lacustrine soil areas generally have fewer problems;
- Construction – some areas have older homes that used CMU foundations.

In addition, some inquiries involve minor or peripheral problems such as a chimney, porch, patio, driveway, walkway, or normal shrinkage cracks. A small number of homeowners perform repairs without a permit. Some foundation repair permits are for "normal" home maintenance/improvements. In short, the clustering of data should not be overly interpreted, in fact, most residential areas had at least one or more reported cases of foundation damage.

Finally, the total number of foundation/basements is an estimate. The total number of "households" in 2000 was cited as 45,076 (Amherst IDA, 2005). The Building Department often uses the 43,000 identified parcels in town. We use, perhaps conservatively, the assessment parcel code (from NYS Office of Real Property Service) to identify residential parcels (code 200 series) and a subjective criterion of 600 square feet (minimum house dimensions) to query out structures that likely have a basement. The total number of parcels that met these criteria was about 31,000. We believe this approach provides a reasonable estimate of the actual number of foundations in the town.

2.2.1 Spatial Patterns

The spatial relationship between foundation repair permits/inquiries and lacustrine soils, surficial geology, flood plains, and primary causative factor (lateral pressure or settlement) is examined in this section.

The spatial pattern and severity of foundation damages on a neighborhood scale can be quite irregular. The pattern is akin to earthquake or other natural disaster damage. In only a few places are the damages easily observed from the exterior. It can affect any style of house, a cluster of houses, and a severely damaged structure can be 10 feet from an undamaged structure in the same soil. For example, we inspected six similar aged houses on a cul-de-sac in north Amherst that had different architectural styles and builders. Of the six houses, two had moderate to severe damage, two had some or moderate damage, and two were undamaged.

Figure 24 shows the relationship of foundation repair permits and complaints to the five soils types described in Section 1.5.6.2. Table 7 shows these five soils types account for 42% of the town area, 48% of the total number of foundations, and account for 75% of the foundation repair permits and 82% of the complaints.

We then subdivided the 470 complaints into cases of lateral pressure (254) and settlement (216) based on a Town Inspector's diagnosis. In addition, we reviewed and subdivided 213 foundation repair permit cases (2001-03) into lateral pressure (110), settlement (72), both (20), or undetermined (10). These results were re-plotted on the lacustrine soils, however, no definitive pattern emerged. There was a weak association between older neighborhoods, which often used CMU construction, and lateral pressure damage.

Figure 25 shows the relationship of surficial geology units and foundation repair permits and foundation inquiries. The geologic units do not appear to be a good predictor of foundation-related problems.

It is interesting that houses with settlement problems occur in areas that generally do not have an underlying soft stratum (c.f., Figures 6 and 23). This might suggest the importance of shrink/swell behavior as the primary causative factor in settlement.

Figure 26 shows the location of foundation repair permits and complaints in relation to the 100- and 500-year flood plain. Foundation-related problems are both within and outside the flood plain boundary. Potentially interesting, is the near coincidence of the floodplain boundary and soft stratum areas (c.f., Figure 6).

2.2.2 Rate of Occurrence

Table 7 shows that when complaints and foundation repairs are normalized by the number of foundations, no particular lacustrine soil type is more problematic than another. The rate of complaints and foundation repair permits on lacustrine soils averages about 2.9 and 2.4 percent, respectively. This estimate of the damage rate generally excludes homes on or near the escarpment.

Nonetheless, the single-digit rate does not reflect the much higher rate we observed or heard described in some affected areas. The Corps team interviewed homeowners who track foundation damages on their street, cul-de-sac, or neighborhood. We promised anonymity and defined "damage" as clusters of homes having or needing an average $10,000 or more in repairs. Some rates of damage from central and northern Amherst are summarized below:

- 12 of 24 homes damaged in cul-de-sac "A"
- 40 of 95 in neighborhood "A"
- 26 out of 49 homes, 8 of 10, and 24 of 44 are three estimates from neighborhood "B"
- 60 of 1,300 in neighborhood "C"
- 4 of 6 in cul-de-sac "B"
- 6 of 16 condominiums in neighborhood "D"

These local estimates are an order of magnitude or more greater than town-wide estimates and suggest that some areas are seriously affected. In one hard-hit development, we observed and estimated a 25 percent damage rate. Rarely did the data on the foundation repair permit/inquiries map (Figure 23) indicate the actual number of damaged houses that homeowners could cite from their driveway perspective. This discrepancy may reflect the reluctance of homeowners to report damages to the Town.

In summary, we judge that the number of repair permits will increase and may someday total as many as 2000 houses, but the timing depends on several less predictable factors (e.g., climate, funding).

2.2.3 Age of Damaged Homes

Figure 27 shows the number and the age class of houses that received a foundation repair permit since 1987 (Town data). The average house was built in 1964 ± 15 (1s) but ranges from 1887 to 1996, thus the mean age is about 41 years old (n=501). Considering houses built after 1950, the average age drops to 36 years (n=444), and the elapsed time from house construction to foundation repair permit is 30.6 ± 9.9. These statistics are not particularly meaningful because they are biased by the total number of houses built (different for each decade) and foundation repair permits were not issued prior to 1987.

Twenty-eight homeowners in the phone survey knew the age of their home and the year they first noticed problems. The average age of these houses was 1970 ± 6 years (1s) but ranged from 1954 to 1983. The average number of years without a problem was 24 ± 11 (1s) years, with a range of 3 to 47 years. Similarly for the houses we inspected, the average house was built in 1972 ± 9 years (1s) but ranged from 1950 to 1985 (n=39). The average number of years without a problem was 19 ± 12 (1s) years, with a range of 5 to 48 years (n=12). We speculated the onset of damage would not generally coincide with the date of the foundation repair permit because homeowners appear to tolerate incremental damage for many years, require time to prepare financially, or are unaware of problems for several years because of wall coverings, but these results suggest the difference is relatively small.

2.2.4 Repair Costs

The repair cost provided on the permit application can be misleading. Sometimes the eventual cost is much greater than the initial estimate, and some homeowners make incremental repairs, addressing the most affordable or urgent repair first, so the total cost is not reflected on the initial permit request. Accounting for multiple permits situations (but not inflation), the average repair cost is about \$7,921 ± \$8,440 but ranged from \$450 to \$71,000 (n = 501).

In our phone survey, 29 respondents knew their total repair costs or had a recent repair estimate. The average repair cost was \$23,700 ± \$20,300 (1s) but ranged from \$1,000 to \$80,000 (the median cost was \$17,000). This relatively small sample suggests the average repair cost is somewhat greater than repair data suggests.

2.3 REMOTE SENSING

The University of Buffalo's Earth Sciences Remote Sensing Lab was tasked with applying space-based radar interferometry techniques to determine and delineate long-term surface elevational changes in the Amherst area (Sultan and Becker, 2005). The research question: Is there evidence of long-term neighborhood-scale subsidence? These techniques are routinely used to detect basin subsidence resulting from groundwater overdraft in the Southwest. UB used two techniques in the exercise including the 3-Pass DINSAR and the Small Baseline technique. Several interferograms were unwrapped, with the best results coming from two interferograms spanning one of the dryer periods in western New York (1992-95).

Preliminary results show that they were able to observe topographic changes in the Amherst area (Appendix 6.2). One area of interest is between Maple and Sheridan, where local differential surface deformation is suggested. At this point, however, and with the limited budget and time we had to fund this research, the results are not conclusive.

2.4 RELATED FINDINGS

2.4.1 Interviews

Interviews with homeowners, contractors, town officials, and others provided several clues regarding the scope, extent, and causative factors. A selection of representative statements are paraphrased below, again with the author's identity obscured for privacy considerations.

"When we poured concrete back then [1970s], especially in the summer, we had to water it down to push it to the back wall with our shovels -- also, because footings were not surveyed the way they are today, sometimes the wall didn't center on the footing – and sometimes, the footing forms contained loose sediment that was simply blended into the concrete." *Building Contractor*

"During one very dry summer, several homes in my neighborhood experienced problems at nearly the same time… nearly on the same weekend." *Homeowner*

"My cracks widen in the summer and close in the winter… but they didn't do it this past [2004] summer, it was really wet…" *Homeowner*

"I have a crawl space under my family room and it is settling, but the rest of my home is pretty good." *Homeowner*

"Sometimes the soil around the excavated footing is so dry that we have to use jack-hammers chip it away… and sometimes you can place your hand between the footing and the base of the wall" *Repair Contractor*

"No matter how much dirt I put on it that low spot in the yard, it seems to keep settling." *Homeowner*

"I had one engineer say I should pier my replacement foundation, but the design engineer said a wider footing was sufficient… what should I do?" *Homeowner*

"See that… [shallow roots in basement excavation], that's the problem…" *Repair Contractor*

“I have a fairly new undamaged home, but my brother lives in Amherst and he has an older damaged home… should I relocate to Clarence?” *Homeowner*

“In 18 years I have had some cracks, suspected settlement, 11 piers… about $22,000 worth… and I re-repaired some leaking cracks that are worse in dry weather… the porch settled…door problems…poor drainage in yard…garage floor settled.” *Homeowner*

2.4.2 Associative Damages

We observed and took reports of damage to several features, many peripheral to a house, which included utilities, downspouts, basement slabs, doors and windows, drywall, and exterior flatwork (porch, driveway, garage, walkway, deck, and patio). This section presents some brief comments about these problems.

For instance, many homeowners report damages related to utilities. About 20 of 50 respondents in the phone survey indicated they have repaired their gas (6), electric (7), water (2) or sewer (7) connections. Some respondents had more than one repair. One affected neighborhood researched their water/sewer breaks and found 40 of 95 houses had water main breaks (1989-00), 26 sewer line fractures, and there had been numerous electrical box repairs and maintenance problems. It is difficult to determine from Town inspection records whether the pipe or the house is shifting. Settling of backfill in box-cut trenches is common.

Representatives from the gas, phone, electric and water utility companies and the Town’s Plumbing and Engineering Departments were asked if abnormal rates or a pattern of complaints or maintenance problems were evident in Amherst. The companies and departments generally do not see an unusual pattern, but sometimes the maintenance records are not easily queried or are not mapped. The Town’s Plumbing Department may provide the best opportunity to track water and sewer problems at the individual home level because they inspect repairs.

Problems with downspouts are very common. Nearly 24 of 33 respondents have rebuilt, repaired, snaked, and/or extended portions of the downspout drainage system. Clogging is usually caused by tree roots, debris, or collapse. In a few cases the clog causes the sump pump to recycle water that erodes and saturates the soils along the foundation. Photo 3 shows a typical downspout system. The segmented pipe is vulnerable to frost, construction damage and soil subsidence along the exterior wall.

About 34 of 45 respondents in the survey reported minor to severe cracking of the basement floor slab. Alternatively, the Town’s complaints data show a “basement floor” damage rate of about 12 percent; these sorts of discrepancies are more a function of different samples, procedures, and terminology. Basement floor slab cracking can occur for several reasons. Four of the five slabs we cored were significantly thinner than the blueprint called for (construction issue). The underlying crushed stone thickness varied from one to three inches. Furthermore, the majority of houses in that era did not have control joints to aid in random crack prevention. Control joints create predetermined lines of weakness in a slab. These predetermined lines of weakness provide a location for tensile stress relief in the slab. We did not test the concrete strength of basement floor slabs.

Binding or inoperable doors and windows, distorted glass panes, and wedge shaped gaps at the top and bottom of doors and windows is a common complaint of homeowners, especially with settling homes. Approximately 22 of 40 surveyed indicate they had some to severe problems. During inspections, 28% of homeowners described sticky doors and 16% inoperable windows. Distinguishing normal aging and cyclical swelling from foundation-related damage can be a challenge.

About 30 percent of the inspected houses had drywall cracks. Most drywall cracks appear in the corner of doorframes or windows and result from differential movement between the framing and the drywall. Some wood frame movement can be caused by normal processes such as shrinkage or temperature expansion. Approximately 7 of 19 settlement cases we inspected had drywall cracks.

Damage to exterior flatwork (e.g., driveways, sidewalks, patios, garage slabs, and porches) is common. Cracking of concrete can have a variety of causes including swelling soils, concrete shrinkage, settling, frost heave, tree roots, and poor quality of concrete or installation. We examined many front steps, porches, and decks that had been settling with the backfill for several years. Often the flatwork slopes toward the house and desiccation cracks channel water against the basement wall.

Houses with crawl spaces showed a recurring damage pattern. Often a few vertical cracks in the crawl space open and close seasonally. In addition, the fireplace on the terminal end separates slightly from the exterior wall. Among several potential factors, we speculate these shallow footings rest on expansive soil that experiences more acute cycles of shrink and swell, which is often aggravated by landscaping.

2.4.3 Foundation Repairs

We did not explicitly investigate foundation-related repairs, however, nearly a third of the houses we inspected had either been repaired or had a repair estimate. Repairing damaged foundations probably represents the greatest engineering challenge associated with this problem. While the majority of homeowners were satisfied with their repairs, a significant number had repairs that subsequently failed.

Our preliminary observations coincide with Anumba and Scott (2001), who investigated a rash of subsidence problems in the UK in the 1980's and 1990's. They determined that *effective diagnosis and repair of subsidence damage requires considerable experience, skill, and engineering judgment*. We speculate that conditions in Amherst are more complex than in the UK. Our limited experience revealed there is occasional erroneous diagnosis and subsequent implementation of an inappropriate remedial measure. For example, we observed pilasters that were improperly supported, repaired walls that promoted subsequent settlement, the misapplication of carbon fiber strips, and the engineering conviction that wider footings prevent settlement. Most homeowners are not monitoring the situation adequately, and they negotiate directly with contractors without the assistance of a geotechnical or structural engineer. We also observed reputable engineers' design solutions that did not alleviate the settlement and/or lateral pressure problems. Conversely, some "home grown" repairs (e.g., screw jack in crawl spaces and steel braces across fractures) appeared to perform quite well.

A summary of our observations of the deficiencies related to repairing foundations include (see Anumba and Scott, 2001):

- inconsistencies in diagnosis due to the complex interaction between the causative agents;
- lack of systematic inspection/appraisal procedures;
- inexperience and lack of knowledge on the part of investigators;
- inadequate site inspection by the lead engineer;
- insufficient description of monitoring, maintenance and repair options.

2.4.4 Multi-Family Structures

Many multi-family apartment buildings are built with basements and are experiencing foundation damage. We did not inspect the interior of these buildings but observed the exterior of more than 20 buildings. These two-story, often brick veneered structures showed significant lateral pressure damage and some settling. In extreme cases, the brick veneer has fallen away and been replaced. Photo 4 shows an apartment complex with typical damage in south-central Amherst. These damages never appear to be dangerous to occupants, nevertheless, owners are reluctant to discuss their repairs.

2.5 Summary

The Building Department's data represents a starting point for determining rates of occurrence, age, and repair costs of damaged homes. These values will likely change as more information is gathered and potential funding becomes available. The actual damage statistics are unknown without a statistically valid homeowner survey. This approach has not been tried by the Town or Corps because many homeowners are reluctant to provide information that could become public and potentially affect their property values.

The current number of foundation repair permits and foundation inquiries (former complaint and assessment reviews) is 1,095. Assuming the number of foundations is 31,000, then the town-wide damage rate is three to four percent. In affected areas, the rate can be an order of magnitude greater. By way of comparison, two relatively large upstate New York towns, Colonie and Greece, report between one and five foundation-related repair permits per year, as opposed to 40 or more in Amherst (pers. comm., Colonie and Greece Building Departments, 2005).

We judge the eventual number of repair permits will increase and approach 2,000, maybe within a decade, depending on several unpredictable factors (e.g., climate, funding). We base our estimate on the body of evidence gathered in the phone survey, home inspections, field inspections, and from town data.

In addition to foundations repair costs, homeowners also face many non-foundation expenses associated with these soil conditions. Diagnosing and repairing foundation damages represents a real challenge for homeowners and engineers, as no "magic bullet" repair solution has been identified at this time.

2.6 Figures, Tables, Photos

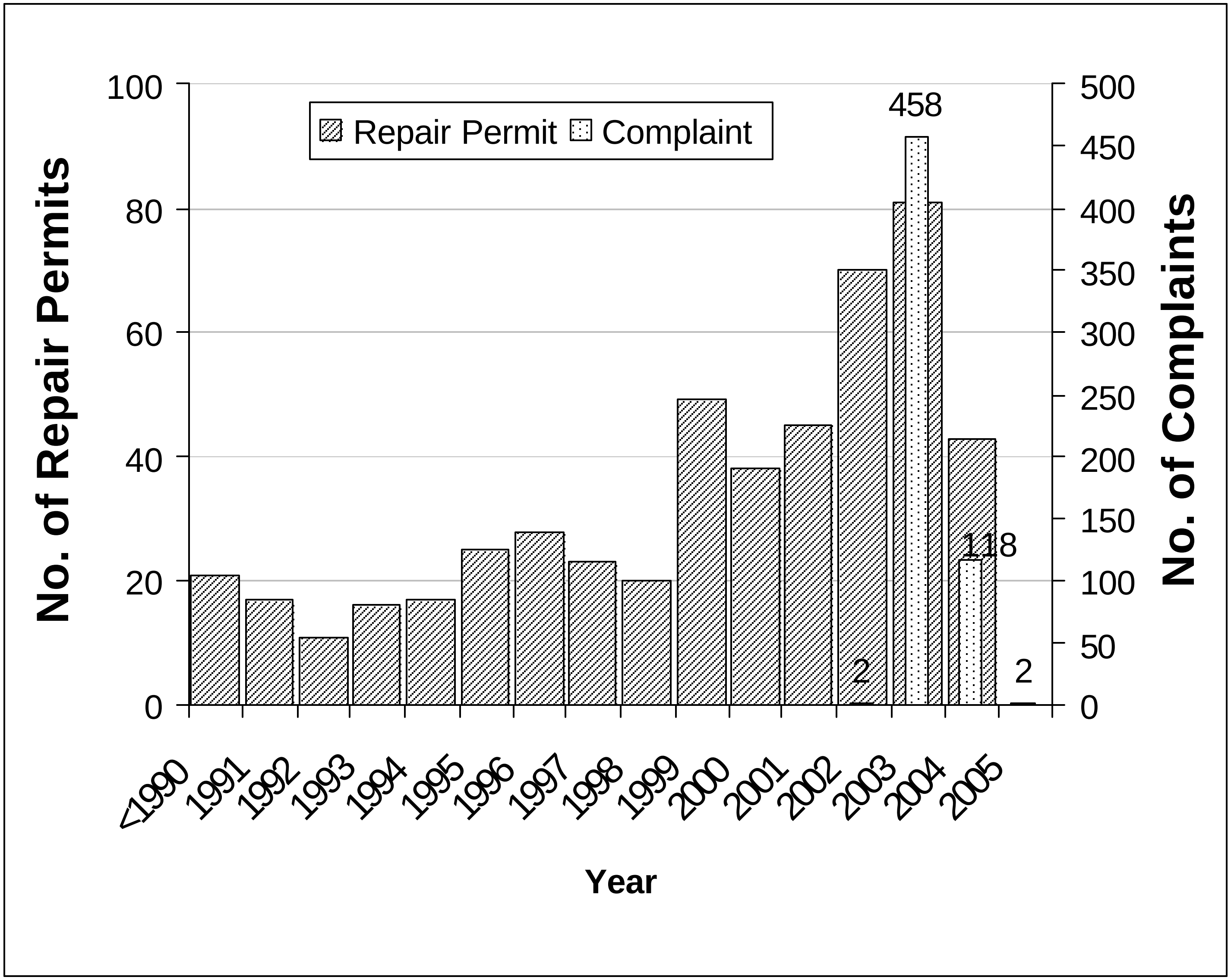


Figure 22. Frequency of foundation-related repair permits and complaints in Amherst, NY, through January 2005.

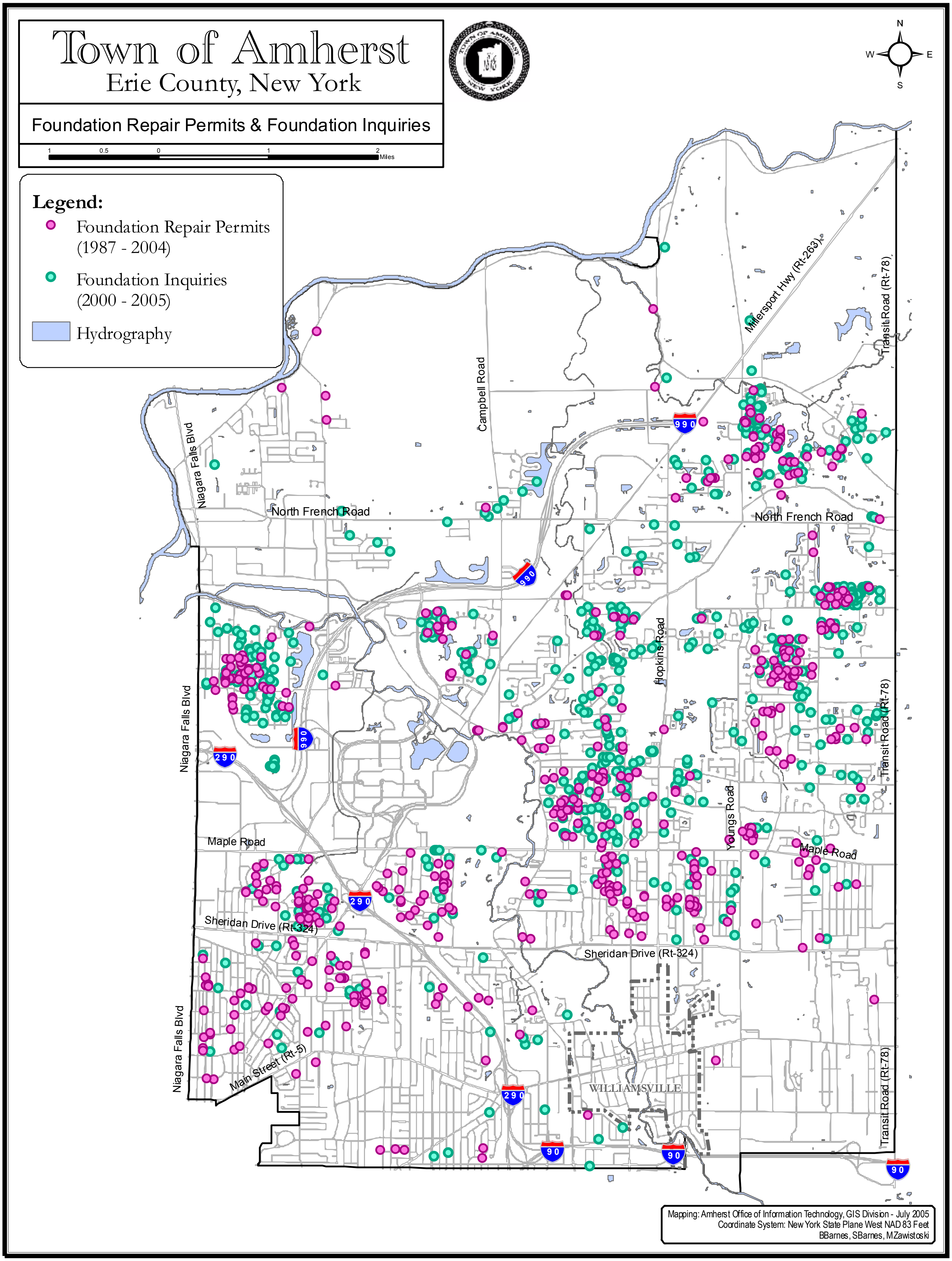


Figure 23: Location of foundation repair permits (501) and foundation inquiries (594) in Amherst, NY, through March 2005.

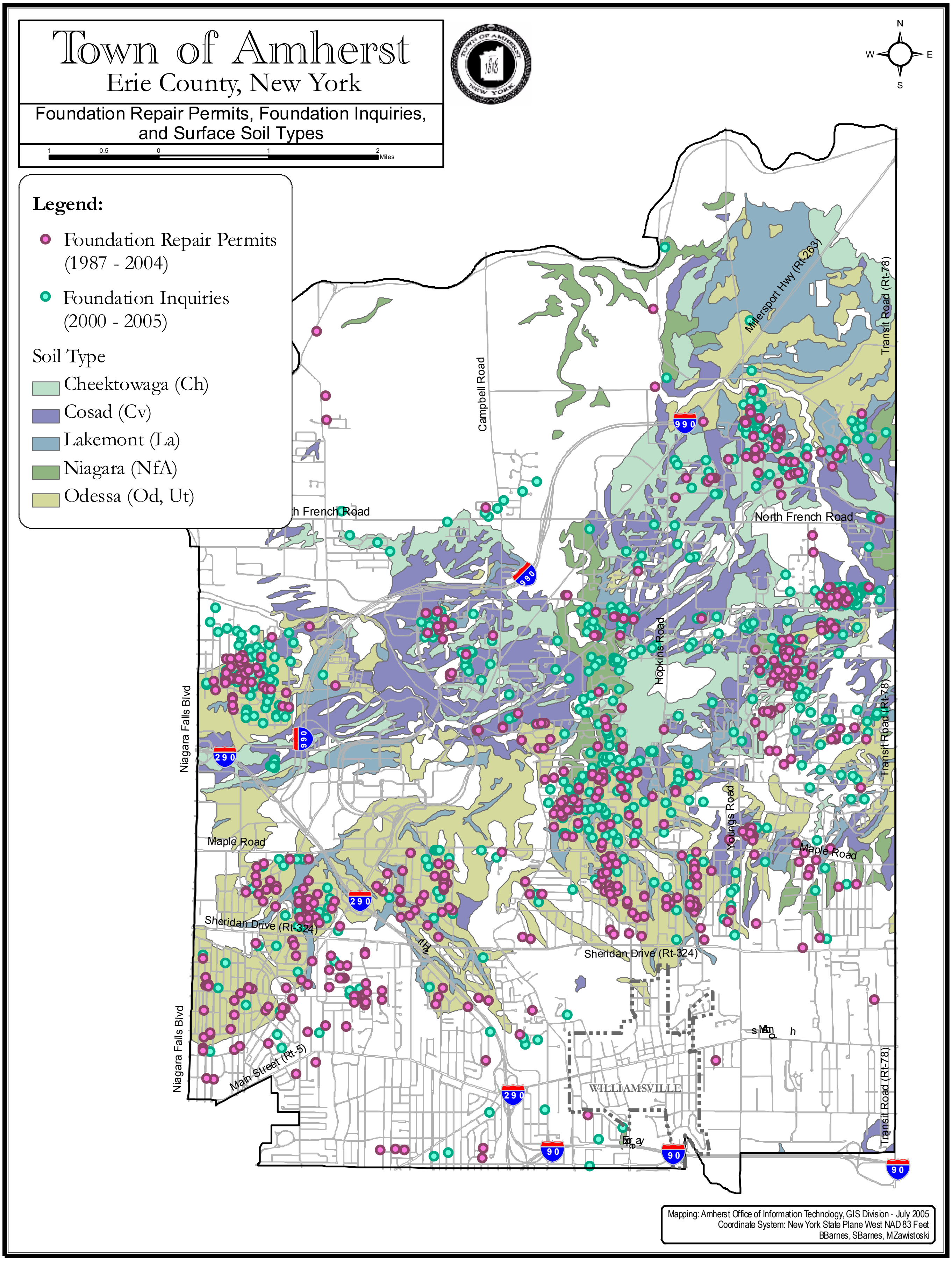


Figure 24: Relationship of foundation-related repair permits and inquiries to five lacustrine surface soils in Amherst, NY.

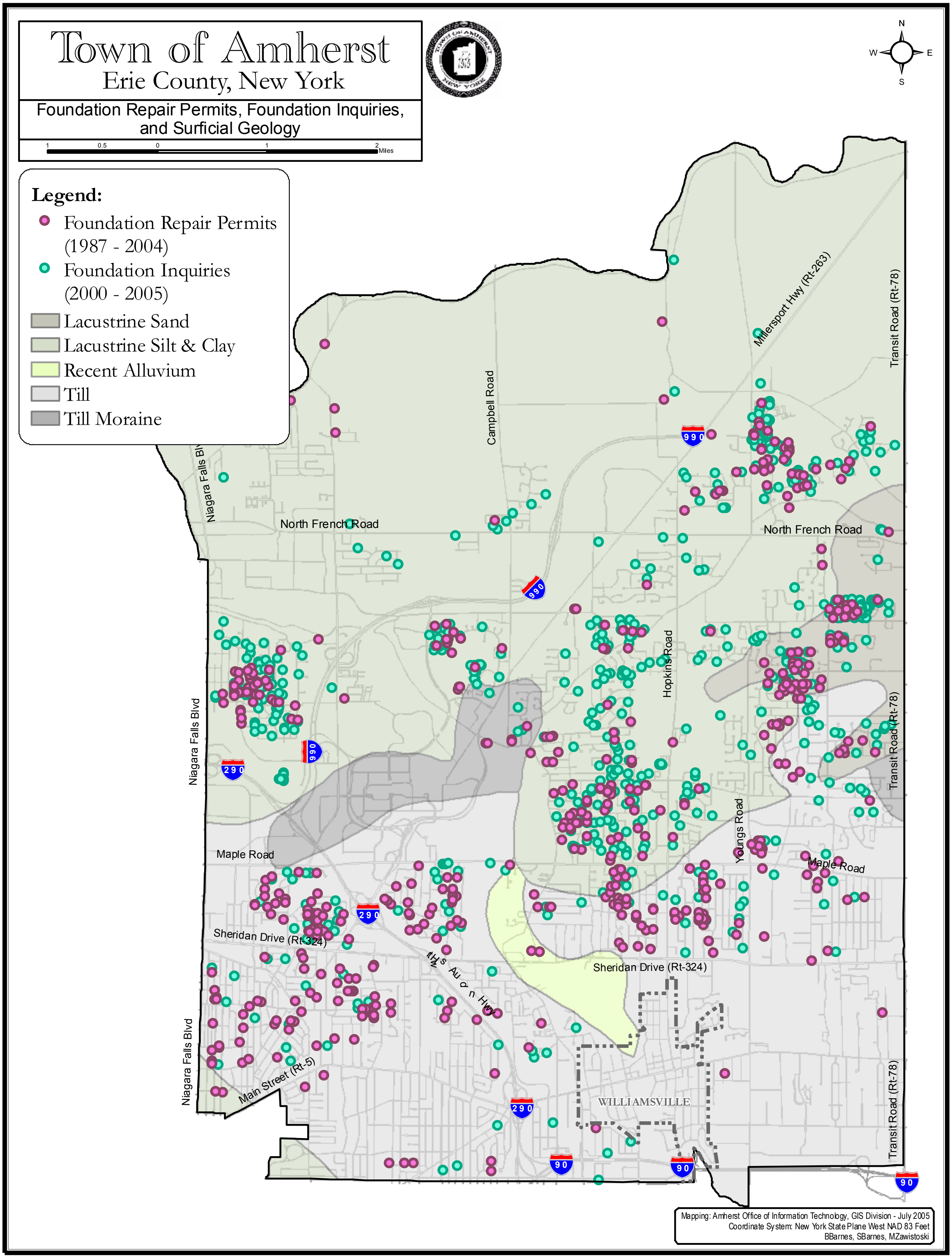


Figure 25: Relationship of foundation-related repair permits and inquiries to surficial and bedrock geology.

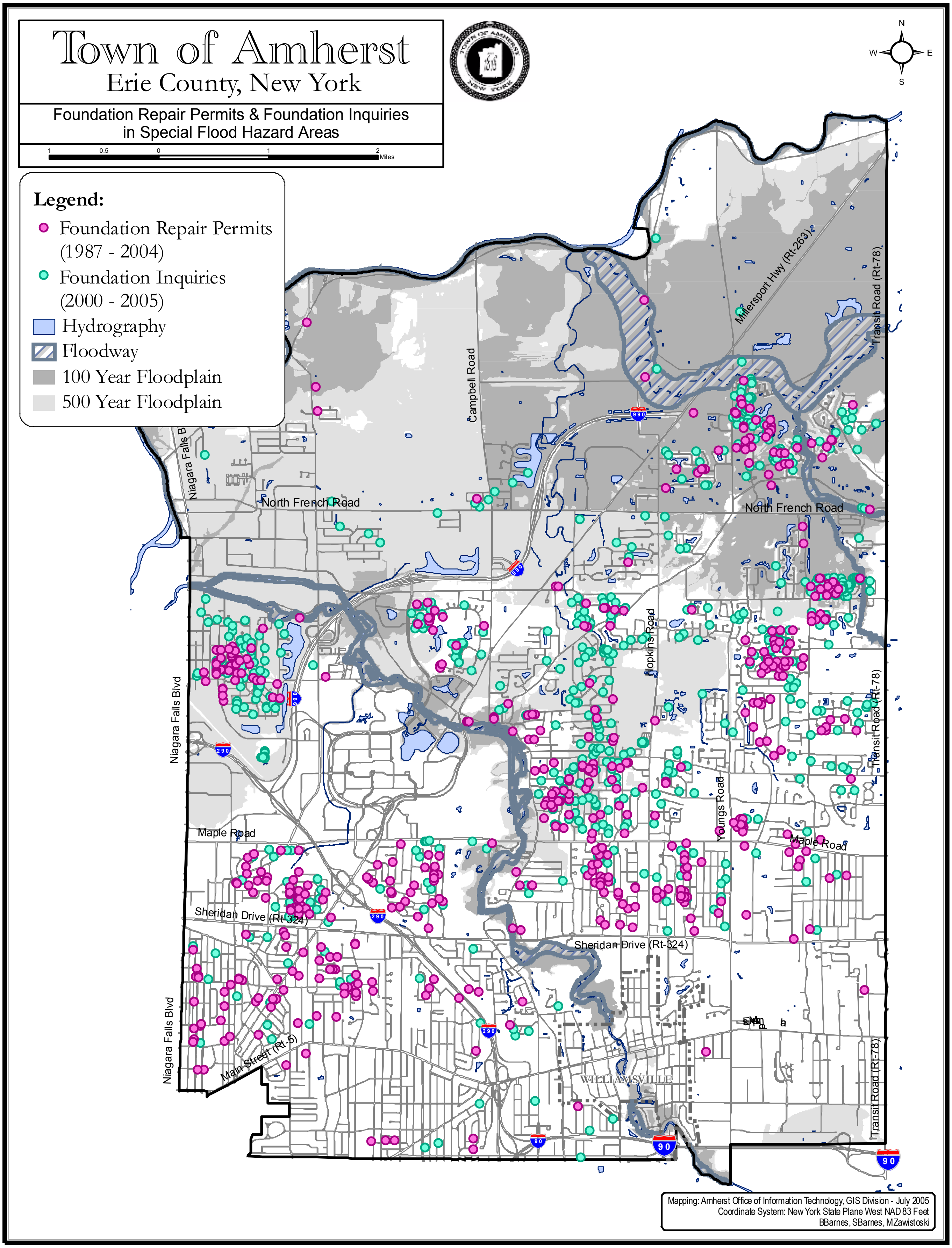


Figure 26: Relationship of foundation-related repair permits and inquiries to most recent special flood hazard areas. Note the floodplain maps for Amherst have been revised several times since 1977, and the 1984 map contained a much larger 100-year floodplain around Ellicott Creek before the Corps completed its diversion channel project in the late 1980s.

Figure 27. Number of repair permits issued by age class.

Table 7. Rate of occurrence of foundation-related repair permits and complaints on lacustrine soils in Amherst, NY.

Soil Name	Symbol	Town-wide Area %	No. of Foundations Total (%)	No. of Homeowner Complaints[1]	No. of Repair Permits[1]	No. of Complaints/ No. of Foundations %	No. of Permits/ No of Foundations %
Cheektowaga	Ch	8	2,105 (7)	72	58	3.4	2.8
Cosad	Cv	10	2,705 (9)	90	43	3.3	1.6
Lakemont	La	6	1,263 (4)	34	39	2.7	3.1
Niagara	NfA	3	1,211 (4)	36	14	3.0	1.2
Odessa	(Od & Ut)	15	7,443 (24)	152	233	2.0	3.1
Subtotal			14,727	384	387		
Town-wide Total			31,000[2]	466	516		
Town-wide %		42 %	48 %	82 %	75 %	2.9 %	2.4 %

[1] These are town data from May 2004 and may slightly different than totals reported elsewhere in this report. [2] Estimate based on parcel code and minimum footprint of 600 sq. feet.

Photo 3. Downspout drainage system discharging to rear yard collector in north Amherst , NY (June 2004). Damage from construction, frost, soil subsidence, and clogging are common reported problems.

Photo 4. Lateral pressure damage to multi-family apartment complex in south central Amherst (June 2004). Basement walls have corner cracks (patched), and corner block is rotated out. Perimeters soils have settled and pitch into basement wall,; down spouts are often extended and step is settling.

SECTION 3 – CAUSATIVE FACTORS

3.1 Overview

Freeman et al. (1994) suggests foundation movement may result from a wide range of factors, which can include: (1) compression of a soft layer in the ground as a result of the applied foundation or perimeter loads; (2) shrinkage and swelling of clays caused by changes in moisture content; (3) soil softening; (4) compression of filled ground; (5) frost heave; (6) variations in groundwater level; (7) erosion; (8) nearby construction or excavation; (9) chemical attack on foundations; (10) collapse of mine workings or natural cavities; and (11) vibration. Interestingly, nearly all these factors (except item 9) were suggested by homeowners, engineers, town officials, and others during the course of this study. The first six factors are perhaps the most pertinent to Amherst.

Aside from soil conditions, Meehan and Karp (1994) describe housing damage related to expansive soils in California as one where marginally effective foundation designs have led to differential foundation movements. Diaz et al. (1994) suggests inadequate design of basement walls before construction probably accounts for 75 to 85 percent of all problems in residential structures in Ohio. In this study, the mode of basement wall failure and computer modeling also suggest that marginally effective foundation design is a related factor for basement failure in Amherst.

With the constraints of a one-year field investigation, we elected to separate foundation problems into two broad classes based on the predominant damage characteristics. The two classes – *lateral pressure* and *differential settlement* – represent basement failure caused by horizontal and vertical movements. Both classes have numerous potential but generally few primary causative factors. Our investigation attempts to demonstrate the potential for or existence of primary causative factors. Therefore, we did investigate every potential factor (e.g., vibration).

3.2 Lateral Wall Pressure

Four sources likely contribute to lateral pressures on basement walls in Amherst: (1) pressure from soil weight, (2) pressure from soil swell, (3) hydrostatic pressure, and (4) pressure from frost. Identifying lateral pressure damage is not particularly difficult, but accurately quantifying the contribution from each source to past maximum lateral pressures is very difficult. In addition, the four sources depend upon numerous factors that vary throughout the life of any given wall. This section describes each source in greater detail.

3.2.1 Symptoms

The inward bowing of a basement wall is the simplest indication of a lateral pressure problem. The bowing generally occurs when external forces exceed the wall strength and/or the strength of the wall supports. Along the length of the wall, the maximum bowing will often occur near the center of the wall because the adjoining perpendicular walls provide support in the corners. Photo 5 shows a typical bowing wall. If bowing becomes severe, basement walls can collapse inward.

Cracking can also occur when stresses induced by lateral pressure exceed the strength of the concrete or CMU wall. The most common crack pattern originates in the corners and radiates up or down at approximately 45-degree angles. For CMU walls, the cracks propagate along the mortar joints in a stair step pattern. Often these cracks terminate at a long horizontal fracture that parallels the basement floor about two-thirds the way up the wall. The third crack type is vertical and is located near the mid-span (major), but minor cracks can form near the corners (Photos 4, 5, 6). Many cracks have telltale offset that indicates relative movement of the two wall sections.

Excessive lateral pressure can affect the overall integrity of a house. Severe damage results in a visible opening between the superstructure and the top of the basement wall. Like other major fractures, water and pests can easily enter the basement. Exceptional movement destroys a portion of the sill plate and wood frame of the house. In extreme cases, the sill plate loses its support and downward movement of the wood frame occurs.

3.2.2 Soil Weight

Due to a difference in elevation between the outside ground surface and the basement floor, a basement wall supports an adjacent mass of soil, preventing the soil from entering the basement. Therefore, the weight of the retained soil mass induces lateral pressure on the basement wall. Lateral pressure from soil weight is typically considered during design of engineered basement walls using theoretical at-rest earth pressures.

3.2.3 Soil Swell

Soils containing clay undergo volume change when the moisture content of the soil changes (Section 1.5.6.5). When expansive soils are placed against basement walls, the swelling of these soils can induce lateral pressures not typically accounted for in design and construction of walls. For example, swelling pressures from marine clay backfill soils have reportedly damaged basement walls in Northern Virginia (DPWES, 2002). These marine clay backfills often go through several yearly cycles of shrinking and swelling before damaging the walls, and lateral pressures are believed to increase over time due to gradual settlement and infilling of shrinkage cracks. Cyclic shrink/swell can also reduce the shear strength of the backfill and thereby increase lateral earth pressures.

According to Section R403.1.8.1 of the *Residential Code of New York State* (NYSDOS, 2003), soils meeting all four of the following provisions are classified as expansive:

1. Plasticity Index (ASTM D 4318-00) of 15 or greater

2. More than 10 percent of the soil particles pass the No. 200 sieve
3. More than 10 percent of the soil particles finer than 0.005 mm
4. Expansion Index (UBC Standard 18-2) greater than 20.

In order to characterize backfill materials in Amherst for this study, hand auger borings were placed adjacent to nineteen basement walls sites in Amherst to obtain representative samples of backfill. Seventeen of the nineteen basement walls exhibited damage related to lateral pressure. Figure 28 shows the boring site locations. Composite backfill soil samples were tested to facilitate expansive soil classification in accordance with Section R403.1.8.1 of the *Residential Code of New York State* (NYSDOS, 2003). The results of laboratory testing for backfill soils are presented in Table 8.

The expansive soils criteria described above are provided at the bottom of Table 8. Note that all backfill soil samples satisfy all four criteria and are classified as expansive (column 16). The potential expansion rating (ASTM D4829-03) for all but one of the backfill samples ranged from *medium* to *high*. These results indicate that the backfills used at many Amherst sites contain expansive soils.

3.2.4 Hydrostatic Pressure

Hydrostatic pressure is pressure exerted by a fluid due to its weight. Hydrostatic pressure against a basement wall develops when water fills the void spaces within the backfill immediately adjacent the wall. Water buildup against basement walls was confirmed in a survey of 41 homeowners who reported having moderate (43%), some (17%), or minor (14%) leaking. We observed and homeowners described evidence of water buildup that included spurting, dripping, dampness, or efflorescence (salt residual). Other homeowners described leakage during rain events, in the crawl space, and around wall anchors. The following conditions can promote water accumulation against basement walls in Amherst:

- Allowing the ground surface to pitch toward the basement walls. This problem tends to worsen with time after construction because backfill materials adjacent to basement walls typically consist of clay soils (Table 8) classified as silty clay (CL) or fat clay (CH). Clay backfills are very susceptible to densification and subsequent settlement with time as clumps of clay break down. Evidence of post-construction settlement of backfill is widespread in Amherst, and the result is a depression where surface water tends to accumulate.
- Approximately 27% of all inspected homes had detached downspout/gutter systems that discharge water onto the backfill soils adjacent to basement walls.
- Subsurface drains were not installed against the exterior of basement walls of many older houses in Amherst.
- Desiccation cracks, which channel water against basement walls, are common in clayey backfill materials in Amherst.

For newer houses where subsurface gravel drains are installed adjacent to wall footings, drainage of water through the relatively impervious clay backfill soils to the gravel drain is not ensured. If the gravel drain is not protected with a suitable filter fabric, the drain can clog over time as soil is carried into the drain by water moving through the backfill. The effectiveness of a gravel drain for removing water from backfill will be reduced by clogging. Approximately 9 of 52 older houses reported having a drain tile repair.

It is clear that the potential exists for hydrostatic pressures to buildup against basement walls in Amherst due to the factors presented above. Like soil swell, lateral hydrostatic pressure is not typically considered during design and construction of basement walls.

3.2.5 Frost

Water accumulation in backfill soils against basement walls in Amherst is common. If this water freezes, large lateral pressures may develop against basement walls as the freezing water expands. Damage from frost-induced lateral pressure on basement walls in Ohio has been reported by Diaz et al. (1994). The potential depth of frost penetration in Amherst is 3.5 to 4 feet (USACE, 1992). However, lateral pressures from frost may be uncommon because of heat loss from houses combined with the insulating effects of snow. Nonetheless, the potential exists for frost-induced lateral pressures, which are not typically considered during design and construction of basement walls.

3.2.6 Summary

Clay-rich soils often present long-term problems as backfill materials. Their lumpy, cohesive nature, as produced by common excavation techniques, makes it difficult, if not economically or practically impossible to recompact them to states of uniform moisture content and density that will ensure minimum future settlements, minimum swelling potential, minimum hydrostatic pressures, and thus minimum lateral pressure. Beyond the obvious problems of large and protracted surface settlements, clay backfills require significantly stronger basement walls to withstand the larger horizontal earth pressures (CMHC, 2004; Jalla, 1999).

3.3 Settlement

Structural settlement is characterized as either total and/or differential settlement. Total settlement is the magnitude of downward movement. Differential settlement is the difference in vertical movement between various locations of the structure causing distortion of the structure. Generally, the magnitude of total settlement is not a critical structural factor as long as it is uniform. Even relatively small differential settlements can cause cracks in floor slabs, exterior masonry walls, and wall finished with plaster or drywall. Differential settlement can also interfere with the function of the structure.

Settlement can be tolerated in most residential structures provided it is within certain specified limits. A small amount of settlement, including differential settlement, is usually anticipated. However, when houses are constructed on very poor soils where the potential for excessive settlement exists, special procedures must be employed to limit the amount of settlement and/or provide a structure that can tolerate the estimated settlement (Whitlock and Moosa, 1996).

3.3.1 Symptoms

Several damages that result from settlement are described in Section 2.4.2. Total settlement of houses can damage connections to outside utilities, interfere with drainage of surface water away from houses, and/or interfere with the effective functioning of entryways. Differential settlement can cause wood-framed floors to become out-of-level and framed walls and other components can become distorted and distressed. In addition to measured differential settlements, other symptoms include cracking of basement/foundation walls, cracking of concrete slabs, and deflection of structural members directly supported by footings.

Differential movements along strip footing supporting basement/foundation walls can cause walls to crack. Wall cracking associated with differential settlement can be difficult to distinguish from wall cracking caused by lateral pressure (see Diaz et al., 1994, and Freeman et al., 1994). Wall cracks associated with differential settlement tend to extend through the full thickness of the wall and tend to be nearly vertical or diagonal. Diagonal cracks with a stair-step pattern are common in concrete block walls. Diagonal cracks induced by differential settlement are caused by the wall segment on the bottom side of the crack moving down relative to the wall segment on the upper side of the crack. Close examination of vertical cracks will often indicate the relative direction of differential movement across the crack. Rotation associated with differential settlement may result in wider crack thickness at the top of the wall relative to the bottom.

3.3.2 Allowable Settlement

Typically, three types of settlement can affect the performance of a house foundation system in Amherst. The three types of settlement include; 1) total settlement of the house, 2) general differential settlement across the foundation footprint, and 3) differential settlement along the longitudinal axis of perimeter strip footings.

According to *Settlement Analysis* (USACE 1990), total settlement should not exceed 2 inches for most facilities, and a typical specification of total settlement is 1 inch to prevent problems associated with total settlement.

Differential settlement can be quantified in terms of angular distortion. Angular distortion is vertical settlement divided by the horizontal distance over which the settlement occurs. Poulos et al. (2002) suggests angular distortion of 1/250 to 1/150 as an allowable range for preventing structural damage in framed buildings. Meehan and Karp (1994) discuss allowable differential settlements for wood-framed slab-on-grade houses. They suggest that floors experiencing angular distortion of between 1/240 and 1/120 usually indicate post-construction movement with associated damage – cracking of walls and ceilings, sticking doors, etc. In addition, they suggest that angular distortions exceeding 1/120 are usually associated with moderate to severe damage for typical residential buildings. Duncan (1993) suggests that architectural damage, which implies impairment of aesthetics or function, seldom occurs if the angular distortion is less than 1/500.

The most damaging type of differential settlement related to house foundations in Amherst may be caused by differential settlement along the longitudinal axis of perimeter strip footings that support perimeter basement/foundation walls. Differential movements along strip footings can cause walls to crack, allow water to leak in, decrease the capacity of walls to resist lateral pressure, redistribute structural loads causing concentration of loads on portions of footings, and cause progressive structural deterioration.

Unreinforced basement/foundation walls are much more susceptible to significant cracking due to differential settlement than are reinforced walls. Poulos et al. (2002) suggests limiting angular distortion to between 1/2500 and 1/1250 to prevent unacceptable cracking of unreinforced bearing walls, i.e., where the end of the wall settles relative to the midspan. For reinforced bearing walls, the angular distortion should be limited to 1/500 to prevent unacceptable cracking (Poulos et al., 2002).

The use of angular distortion to define allowable differential settlements excludes many important factors (Poulos et al., 2002). A more rational but complicated approach would involve consideration of flexural and shear stiffness of house sections, degree of slip between the foundation and the underlying soils, and house configuration. In addition, the level of distress induced by differential settlement can be affected by the rate at which the settlement occurs (Feld, 1965). Relatively high rates of differential settlement can induce more damage than slower rates of settlement due to the inability of a structure to adjust to rapidly changing foundation support conditions.

3.3.3 Differential Settlement

Figures 7 and 8 shows that house footings built in Amherst's lacustrine soils are typically positioned on the stiff silty clay stratum (hereafter, *stiff stratum*). In many places, the stiff stratum is underlain by firm grading to very soft clay (hereafter, *soft stratum*). *Both the stiff stratum and the soft stratum can contribute to differential settlements of houses* as discussed below. Differential settlement in this section refers to any relative vertical movement between/along footings, including upward and downward movements.

3.3.3.1 Stiff Stratum

Soil volume change due to changes *in soil moisture content* is the primary settlement-related issue for the stiff stratum. Foundation settlement or heave occurs when the moisture contents of soils supporting the foundation change after construction. If the changes in moisture content are not laterally uniform across the footprint of the foundation, differential settlement will occur. Due to the relatively low permeability of the stiff stratum and dynamic causative factors, post-construction changes in foundation soil moisture content can occur from months to decades after construction (see Table 2).

3.3.3.1.1 General Characterization

Soil samples were obtained from stiff soils beneath house foundations at fourteen sites across Amherst (Figure 29). Stiff lacustrine soils were encountered below the typical footing bearing elevations at all of the sites on Figure 29 except sites 5, 13, and 27. Soils immediately below footings at site 5 were generally firm rather than stiff. Foundation soils at sites 13 and 27 consisted of till. Twelve samples of stiff foundation soils were subjected to laboratory testing to facilitate expansive soil classification in accordance with Section R403.1.8.1 of the *Residential Code of New York State* (NYDOS, 2003). Figure 29 shows the sampling locations, and Table 9 shows the results of laboratory testing. All samples were from lacustrine soils except at sites 13 and 27, which were from glacial till. The criteria used to classify expansive soils are provided in Table 9. All of the lacustrine samples satisfy all four criteria and are classified as expansive soils (column 16). The potential expansion rating for lacustrine soils ranged from medium to high (column 12). The two glacial till samples had potential expansion ratings of low and very low. In summary, the stiff stratum is comprised of expansive soils and foundations placed on the stiff stratum are susceptible to differential settlements caused by moisture-related volume changes.

Regarding the expansive soil criteria, the laboratory test results (Tables 8 and 9) suggest that plasticity index is the most sensitive criterion. That is, if the plasticity index is 15 or greater, the remaining three classification criteria are exceeded. Anderson and Lade (1981) report good correlation between expansion index and both liquid limit and plasticity index. We correlated the 36 expansion index tests with the liquid limit and plasticity index results (Figure 30 and 31). These results suggest that expansion index, which is a relatively time-consuming and expensive test, can be reasonably estimated using liquid limit and/or plasticity index.

3.3.3.1.2 Vertical Strain and Moisture Content

In general, the volume of the stiff stratum changes with moisture content. The vertical component of volume change, hereafter referred to as vertical strain, can induce settlement/rebound of overlying foundations. Shrink testing (Briaud et al., 2003) can be used to estimate the relationship between vertical strain and moisture content. Shrink testing involves measuring the moisture content and corresponding sample volume as an undisturbed sample is dried in the laboratory.

The undisturbed shrinkage limit can be estimated from shrink testing. At moisture contents above the undisturbed shrinkage limit, volume change is approximately linearly proportional to moisture content (Briaud et al., 2003). For conditions dryer than the undisturbed shrinkage limit, the change in volume is relatively small. For conditions moister than the undisturbed shrinkage limit, the slope of the vertical strain versus moisture content line is referred to as the vertical shrink-swell coefficient, or S_v. S_v equals the amount of vertical strain for each percentage point change in moisture content.

Six shrink tests were performed on undisturbed soil samples of the stiff stratum. Undisturbed samples were collected from open excavations across Amherst using a 3-inch diameter drive-cylinder (Photo 7).

Shrink test results are presented in Table 10, and sampling location are shown on Figure 29. All the samples satisfy the expansive soils criteria (column 12) and have a potential expansion rating of medium (column 10). The undisturbed shrinkage limits determined from shrink testing are presented in column 15, and the calculated values of S_v are presented in column 16. The average value of S_v is 0.61. Therefore, we estimate that soils comprising the stiff stratum experience approximately 0.6% vertical strain for each percentage point change in moisture content.

Alternatively, the vertical shrink-swell coefficient can be theoretically estimated. In theory, in-situ clay is fully saturated or nearly saturated when the moisture content exceeds the undisturbed shrinkage limit. The shrink test results for undisturbed samples collected in Amherst support this statement and samples were saturated or nearly saturated above the undisturbed shrinkage limit (see Briaud et al., 2003). If water and soil solids are considered to be incompressible, volume change of the saturated soils due to changes in moisture content simply equals the volume of water gained or lost. If strain is equal in all directions, the theoretical vertical shrink-swell coefficient can be calculated using the following equation:

$$S_{v\,(theor)} = \gamma_d / 3\gamma_w$$

where,

$S_{v\,(theor)}$ = theoretical vertical shrink-swell coefficient

γ_d = dry unit-weight of the soil

γ_w = unit-weight of water

$S_{v\,(theor)}$ values for the six shrink test samples are presented in column 17 of Table 10. In general, S_v determined via shrink testing approximates $S_{v\,(theor)}$. The average S_v is 0.61 compared with the average $S_{v\,(theor)}$ of 0.54. In general, S_v exceeds $S_{v\,(theor)}$ due to anisotropic strain.

3.3.3.1.3 Foundation Soil Moisture Content

As discussed above, S_v, defines the relationship between moisture content and vertical strain. If soil moisture contents beneath a house foundation are laterally uniform at the time of construction, subsequent laterally variable changes in foundation soil moisture content will induce differential settlements. Therefore, current lateral variations in soil moisture content beneath the footprint of a house would suggest that post-construction changes in foundation soil moisture content could or have contributed to observed differential settlements.

Potential lateral variations in foundation soil moisture content were investigated at five sites in Amherst. At four of the five locations, evidence of post-construction differential settlement between perimeter basement strip footings and interior basement spread footings was observed. Differential settlement at the remaining site consisted of settlement of one perimeter basement strip footing relative to other footings. Magnitudes of observed differential settlements ranged from approximately 2 to 5 inches.

At each site, a hand auger boring was used to obtain soil samples beneath and adjacent to both the relatively low footing and the relatively high footing. A piece of the concrete floor slab was removed to gain access to soils near interior footings. A hand auger was advanced and discrete soil samples were generally collected every 6 inches. The discrete soil samples were subjected to laboratory moisture content testing.

At three of the five locations, moisture content testing indicated that foundation soils beneath the relatively low footing were significantly drier than those beneath the relatively high footing. Figures 32 and 33 show lateral moisture content variations between interior and perimeter foundation soils at Site 7 and Site 4, respectively (Figure 28 and 29 show site locations). Assuming that moisture contents of foundation soils were laterally uniform prior to construction, the existing measured moisture content profiles at these sites suggest post-construction changes in foundation soil moisture content have contributed to observed differential settlements.

3.3.3.1.4 Estimated Differential Settlement

Using S_v determined by shrink testing, the change in thickness of a soil stratum due to changes in soil moisture content can be estimated if the change in soil moisture content is known. The equation used to estimate the change in thickness of a soil stratum is:

$$\Delta h = h_o \, S_v \, \Delta w$$

where,

Δh = change in thickness of soil stratum;

h_o = initial thickness of soil stratum;

S_v = slope of vertical strain vs. moisture content plot as determined by shrink testing;

Δw = average change in moisture content in soil stratum.

Assuming that foundation soil moisture content was laterally uniform at the time of construction, post-construction differential movements resulting from development of the measured moisture content profiles at Site 7 and Site 4 are calculated using the preceding equation. An average S_v value of 0.61 was assumed based on shrink testing of undisturbed samples (Section 3.4.3.1.2.) The calculated differential movements at sites 7 and 4 are 1.9 and 1.8 inches respectively. Calculations are provided in the Appendix 6.6. These calculated differential settlements would produce angular distortion of 1/128 and 1/102. Recall, Meehan and Karp (1994) suggest limiting angular distortion of floors to 1/240 to avoid damage in wood-framed houses.

The calculated magnitudes of differential movement at Sites 7 and 4 only consider moisture content variations down to 4 and 5 feet below footing elevation, respectively. Moisture content variations likely continue below these depths. Moisture content variations below these depths will increase the magnitude of calculated differential settlements beyond those calculated. Furthermore, soil samples were collected in early July 2004 during a relatively wet summer season. We speculate moisture content variations between interior and exterior foundation soils have been more severe during past periods of drier weather.

3.3.3.1.5 Moisture Content Changes

If the moisture content in the stiff stratum beneath a house changes after the house is constructed, the soils will shrink and/or swell, and foundation settlement and/or heave will occur. Post-construction moisture content changes in the stiff stratum are generally controlled by four factors including, 1) concentration and mineralogy of clay in the soil, 2) water availability, 3) confining pressure, and 4) initial moisture content. Each of these factors is discussed below.

3.3.3.1.5.1 Concentration and Mineralogy of Clay

Laboratory determinations of the clay-sized fraction, liquid limit, plasticity index, unified classification, and expansion index reflect expansive potential due to clay concentration and mineralogy. The combined laboratory test results for soils collected from the stiff stratum (Table 9) indicate medium to high potential for changes in soil moisture content and corresponding shrinkage and/or swelling.

3.3.3.1.5.2 Water Availability

When free water is available to the stiff stratum, it is more susceptible to hydration and swell. The removal of water or the absence of free water makes the stiff stratum more susceptible to desiccation and shrinkage.

Sources of water available to the stiff stratum include infiltration of surface water and capillary rise from the groundwater table. Varved clays contain silt seams that promote the movement of groundwater into and out of the stiff stratum. Leaking utility lines can also feed water to the stiff stratum. Evaporation, transpiration, and subsurface drainage reduce the amount of water available to the stiff stratum and can remove water from the stiff stratum.

Site development and typical house construction can significantly alter water availability. The following factors can increase the amount of water available to the stiff stratum as a result of site development and house construction.

- It is common for water to collect in basement excavations during construction.

- Water accumulation within backfill soils adjacent to basement walls and foundation walls is common (Section 3.2.4).

- Water can collect in gravel beneath basement slabs.

- Water can accumulate in relatively loose and/or relatively pervious soils used to backfill utility trenches.

- Leaking utility lines can feed water to the stiff stratum.

- A basement slab will reduce the evaporation rate beneath the slab.

- Watering of plantings adjacent to a house can feed the stiff stratum with water.

As a result of site development and house construction, the following factors can decrease the amount of water available to the stiff stratum or remove water from the stiff stratum.

- Sump pumps and foundation drainage systems can remove water.

- Roots from trees and vegetation can remove water.

- Surface grading, pavement, and storm sewage collectors can reduce infiltration.

- Relatively loose and/or relatively pervious soils used to backfill utility trenches can intercept and remove groundwater.

3.3.3.1.5.3 Confining Pressure

The stiff stratum is more susceptible to absorbing water and swelling at lower confining pressures than at higher confining pressures. To investigate the effects of confinement on swell potential, two undisturbed soil specimens obtained from the stiff stratum at Site 31 were subjected to swell testing at different confining pressures. Table 8 shows that soil from Site 31 is representative of the stiff stratum at other sites in Amherst. Swell testing of the undisturbed specimens involved applying a confining pressure to the specimen prior to inundating it with water in accordance with ASTM D 2435-03. The moisture content of the specimen confined with 1800 pounds per square foot (PSF) increased 0.3 percentage points and exhibited one-dimensional swell of 0.09% after being inundated. The moisture content of the specimen confined with 400 PSF increased 1.8 percentage points and exhibited one-dimensional swell of 0.91% as a result of inundation. These results demonstrate that confining pressure significantly influences the ability of the stiff stratum to absorb water and swell.

3.3.3.1.5.4 Initial Moisture Content

Portions of the stiff stratum that are initially relatively moist during house construction are more susceptible to post-construction desiccation and shrinkage. Conversely, foundation soils that are initially relatively dry are more susceptible to post-construction hydration and swell. Therefore, the initial moisture content of foundation soils influences the potential for post-construction moisture content changes. For example, if water accumulates in a basement excavation for an extended period of time during construction, the foundation soils may become relatively moist. If post-construction conditions promote desiccation of the foundation soils, the foundation soils will shrink and settlement will occur. Conversely, if the basement excavation remains relatively dry during and after construction, the same post-construction conditions are less likely to cause desiccation and shrinkage of the foundation soils.

3.3.3.1.5.5 Laterally Variable Moisture Content Changes

If the moisture content in the stiff stratum beneath a house changes after the house is constructed, the soils will shrink and/or swell and foundation settlement and/or heave will occur. If moisture changes are laterally variable across the foundation footprint, differential settlements will occur. Several factors promoting laterally variable moisture content changes and corresponding differential settlements are described below:

- Confining pressure on the stiff stratum beneath a typical house foundation can vary significantly across the foundation footprint.
- Typically, basement excavations are not consistently sloped to achieve positive drainage to the sump. During and after the house is constructed, water may enter the excavation and begin to permeate into and moisten foundation soils. In areas with positive drainage, ponded water is only temporary. However, in areas without positive drainage, water may pond for extended periods of time, resulting in laterally variable moisture content changes.
- A common situation promoting laterally variable moisture content changes is when footings are located at different elevations. For example, strip footings supporting crawl-space foundation walls are typically located higher in the soil profile than strip footings supporting basement walls. The amount of water available to soils beneath crawl-space footings is more likely to fluctuate on a seasonal basis. Furthermore, confining pressures beneath crawl-space footings are generally less than those beneath perimeter basement wall footings (Photo 8).
- Concrete slabs influence evaporation, infiltration, and transpiration variably across the foundation footprint. For example, during prolonged dry periods, evaporation and transpiration may remove moisture from perimeter foundation soils while a concrete slab restricts moisture loss from interior foundation soils.
- Trees or other vegetation near foundation walls intercept and remove moisture from beneath their canopy. Normally 90 percent of a tree's root system is within two feet of the ground surface (Biddle, 2001), but roots are opportunistic and will

proliferate where conditions are most conducive for obtaining water. We observed root hairs in the sump pits at a few houses (Photo 9).

- Negative surface drainage or other sources may feed foundation soils with water nonuniformly across a foundation footprint.
- The initial moisture content of the stiff stratum can vary laterally across the foundation footprint prior to construction. For example, soils below large trees, which are removed during construction, may initially be drier than other foundation soils due to transpiration (Meyer and Read, 2001)
- Foundation drainage systems may remove water near the sump pit and/or adjacent to perimeter footings while failing to remove water beneath interior areas of the basement.
- Cyclic drying and wetting of foundation soils combined with variable confining pressures across the foundation footprint can promote laterally variable moisture content changes and differential settlements. Soils beneath slabs and relatively lightly-loaded footings are relatively lightly confined. After these soils dry and shrink, the lack of confining pressure allows them to absorb water and swell when water is reintroduced. Confining pressure for foundation soils beneath relatively heavily loaded footings is much greater. As such, after these soils dry and shrink, the relatively high confining pressure prevents these soils from absorbing water and swelling when water is reintroduced
- Soil temperature can influence soil moisture content (Nelson et al., 2001), therefore temperature gradients beneath a foundation footprint may contribute to laterally variable moisture content changes.

3.3.3.1.6 Summary

Many house footings in Amherst are resting on the stiff stratum that is comprised of expansive soil (Photo 7,10). In general, volume change will occur with changes in soil moisture content. Measured lateral variations in foundation soil moisture content at several sites in Amherst suggest that post-construction changes in moisture content have contributed to the observed differential settlements. Post-construction moisture content changes in the stiff stratum are generally influenced by four factors that include: 1) concentration and mineralogy of clay in the soil, 2) water availability, 3) confining pressure, and 4) initial moisture content. There are numerous factors that promote laterally variable changes in moisture content across the foundation footprint.

3.3.3.2 Soft Stratum

Like the stiff stratum, changes in moisture content in the soft stratum will result in volume changes. Unlike the stiff stratum, the soft stratum is sufficiently below the seasonally fluctuating water table to maintain a relatively stable soil moisture content. Therefore, variations in moisture content within the soft stratum primarily depend on changes in confining pressure, which is commonly referred to as effective stress. An increase in effective stress will squeeze water out of the clay resulting in settlement. Conversely, a reduction in effective stress will result in rebound, which pulls water into the soil. The strain response of clay soils due to changes in effective stress is *gradual* because fluid must migrate to/from void space in the soil. This gradual strain response is known as primary consolidation.

In order to characterize soils from the upper portion of the soft stratum, samples were collected at six sites across Amherst using a hand auger. Laboratory test results and sampling locations are shown in Table 11 and Figure 29 respectively. In general, the samples collected from the soft stratum exhibit higher plasticity than samples collected from the overlying stiff stratum.

The soft stratum was further characterized by investigating the stress-strain behavior of undisturbed samples collected with a drill rig from four sites in Amherst. Laboratory consolidation test (ASTM D 2435-03) results were used to investigate the strain response of the soft stratum due to changes in effective stress. Sampling at sites 30 and 31 was performed in late 2004 with project funding (Figure 29). Laboratory test results for samples collected from sites 29 and 32 were obtained from existing geotechnical engineering reports (Ward, 1973; Daigler, 2004). Table 12 summarizes the consolidation test results. Compression ratios ranged from 0.15 to 0.26, and recompression ratios ranged from 0.015 to 0.025. These results are in agreement with McGuffey et al. (1981), who published results of consolidation testing performed during design of the Lockport Expressway in Amherst. Compression ratios reported by McGuffey et al. (1981) for 68 consolidation tests performed on samples of the soft stratum ranged from 0.13 to 0.35, with a mean of 0.23. Recompression ratios reported for 19 consolidation tests performed on samples of the soft stratum ranged from 0.014 to 0.031, with a mean of 0.022 (McGuffey et al., 1981).

McGuffey et al. (1981) found that the soft stratum is overconsolidated at the top of the stratum, and the apparent preconsolidation stress decreases with depth. The laboratory test results presented in Table 12 are consistent with their findings. Columns 7 and 8 indicate that samples collected from a depth of 14 feet to 17 feet were over consolidated, and samples collected from greater than 20 feet were normally consolidated. We speculate that a historical drop(s) in the groundwater table and desiccation are responsible for the apparent overconsolidation in the upper levels of the soft stratum.

The consolidation test results can be used to estimate the strain response of the soft stratum due to various loading conditions. If the soft stratum beneath a house strains non-uniformly across the foundation footprint, differential settlement/rebound will occur. There are at least three potential loading conditions that can contribute to non-uniform straining of the soft stratum beneath houses in Amherst including 1) removal of soil from basement excavations during construction, 2) raising lots with significant amounts of new fill around the perimeter of houses, and 3) long-term lowering of the groundwater level.

In order to evaluate the potential for these loading conditions to induce significant magnitudes of settlement and/or rebound, we considered a hypothetical house at Site 29. Figure 34 shows a schematic drawing of the hypothetical square house located within a 30’ x 30’ x 6’ basement excavation. Four specific load cases are described below:

- Load Case I – This load case considers only rebound induced by the difference in weight between the excavated soil and the house. It is assumed that 50% of the rebound occurs after excavation but prior to foundation construction.
- Load Case II – Load Case I combined with the addition of 2 feet of fill placed around the perimeter of the house after construction.
- Load Case III – Load Case I combined with a 4-foot drop in the water table after construction.
- Load Case IV – Load Case II combined with a 4-foot drop in the water table after construction.

For settlement analyses, the soft stratum was divided into two substrata. Consolidation test results for samples collected from the middle of each substratum at Site 29 were used to estimate the strain response of the substratum due to changes in effective stress induced by the various loading conditions. Changes in effective stress at the middle of each substratum were estimated using Boussinesq analysis. Settlement/rebound calculations are included in the Appendix 6.6.

3.3.3.2.1 Load Case I

If house construction involves a significant amount of excavation to facilitate basement construction, the weight of excavated soil typically exceeds the weight of the house and its contents. For example, the combined total live and dead weight for a 30' x 28' rectangular two-story house with basement is approximately 80 tons (Willenbrock et al., 1998). Assuming the basement slab and loading on the basement slab contribute an additional 20 tons, the total weight supported by underlying foundation soils is approximately 100 tons. Soils removed from a 30' x 28' x 6' basement excavation weigh approximately 300 tons - nearly 3 times the combined live and dead weight of the house. Therefore, the soft stratum beneath the basement generally "feels" less stress after construction of the house than it did prior to construction due to removal of the weight of the excavated soil. The degree to which the soils beneath the basement are unstressed varies across the basement footprint. For example, soils beneath the center of the basement are unstressed significantly more than those soils beneath the corners of the basement. This unbalanced unloading will result in differential rebound of the soft stratum.

For the hypothetical house and subsurface conditions illustrated in Figure 34, a net average pressure reduction of 480 PSF at the base of the basement excavation is realized due to the difference between the weight of the excavated soil and the live and dead weight of the house. The soft stratum will rebound as a result of this pressure reduction.

As previously discussed, the strain response of clay to changes in effective stress occurs gradually. The results of consolidation testing at sites 30 and 31 were used to estimate the coefficient of consolidation during unloading. Laboratory coefficients of consolidation for the soft stratum during unloading were calculated to be approximately

0.1 $feet^2$/day. Based on this information, the time required to achieve 50% rebound due to excavation is approximately 200 to 400 days. However, relatively pervious silt seams within the soft stratum likely cause pore pressures to equilibrate faster than predicted by consolidation theory, and rebound will probably occur significantly faster than predicted by laboratory consolidation testing. Therefore, considering the conceptual nature of the calculations, it is reasonable to assume that 50% of the rebound occurs after house construction is completed. Using this assumption, the magnitudes of post-construction rebound are calculated for various locations across the footprint of the house. The calculated magnitudes of post-construction rebound at the center of the basement, wall midpoints, and the corners of the basement are presented for Case I in Table 13, and the corresponding magnitudes of angular distortion are presented for Case I in Table 14.

3.3.3.2.2 Load Case II

Many homes in Amherst, both past and present, use one to four feet of fill to raise the surface elevation of the lot (Photo 11). When the new grade is significantly higher than the pre-construction grade, the soft stratum beneath the perimeter of the basement "feels" the weight of the new fill much more than soils beneath the center of the basement. The degree to which the soils beneath the basement are stressed by the placement of perimeter fill varies across the basement footprint. For example, soils beneath the center of the basement are stressed significantly less than soils beneath the corners of the basement. This unbalanced stressing will result in differential settlement.

Case II considers the hypothetical house (Figure 34) and the combined strain response of the soft stratum due to basement excavation and placement of 2 feet of fill adjacent to basement walls. The combined effects of basement excavation and perimeter filling result in rebound at the center of the basement and settlement at the perimeter. The magnitudes of rebound/settlement and angular distortion are presented in Tables 13 and 14.

3.3.3.2.3 Load Case III

Section 1.5.8 suggests that groundwater elevation in the middle soil zone may fluctuate several feet during some periods and that land development can impact the hydrologic budget of an area. Here we consider how a drop in the groundwater level within the soft stratum beneath houses can contribute to laterally variable straining of the soft stratum. As the groundwater level drops, previously submerged soils become effectively heavier due to a loss of buoyancy. The increase in effective weight of these soils increases effective stresses in the soft stratum. Also, a drop in the groundwater level within the soft stratum will induce negative pore pressures in the previously submerged soils that remain saturated due to capillarity. The negative pore pressures increase effective stresses in the capillary zone. In short, a drop in the groundwater level within the soft stratum increases effective stresses in the soft stratum due to a combination of loss of buoyancy and negative pore pressures.

For the hypothetical house (Figure 34), the combined strain response of the soft stratum due to basement excavation and a 4-foot drop in the water table results in varying magnitudes of settlement across the basement footprint. The magnitudes of settlement and angular distortion are presented in Tables 13 and 14.

3.3.3.2.4 Load Case IV

Case IV considers the strain response of the soft stratum due to basement excavation, with placement of a 2-foot thick fill around the perimeter and a 4-foot drop in the water table. The calculated strain response of the soft clay stratum produces varying magnitudes of settlement across the basement footprint. The magnitudes of settlement and angular distortion are presented in Tables 13 and 14.

3.3.3.2.5 Summary

The estimated magnitudes of settlement/rebound presented in Table 13 demonstrate that fill placement around the perimeter of a house and/or a drop in the groundwater level can result in significant total and differential settlements due to laterally variable strain response of the soft stratum. Meehan and Karp (1994) suggest limiting angular distortion of floors to 1/240 to avoid post-construction damage of wood-framed houses (Section 3.4.2). The magnitudes of angular distortion presented in Table 14 do not exceed 1/240 but approach this limit for Case IV conditions (1/280). For unreinforced bearing walls, where the end of the wall settles relative to the midspan, Poulos et al., (2002) suggest limiting angular distortion to between 1/2500 and 1/1250 to prevent unacceptable cracking. The limiting value of angular distortion proposed by Poulos et al., (2002) depends on the length to height ratio of the wall. For the hypothetical basement walls illustrated on Figure 34, the limiting value for angular distortion is 1/1500. The magnitudes of angular distortion between the wall midpoint and the corner exceed 1/1500 for the Case II, III, and IV loading conditions (Table 14). These results only consider primary consolidation response of the soft stratum and not potential movements due to shrink/swell of the overlying stiff stratum. In summary, *the soft stratum*, where present, *is a potential primary causative factor* for settlement and must be considered in the design of houses in Amherst.

3.4 DESIGN AND CONSTRUCTION

The following section briefly discusses design and construction practices that affect the wall strength, particularly with respect to lateral pressures. We examined foundation drawings during home inspections and noted common design features that, in some cases, may have contributed to foundation problems. This section reviews these design features and highlights some discrepancies between drawings and the actual structure.

3.4.1 Design

A structurally stable house foundation begins with an accurate design that accounts for horizontal and vertical forces. The horizontal component of these lateral forces is transferred largely through a combination of soil friction along the bottom of the footings and passive soil pressure on the sides of the footings and foundation walls. Foundation walls generally provide support for the superstructure above as well as enclose a basement or crawl space below.

3.4.2 Footing

The blueprints for about 40 houses revealed that all designs utilized the minimum footing thickness (8") and met the minimum footing projection based on conventional "rules-of-thumb" and the current building code requirements (NYSDOS, 2003). Only 19% of the houses had footing projections that were greater than one-half the foundation width.

We speculate nearly all builders prior to 2003 relied on general presumptive values (Section 1.6.3) instead of a geotechnical evaluation in their foundation design. Diaz et al. (1994) states that caution should be taken when using average values supplied by codes because they *are* simply presumptive values. Several home designers/engineers address unforeseen potential bearing capacity problems by annotating the house plans with the instruction, "if excavation uncovers soil of less than this value, notify engineer." In areas with problem soils such as Amherst, these procedures are likely too lenient.

In general, rules-of-thumb and presumptive guidelines result in foundation designs intended for average conditions, but soil conditions in Amherst are generally not average and warrant more detailed designs.

3.4.3 Concrete Strength

Concrete strength can be an important factor when discussing the overall strength of a foundation wall. We investigated concrete strength using a non-destructive testing procedure that utilizes a rebound hammer (similar to Schmidt Hammer®).

At the majority of inspection sites, we measured the concrete strength on a representative number of walls (4 to 10). Twenty readings were recorded on each wall, with the highest and lowest values being rejected. The indicated average strength for all the homes tested exceeded the current NYS Residential Building Code minimum compressive strength requirement of 3,000 pounds per square inch (psi, 28 days after placement). More importantly, no particular wall showed a significantly lower compressive strength than any other wall; with all values being greater than 3,900 psi. This finding suggests that concrete strength does not appear to be a primary causative factor. We recognize the limitation of this testing method and recommend core sampling where more accurate measurements are needed.

3.4.4 Wall Thickness

All houses we inspected had 8" thick cast-in-place concrete or CMU walls except for walls supporting brick facia, which used 10" walls. These wall dimensions generally represent the minimum standard for the current building code, and we presume the minimum requirement during the period of construction. Moreover, the foundation drawings do not indicate reinforcement steel was used in the walls, but we did not independently confirm this with alternative testing methods (e.g., ultra sonic) because of cost. Section 3.5.11.1 will show that unreinforced concrete walls can yield to lateral pressure forces generated by typical soil conditions in Amherst.

3.4.5 Backfill

As cited by Diaz et al. (1994), lateral pressure problems are exasperated by using unsuitable backfill material, usually from on-site excavation. None of the foundation blueprints prescribed the material to be used for backfill. As shown in Section 3.2.3, all backfill soils analyzed in this study are classified as being expansive, which is generally not recommended for backfill material (Jalla, 1999).

3.4.6 Anchor Rods

Approximately 81% of the blueprints called out the inclusion of anchor rods through the sill plate to securely attach the wood frame superstructure of the house to the foundation walls (Figure 35). This additional lateral support is important for the header walls (i.e., walls that are perpendicular with the floor joists), but are even more critical for the stringer walls (i.e., the walls that are parallel with the floor joists).

3.4.7 Geometry

House geometry in Amherst has evolved from comparatively rectangular to more irregular designs. For example, modern homes include cubbyholes, bay and build out windows, and mudrooms. These features result in additional concentrated stresses in the basement walls and should be accounted for in the design. Additional design measures were not evident on the blueprints we reviewed.

3.4.8 Concentrated Loadings

Chimneys, pilasters and other heavy masonry-type features can add tremendous weight to a standard foundation. We did not observe design features that account for these concentrated loads.

3.4.9 Exterior Foundation Drains

Only 13% of the blueprints called out a drainage system below the backfill on the exterior of the foundation. Town Inspectors confirmed that perforated pipe systems along the exterior of the footings are a relatively new house construction feature in Amherst.

3.4.10 Construction

Nearly 58% of blueprints did not match the structure built. The degree of modification varied but typically included a reversal in position of the garage, bedrooms, or crawl space, but changes in the position of the support beams, construction material, sump pump or utilities were also common. Only 7% of the homes actually had anchor rods installed (c.f., 81% of the blueprints). Approximately 1% of the homes had missing structural members that were intended to carry superstructure loadings in the home.

Regarding new building sites, we speculate that geotechnical recommendations from the geotechnical engineer are not being utilized by all contractors. For example, many geotechnical engineers recommend that foundation excavations should not become saturated, frozen, and disturbed prior to footing construction (Photo 12), but the practice seems relatively common.

3.4.11 Wall Strength Modeling

Basement wall computer modeling was accomplished using GTSTRUDL– Version 26 (GIT, 2002). This software was developed by Georgia Institute of Technology as a structural engineering analysis and design tool. Several computer-based models were developed to analyze the forces, specifically lateral pressures, acting on and in the basement walls of the homes in the study. Finite element modeling was used to generate graphical analyses of the stress and strain in the walls. These analyses were done to develop an understanding of the internal forces acting in the basement walls to assist in identifying and classifying field observations. As more and more home inspections were completed, this analysis was also edited and then used to possibly confirm and explain what was witnessed in the field. The analyses were not intended to determine the structural soundness of the basement walls or to provide a design for use by the Town, or any designer/engineer, or homebuilder. As a check of the finite element analysis, another analysis was performed using only a simple beam method. Parameters used in the model are provided in Appendix 6.5.

Two different unreinforced concrete wall thicknesses (8” and 10”) were analyzed for this study. The lengths of walls analyzed were 20 and 40 feet. Two support conditions were also included in our modeling. The supported condition presumes support at the top in addition to the bottom and both ends of the wall. This was utilized to represent a foundation wall that had the appropriate number, size and location of anchor rods at the top of the wall, while the unsupported top condition was one without any anchor rod attachments being supplied. The soil types used in this analysis were a compilation of soil information available in the early part of this study. Generally speaking the five backfill types acting against the walls for this modeling are (1) a granular soil in the dry condition, (2) a granular soil in the saturated condition, (3) a non-expansive wet clay, (4) a dry moderately expansive clay, and (5) dry highly expansive clay.

3.4.11.1 Modeling Results

Figures 36 and 37 summarize the output of the finite element analyses for the various wall thicknesses, support conditions, and soils types. These results highlight regions of stress, which are especially concentrated from the bottom corners of the wall at approximate 45-degree angles towards the center of the wall, similar to the fracture patterns observed during house inspections (Section 3.2.1). Values above the red line exceed the maximum tensile stress of standard concrete (~389 psi). This implies that cracking could occur or will happen depending on the actual strength of the concrete wall. The summary below is concerned with the moderately expansive clay simulation as an approximation of the soil conditions in Amherst.

For the 20-foot wall (Figure 36), the unsupported 8- and 10-inch thick wall stresses are at or near the threshold needed to initiate cracking. In the 40-foot wall simulation (Figure 37), both unsupported top condition walls have stresses that exceed the tensile strength of wall and cracking may occur. Interestingly, if the soils are highly expansive, even an 8-inch supported 40-ft wall is near the cracking threshold.

Modeling was also used to help understand deflection related to the effect of lateral pressure at the top of the foundation walls. An unsupported foundation wall was modeled using GTSTRUDL. Expectedly, the deflection at the top of wall was greater for a thinner and longer wall for all the soil types modeled. In fact, an 8-inch thick, 40-foot long wall had almost double the deflection of a 10-inch thick, 40-footlong wall for the most expansive clay soil type.

The modeling results demonstrate that (1) granular backfills can reduce lateral pressures, (2) top wall support is a critical design and construction element, and (3) historical wall designs are at or near the threshold for cracking. In one home affected by lateral pressures, we observed as much as nine inches of deflection along the top of an 8" bowing wall (Photo 13).

3.5 Associative Factors

Besides quantified geotechnical evidence about causative factors, there are several associative causative factors that we speculate to be important in explaining foundation damage at some sites. These qualitative observations/findings are diverse and we generally have limited information about these factors.

Heterogeneous soil conditions or soil discontinuity beneath a house foundation can induce differential settlements if the compressibility of soils beneath the footings varies significantly across the footprint of the house (Diaz et al., 1994). For example, if one end of a strip footing is placed on stiff native clay and the other end on loose backfill material, then the end of the footing placed on loose backfill material could be expected to settle with respect to the end placed on stiff clay. A location where this commonly occurs is at the transition between the basement and crawl space. Near the basement wall, strip footings supporting crawl-space foundation walls often bear on loose backfill material placed against the basement wall. Even if house foundations are placed on initially homogeneous soils, areas of the foundation subgrade can be softened by water and frost, resulting in variable soil compressibility beneath the footprint of the house (Photo 10). Foundation subgrade soils are especially susceptible to softening by water and/or frost during construction activities when the subgrade soils are exposed for prolonged periods. Heterogeneous soil conditions with expansive and non-expansive soils are within the same excavation may also promote differential movements (Photo 8).

At locations where footings are placed near the soft stratum or atop relatively compressible soils, differential settlement can be induced by variable footing pressures. Heavily loaded areas with high footing pressure would be expected to compress the underlying soft soils more than lightly loaded areas.

Erosion of soils beneath footings could potentially contribute to differential settlement at some sites (Photo 14). It is hypothesized that water moving along foundation surfaces can potentially erode foundation soils. Foundation soils consisting of silts and fine sands are most susceptible to erosion. Although it is not a definitive symptom, accumulation of soil in the sump pit suggests potential foundation soil erosion.

Historically in Amherst, house foundations have been designed to limit footing contact pressures to an allowable bearing capacity to prevent bearing capacity failure of foundation soils. Bearing capacity failure occurs as the soil supporting the foundation fails in shear. House footings in Amherst are typically placed on stiff soils. Bearing capacity failures in stiff soils typically result in sudden and catastrophic downward movement shortly after a footing is loaded (Poulos et al., 2002). Foundation failures in Amherst do not typically occur during or shortly after construction. Therefore, although possible at some locations, bearing capacity failures are not suspected to be a significant contributing factor to foundation settlement problems in Amherst.

Hydrostatic pressure has likely contributed to uplift and cracking of floor slabs at some locations. Hydrostatic uplift pressures will develop beneath a floor slab whenever the foundation drainage system fails to keep the groundwater level below the base of the slab. Evidence of hydrostatic pressure buildup includes water entering through cracks in the basement walls and/or slab. Concrete basement floor slabs are not typically designed/constructed to withstand uplift pressures. At one inspection site, the sump pump had recently failed and the slab had clearly fractured, lifted, and hollow areas beneath the slab were detectable with a hammer.

The Town Building Department has robust inspection requirements for new construction (Section 1.6.3). By law, inspections may include but are not limited to building location, site preparation, excavation, foundation, framing, superstructure, electrical, plumbing, heating, ventilation and air conditioning (NYSDOS, 2003). In some cases, this inspection regime may not be sufficient to ensure compliance with the code. In addition, inspectors acknowledge the need for additional skills training to keep pace with new information and technologies.

3.6 Figures, Tables, Photos

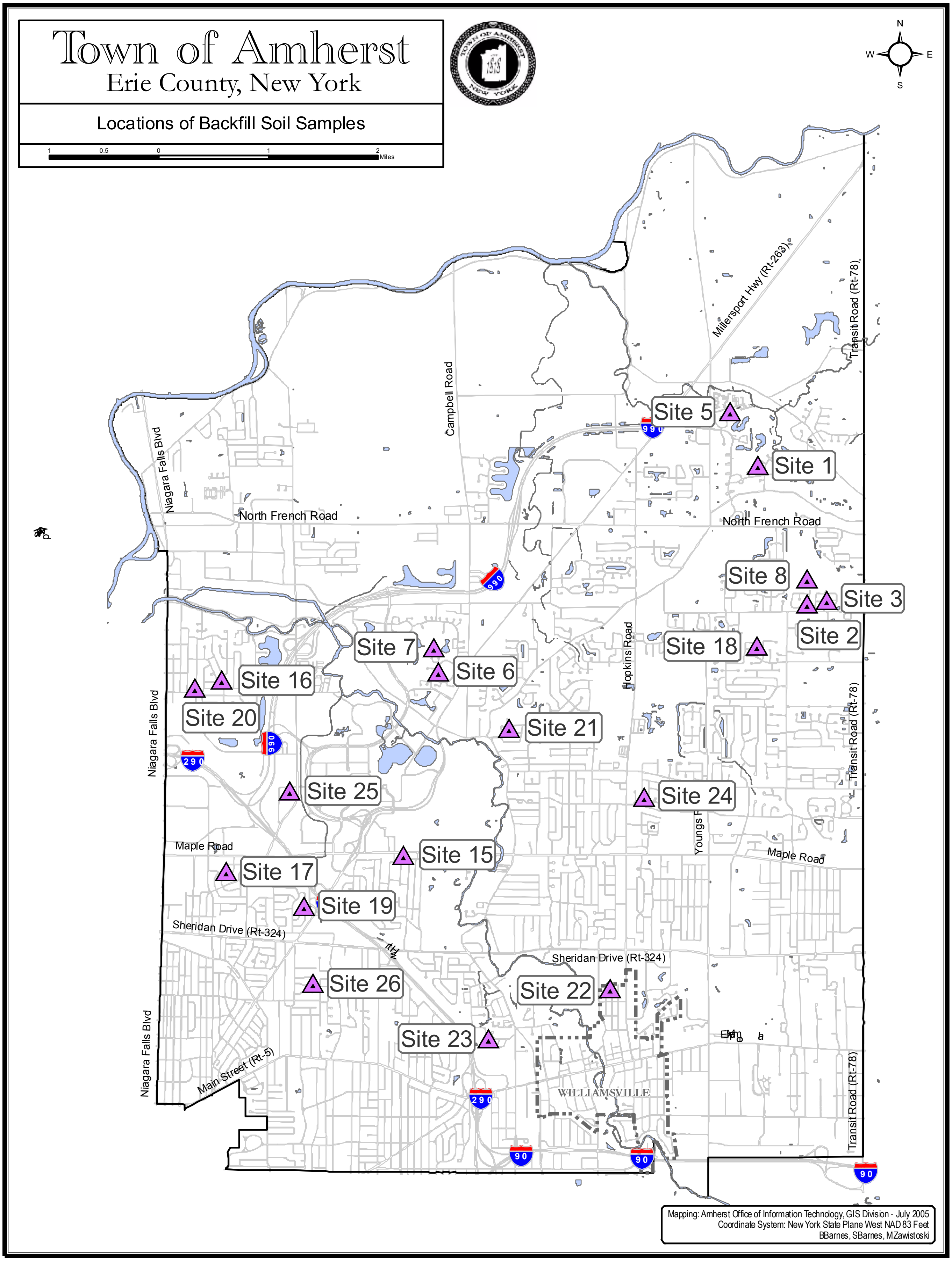


Figure 28: Backfill sampling locations in Amherst, NY. Seventeen of nineteen basement walls exhibited damage related to lateral pressure.

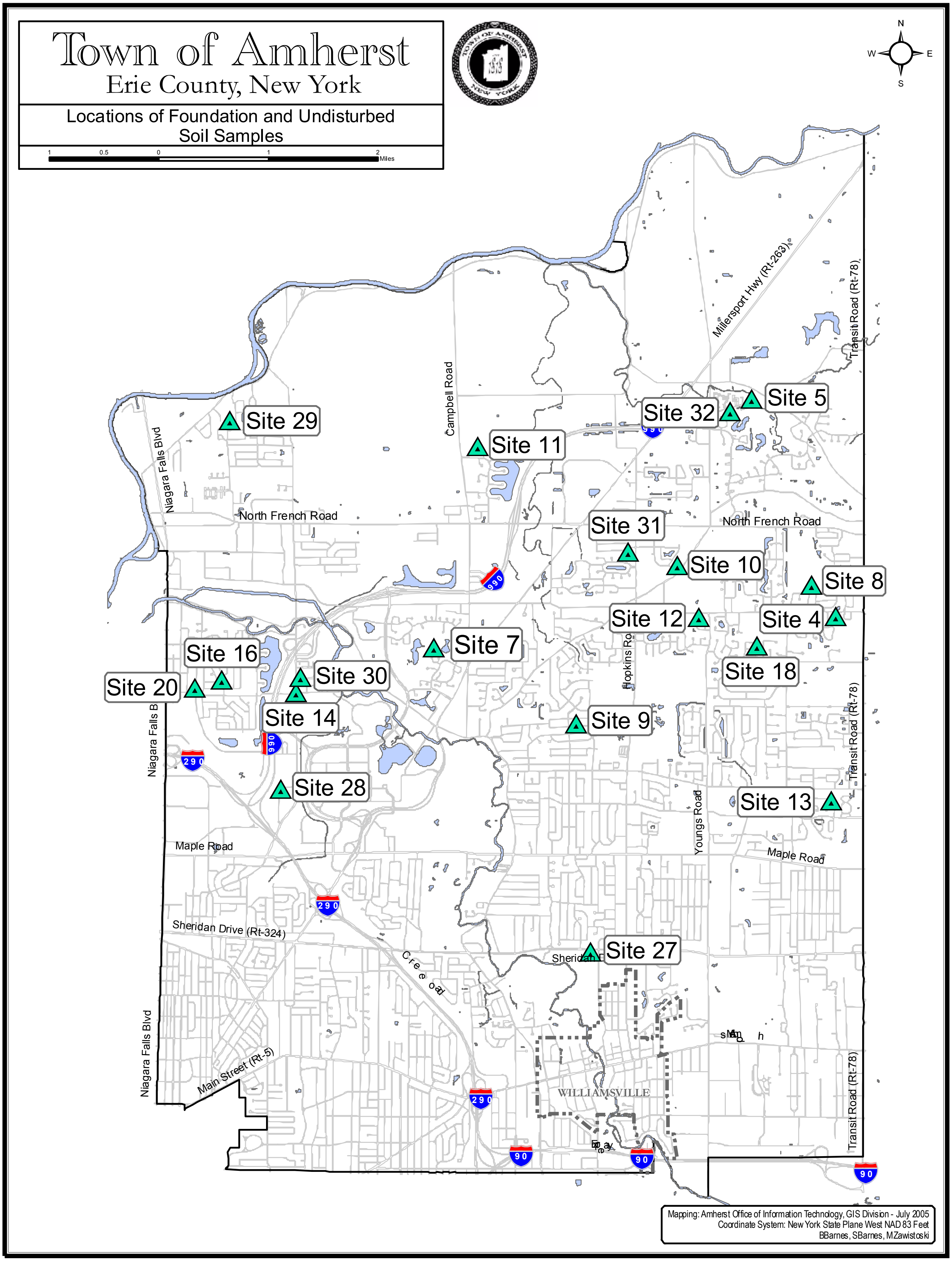


Figure 29: Foundation soil sampling locations.

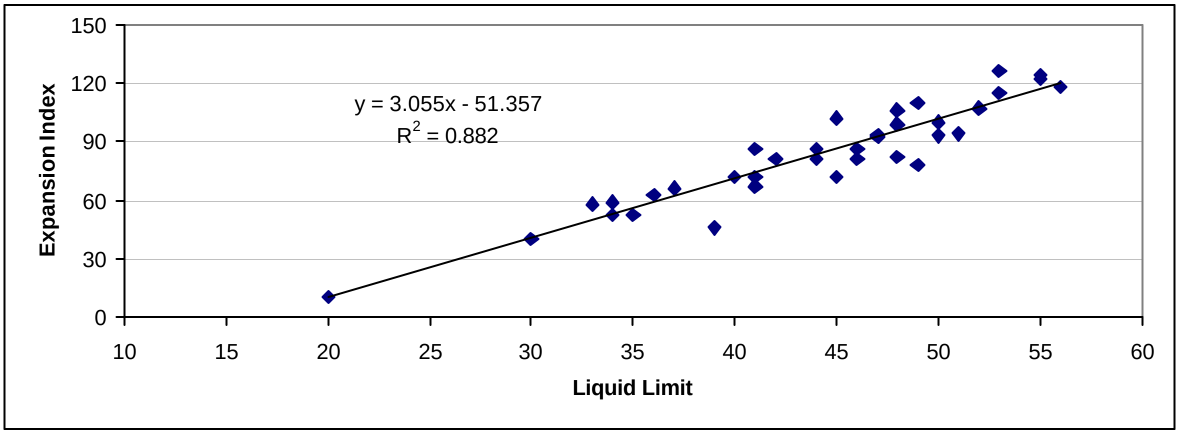


Figure 30. Regression analysis of expansion index (EI) and liquid limit (LL) for backfill and foundation soil samples in Amherst, NY.

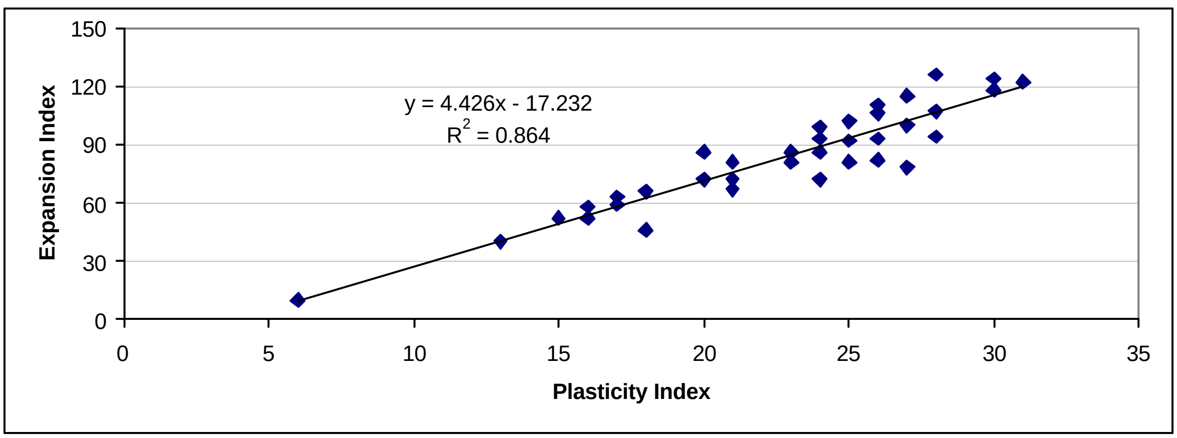


Figure 31. Regression analysis of expansion index (EI) and plasticity index (PI) for backfill and foundation soil samples in Amherst, NY.

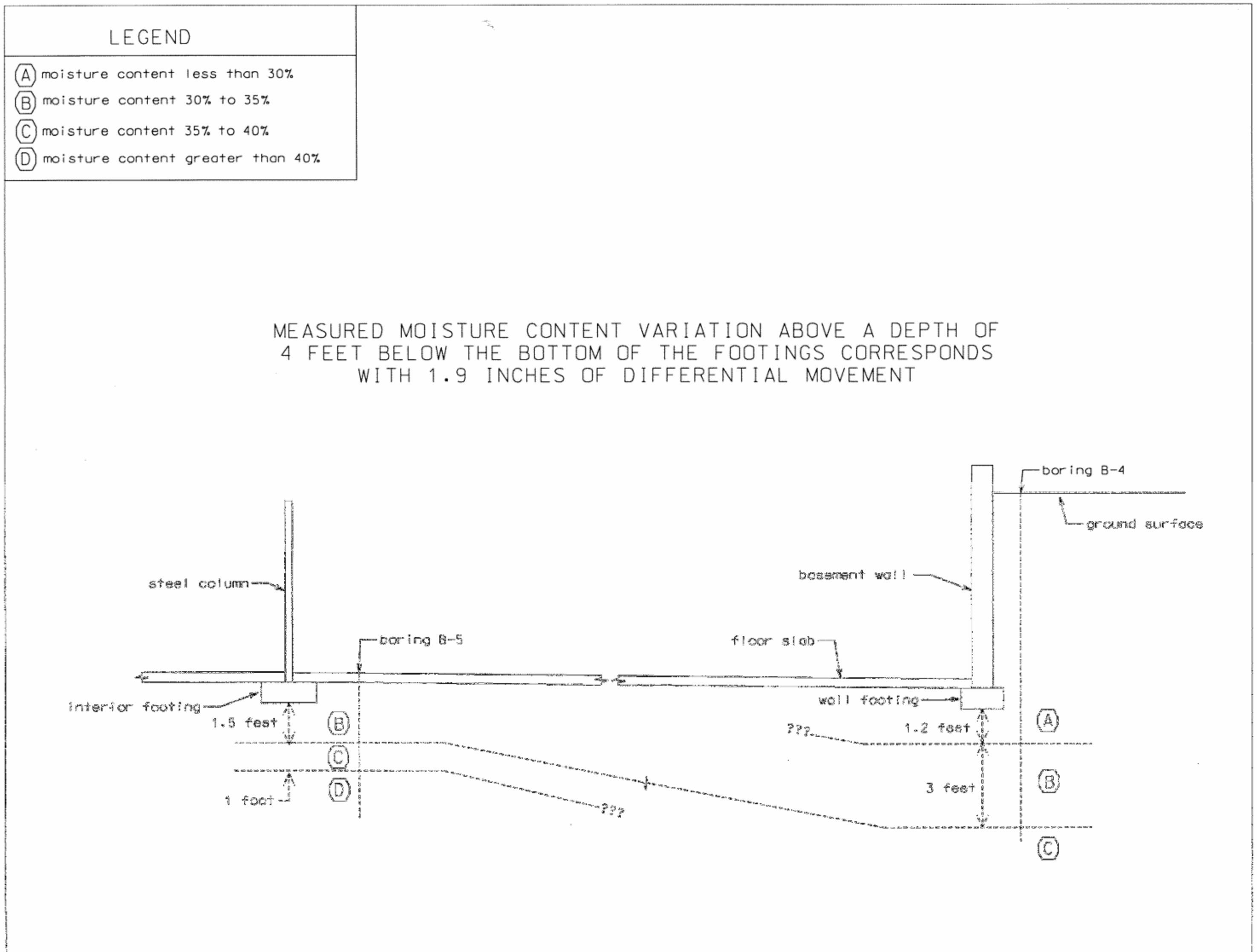


Figure 32. Foundation soil moisture content variation estimated from two boring at Site 7 in Amherst, NY (July, 2004). Gravel under slab is not shown.

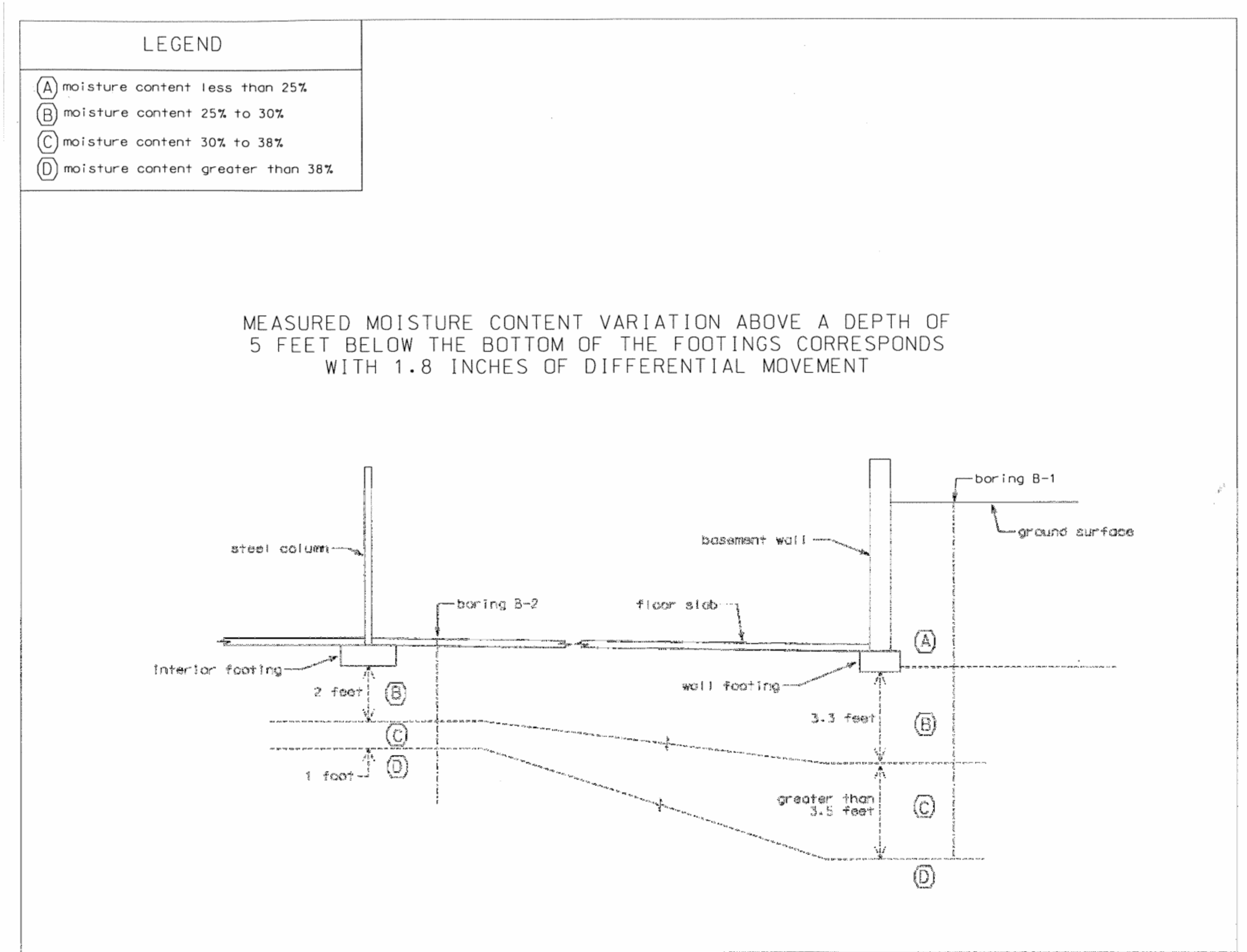


Figure 33. Foundation soil moisture content variation estimated from two borings at Site 4 in Amherst, NY (July, 2004).

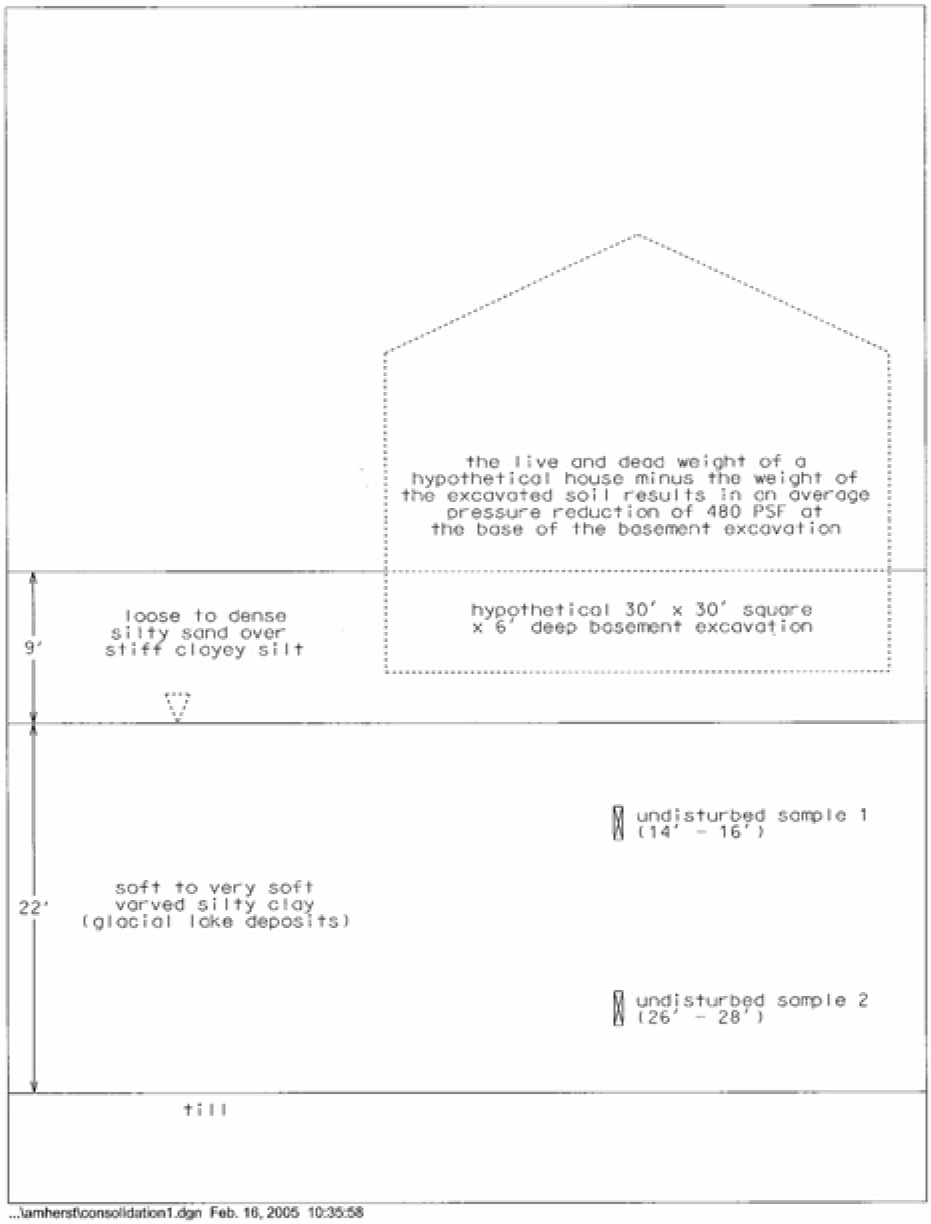


Figure 34. Schematic of hypothetical square house located within a 30' x 30' x 6' deep basement excavation (Section 3.4.3.2).

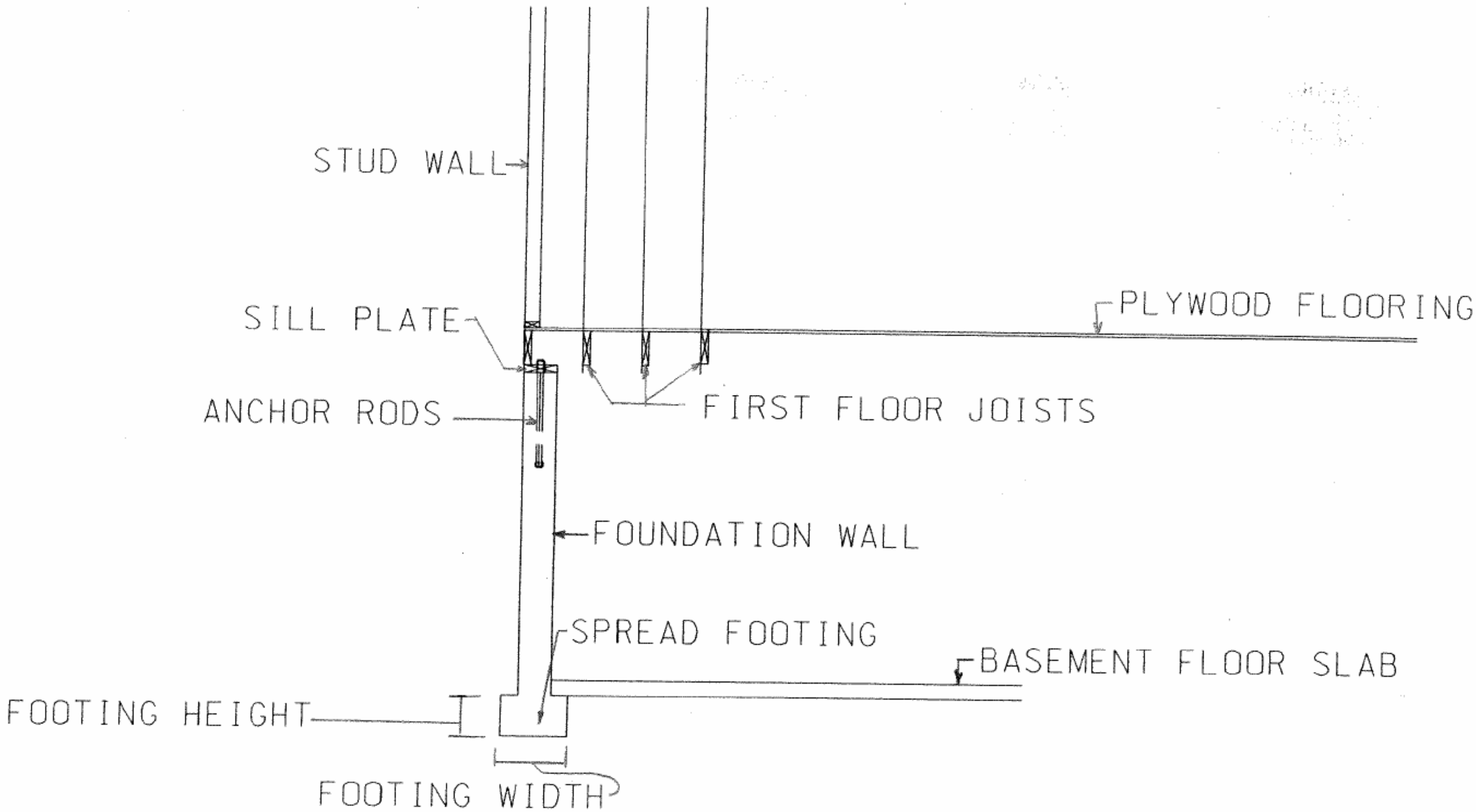


Figure 35. Typical structural components of foundation.

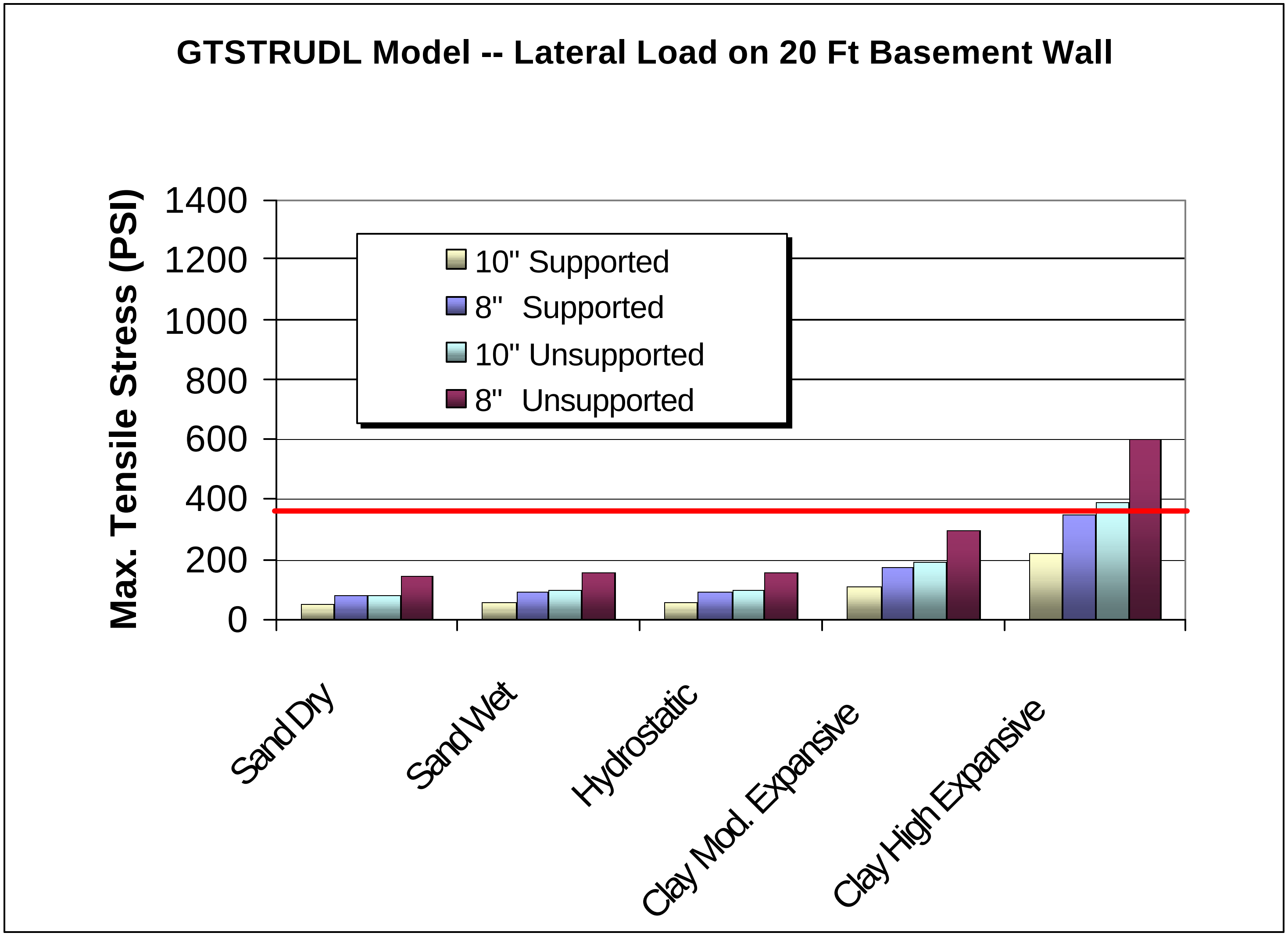


Figure 36. Modeling results of 20 ft basement wall with different backfill conditions, thickness, and support conditions. Red line indicates approximate maximum tensile strength of standard concrete (~389 psi). See Appendix 6.5 for model parameters.

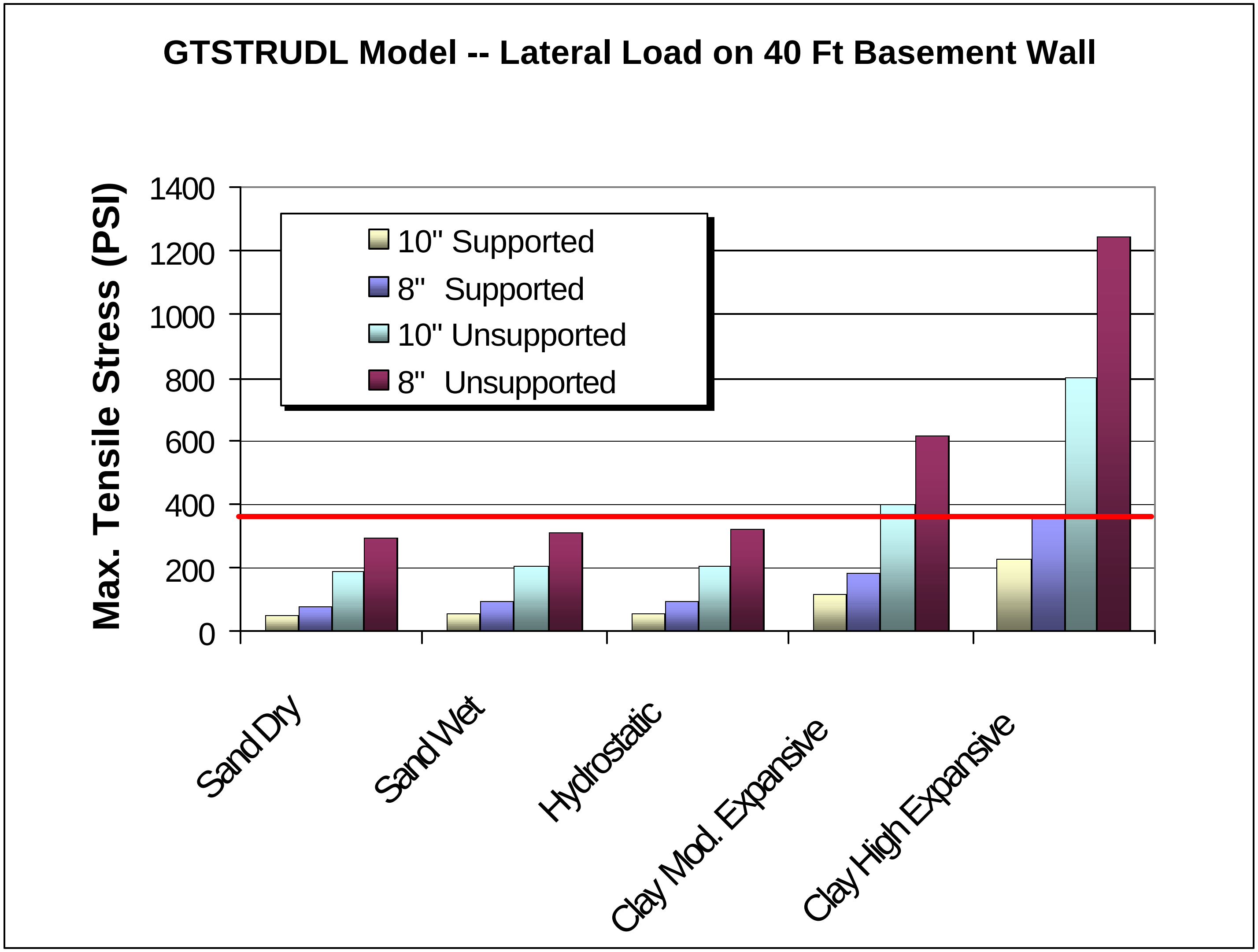


Figure 37. Modeling results of 40 ft basement wall with different backfill conditions, thickness, and support conditions. Red line indicates approximate maximum tensile strength of standard concrete (~389 psi). See Appendix 6.5 for model parameters. Results suggest unsupported walls may yield in the moderately expansive case similar to Amherst.

Table 8. Laboratory test results for basement wall backfill soils in Amherst, NY (2004)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
SITE	**COMPOSITE SAMPLE DEPTH (FEET)**	**CLAY FRACTION – PERCENT FINER THAN 0.002 mm**	**PERCENT FINER THAN 0.005 mm**	**PERCENT FINER THAN #200 SIEVE**	**LIQUID LIMIT (ASTM D4318)**	**PLASTICITY INDEX (ASTM D4318)**	**UNIFIED CLASSIFICATION (ASTM D2487)**	**SHRINKAGE LIMIT (ASTM D427)**	**SPECIFIC GRAVITY (ASTM D854)**	**EXPANSION INDEX (ASTM D4829)**	**POTENTIAL EXPANSION (ASTM D4829)**	**NUMBER OF NATURAL MOISTURE CONTENT TEST SAMPLES[1]**	**HIGHEST MEASURED NATURAL MOISTURE CONTENT (%)**	**LOWEST MEASURED NATURAL MOISTURE CONTENT (%)**	**EXPANSIVE? (Residential Code of NYS, Section R403.1.8.1)**
1	1 - 3	36	47	80	39	18	CL	14	2.65	46	LOW	7	23.7	8.9	YES
2	0 – 4.5	36	48	76	34	17	CL	13	2.67	59	MEDIUM	3	26.6	22.6	YES
3	2 – 4.7	35	47	83	37	18	CL	15	2.68	66	MEDIUM	5	25.8	20.2	YES
5	0.5 – 4	52	71	96	46	24	CL	17	2.72	86	MEDIUM	6	25.3	22.3	YES
6[2]	1.5 – 4.2	36	42	76	33	16	CL	13	2.65	58	MEDIUM	1	25.5	25.5	YES
7	1 – 4.2	48	62	88	40	20	CL	17	2.71	72	MEDIUM	8	28.6	22.6	YES
8[3]	1 – 5	35	47	81	36	17	CL	14	2.69	63	MEDIUM	6	26.7	21.1	YES
15	1 – 4	56	70	93	48	26	CL	17	2.76	106	HIGH	6	31.8	13.1	YES
16	1 – 4	44	56	81	41	21	CL	14	2.71	72	MEDIUM	6	27.3	20.9	YES
17	1.5 – 4.5	58	73	100	50	27	CH	18	2.75	100	HIGH	6	26.3	21.7	YES
18	1 - 5	52	71	95	44	23	CL	15	2.75	81	MEDIUM	6	21.9	19.9	YES
19	1 - 4	60	75	95	52	28	CH	17	2.75	107	HIGH	6	27.1	21.5	YES
20	2 - 4	45	57	86	45	25	CL	15	2.72	102	HIGH	4	20.3	18.1	YES
21	1 - 4	48	64	92	44	23	CL	19	2.74	86	MEDIUM	6	22.1	19.2	YES
22	1 – 4	46	60	97	41	20	CL	15	2.73	86	MEDIUM	5	28.5	20.1	YES
23	2.5 – 4.5	33	45	88	34	15	CL	14	2.73	52	MEDIUM	4	18.4	14.2	YES
24	1 – 4.9	52	68	94	47	24	CL	15	2.74	93	HIGH	5	25.6	23.6	YES
25	1 – 3.2	56	75	100	48	24	CL	16	2.73	99	HIGH	3	30.4	25.7	YES
26	0 – 3.5	50	63	91	42	21	CL	15	2.72	81	MEDIUM	0	NA	NA	YES
MAX		60	75	100	52	28		19	2.76	107			31.8	25.7	
MIN		33	42	76	33	15		13	2.65	46			18.4	8.9	
AVERAGE		46	60	89	42	21		16	2.72	80			25.7	20.1	
STANDARD DEVIATION (+/-)		9	11	8	6	4		2	0.03	19			3.4	4.3	
Residential Code Criteria Defining Expansive Soils			> 10	> 10		≥ 15				> 20					

[1] Discrete samples were collected within the depth range listed in column 2; [2] No significant damage observed at Site 6; [3] No significant damage observed at Site 8

Table 9. Laboratory test results for stiff foundation soil samples in Amherst, NY (2004)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
SITE	**COMPOSITE SAMPLE DEPTH (FEET)**	**CLAY FRACTION – PERCENT FINER THAN 0.002 mm**	**PERCENT FINER THAN 0.005 mm**	**PERCENT FINER THAN #200 SIEVE**	**LIQUID LIMIT (ASTM D4318)**	**PLASTICITY INDEX (ASTM D4318)**	**UNIFIED CLASSIFICATION (ASTM D2487)**	**SHRINKAGE LIMIT (ASTM D427)**	**SPECIFIC GRAVITY (ASTM D854)**	**EXPANSION INDEX (ASTM D4829)**	**POTENTIAL EXPANSION (ASTM D4829)**	**NUMBER OF NATURAL MOISTURE CONTENT TEST SAMPLES[1]**	**HIGHEST MEASURED NATURAL MOISTURE CONTENT (%)**	**LOWEST MEASURED NATURAL MOISTURE CONTENT (%)**	**EXPANSIVE? (Residential Code of NYS, Section R403.1.8.1)**
4	6 – 9	62	81	100	50	26	CH	19	2.74	93	HIGH	14	30	23	YES
8[2]	6 – 7.5	62	82	97	51	28	CH	19	2.74	94	HIGH	3	27	28	YES
9	≈ 7	53	76	100	45	24	CL	18	2.71	72	MEDIUM	1	25	25	YES
10	≈ 7	43	60	100	35	16	CL	16	2.72	52	MEDIUM	1	23	23	YES
11	≈ 6	54	74	100	41	21	CL	16	2.69	67	MEDIUM	1	26	26	YES
12	≈ 6	62	83	100	48	26	CL	17	2.75	82	MEDIUM	1	30	30	YES
13[3]	≈ 6	18	21	64	20	6	CL - ML	12	2.76	10	VERY LOW	1	14	14	NO
14	≈ 7	53	68	90	46	25	CL	17	2.76	81	MEDIUM	1	26	26	YES
16	7 – 7.5	No test	No test	No test	57	33	CH	No test	No test	No test	No test	2	29	28	No test
18	5.5 – 7.5	64	83	95	49	27	CL	17	2.76	78	MEDIUM	7	30	23	YES
20	7 – 10	69	90	100	55	31	CH	20	2.78	122	HIGH	6	34	24	YES
27[4]	≈ 6.5	35	49	94	30	13	CL	14	2.73	40	LOW	1	16	16	NO
28	≈ 1.5	67	83	95	56	30	CH	18	2.75	118	HIGH	1	30	30	YES
31	8 – 10	No test	No test	No test	46	25	CL	No test	2.69	No test	No test	1	25	25	No test
MAX[5]		69	90	100	56	31		20	2.78	122					
MIN[5]		43	60	90	35	16		16	2.69	52					
AVERAGE[5]		59	78	98	48	25		17	2.74	86					
STANDARD DEVIATION[5] (+/-)		8	9	3	6	4		1	0.03	22					
Residential Code Criteria Defining Expansive Soils			> 10	> 10		≥ 15				> 20					

[1] Discrete samples were collected within the depth range listed in column 2; [2] No significant damage observed at Site 8; [3] Soil consisted of glacial till; [4] Soil consisted of glacial till; [5] Excludes sites 13, 16, 27, and 31

Table 10. Laboratory test results for undisturbed samples of stiff foundation soils in Amherst, NY (2004)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
SITE	SAMPLE DEPTH (FEET)	CLAY FRACTION – PERCENT FINER THAN 0.002 mm	PERCENT FINER THAN #200 SIEVE	LIQUID LIMIT (ASTM D4318)	PLASTICITY INDEX (ASTM D4318)	UNIFIED CLASS.	DISTURBED SHRINKAGE LIMIT (ASTM D427)	EXPANSION INDEX (ASTM D4829)	POTENTIAL EXPANSION (ASTM D4829)	DRY UNIT-WEIGHT (POUNDS PER CUBIC FOOT)	EXPANSIVE? (Residential Code of NYS, Section R403.1.8.1)	NATURAL MOISTURE CONTENT (%)	INITIAL SATURATIO N (%)	UNDISTURBED SHRINKAGE LIMIT[1]	S_v[2]	$S_{v(theor)}$[3]
9	≈ 7	53	100	45	24	CL	18	72	MEDIUM	100	YES	25.1	100	18	0.53	0.54
10	≈ 7	43	100	35	16	CL	16	52	MEDIUM	106	YES	22.6	100	20	0.59	0.57
11[4]	≈ 6	54	100	41	21	CL	16	67	MEDIUM		YES					
a	≈ 6									101		25.1	100	17	0.66	0.54
b	≈ 6									100		26.0	100	19	0.59	0.53
12	≈ 6	62	100	48	26	CL	17	82	MEDIUM	94	YES	29.7	100	19	0.68	0.50
14	≈ 7	53	90	46	25	CL	17	81	MEDIUM	100	YES	25.8	100	17	0.63	0.53
MAX		62	100	48	26		18	82		106		29.7		20	0.68	0.57
MIN		43	90	35	16		16	52		94		22.6		17	0.53	0.50
AVERAGE		53	98	43	22		17	71		100		25.7		18	0.61	0.54
STANDARD DEVIATION (+/-)		7	4	5	4		1	12		4		2.3		1	0.05	0.02

[1] Determined via shrink test; [2] Vertical shrink-swell coefficient determined via shrink test; [3] Theoretical vertical shrink-swell coefficient; [4] One grab sample and two undisturbed samples, samples a & b, were collected at Site 11

Table 11. Laboratory test results for upper portion of soft stratum in Amherst, NY (2004)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
SITE	COMPOSITE SAMP LE DEPTH (FEET)	CLAY FRACTION – PERCENT FINER THAN 0.002 mm	PERCENT FINER THAN 0.005 mm	PERCENT FINER THAN #200 SIEVE	LIQUID LIMIT (ASTM D4318)	PLASTICITY INDEX (ASTM D4318)	UNIFIED CLASSIFICATION (ASTM D2487)	SHRINKAGE LIMIT (ASTM D427)	SPECIFIC GRAVITY (ASTM D854)	EXPANSION INDEX (ASTM D4829)	POTENTIAL EXPANSION (ASTM D4829)	NUMBER OF NATURAL MOISTURE CONTENT TEST SAMPLES[1]	HIGHEST MEASURED NATURAL MOISTURE CONTENT (%)	LOWEST MEASURED NATURAL MOISTURE CONTENT (%)	EXPANSIVE? (Residential Code of NYS, Section R403.1.8.1)
4	9 – 13	70	90	100	53	27	CH	21	2.76	115	HIGH	10	46	30	YES
5	8 – 11	62	85	100	47	25	CL	18	2.76	92	HIGH	10	39	30	YES
7	8 – 12	70	94	100	55	29	CH	20	2.76	128	HIGH	14	43	29	YES
8[2]	9 – 12	70	93	100	53	28	CH	20	2.79	126	HIGH	4	48	34	YES
18	7 – 8.5	58	76	90	49	26	CL	18	2.77	110	HIGH	3	35	28	YES
20	10 – 11.5	72	96	100	55	30	CH	21	2.78	124	HIGH	2	37	35	YES
MAX		72	96	100	55	30		21	2.79	128			48	35	
MIN		58	76	90	47	25		18	2.76	92			35	28	
AVERAGE		67	89	98	52	28		20	2.77	116			41	31	
STANDARD DEVIATION (+/-)		6	7	4	3	2		1	0.01	14			5	3	
Residential Code Criteria Defining Expansive Soils			> 10	> 10		≥ 15				> 20					

[1] Discrete samples were collected within the depth range listed in column 2; [2] No significant damage observed at Site 8

Table 12. Laboratory test results for consolidation test samples obtained from soft stratum in Amherst, NY

1	2	3	4	5	6	7	8	9	10
SITE	SAMPLE DEPTH (FEET)	NATURAL MOISTURE CONTENT (%)	LIQUID LIMIT (ASTM D4318)	PLASTICITY INDEX (ASTM D4318)	UNIFIED CLASSIFICATION (ASTM D2487)	APPROXIMATE IN- SITU EFFECTIVE STRESS (POUNDS PER SQUARE FOOT)	APPROXMATE PRECONSOLIDATION STRESS (POUNDS PER SQUARE FOOT)	COMPRESSION RATIO[1]	RECOMPRESSION RATIO[2]
29	14 – 16	49	51	29	CH	1400	2600	0.26	0.025
30	15 – 17	38	48	26	CL	1500	3000	0.20	0.021
31	20 – 22	48	52	29	CH	2000	2000	0.17	0.020
32	20 – 22	46	39	24	CL	2000	2000	0.22	0.018
29	26 – 28	49	49	29	CL	2000	2000	0.15	0.015
MAX		49	52	29				0.26	0.025
MIN		38	39	24				0.15	0.015
AVERAGE		46	48	27				0.20	0.020
STANDARD DEVIATION (+/-)		5	5	2				0.04	0.004

[1] Slope of strain vs. log pressure for virgin portion of consolidation test curve; [2] Slope of strain vs. log pressure for reloading portion of consolidation test curve

Table 13. Calculated post-construction settlement/rebound due to strain response of soft stratum at Site 29 in Amherst, NY

LOCATION IN BASEMENT	CASE I (INCHES)	CASE II (INCHES)	CASE III (INCHES)	CASE IV (INCHES)
Center	0.3 upward	0.2 upward	0.2 downward	0.8 downward
Wall Midpoint	0.2 upward	0.1 downward	0.6 downward	1.3 downward
Corner	0.1 upward	0.5 downward	0.8 downward	1.7 downward

[I] 50% of excavation rebound occurs after house construction

[II] 50% 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

[III] 50% of excavation rebound occurs after house construction and water table drops 4 feet after construction

[IV] 50% 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 14. Calculated post-construction angular distortion due to strain response of soft stratum at Site 29 in Amherst, NY

LOCATIONS IN BASEMENT	CASE I	CASE II	CASE III	CASE IV	ALLOWABLE
Between Center and Corner	1 / 1030	1 / 386	1 / 383	1 / 280	1/240[a]
Between Wall Midpoint and Corner	1 / 2260	1 / 463	1 / 707	1 / 551	1/1500[b]

[I] 50% of excavation rebound occurs after house construction

[II] 50% 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

[III] 50% of excavation rebound occurs after house construction and water table drops 4 feet after construction

[IV] 50% 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

[a] Allowable angular distortion between footings supporting wood framing (Meehan and Karp, 1993); [b] Allowable angular distortion along 30’ long x 7’ high unreinforced basement wall (Poulos et al., 2002).

Photo 5. Lateral pressure affecting basement wall in central Amherst, NY. Wall has deflected inward nearly 3 inches from grounding rod.

Photo 6. Lateral pressure causing vertical fracture in mid-span of basement wall in north-central Amherst, NY.

Photo 7. Undisturbed sampling of stiff stratum in north Amherst, NY.

Photo 8. Heterogeneity in shallow soil conditions in north Amherst, NY. Darker soils are clay and lighter brown are fine sand. Note baking of footing soils (c.f., Photo 12).

Photo 9. Root hairs penetrating into sump pit in central Amherst, NY.

Photo 10. Construction of interior and perimeter footings on stiff stratum in north-central Amherst, NY.

Photo 11. Perimeter loading of house with 3 to 4 feet of fill in northAmherst, NY.

Photo 12. Winter foundation site in East Amherst showing potential for softening from frozen and saturated conditions (March, 2004).

Photo 13. Inward deflection of basement wall (and pilasters) in central Amherst NY. Plum-bob indicates 9" of inward movement. Pipes are damaged and/or relocated.

Photo 14. Erosion of strip footing during construction.

SECTION 4 – SUMMARY AND CONCLUSION

4.1 Summary

This section summarizes the major findings related to extent and scope and causative factors.

4.1.1 Extent and Scope

- Foundation failure is a relatively common problem for residential structures built in expansive soils throughout the world, United States and Amherst, New York.
- Fine-grained lacustrine soils cover much of Amherst, Erie County and western New York.
- Fine-grained lacustrine soils in Amherst have medium to high potential expansion (ASTM D4829).
- Soil boring data indicates central and northern Amherst is underlain by a soft stratum.
- Since 1987, a total of 1,095 homeowners either (a) received a foundation repair permit (501) or (b) made a foundation-related inquiry (594). However, many homeowners are reluctant to contact public officials because of the potential impact to their property value.
- The current damage rate for houses on lacustrine soils is about 3 percent (assumes 31,000 total foundations), but some affected neighborhoods report damage rates that are an order of magnitude greater.
- The average house receiving a foundation repair permit is 41 years old, but the onset of problems can occur from a few to nearly 50 years after construction, with an estimated hiatus of about 20 years.
- The average total repair cost as indicated from repair permits is about $7,900, but the range is about $500 to $71,000.
- Utility company data did not provide a secondary indication of affected areas.
- A small but significant number of foundation repairs have not performed as expected.
- We judge the number of repair permits will increase and may approach 2,000, but the timeframe is uncertain because of several unpredictable factors.

4.1.2 Causative Factors

- Foundation damage generally results from lateral pressure and/or differential settlement.

- 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.
- Historical foundation designs do not appear to account for potential lateral pressures and settlements. Finite-element analyses suggest that historical basement wall design did not adequately consider potential lateral pressures.
- About 93% of inspected houses had basement walls that lacked adequate lateral support at the top.
- Nearly 58% of blueprints did not match the structure built, but only in a few cases were the modifications considered potential causative factors.
- Inadequate concrete strength was not a significant causative factor.
- Stiff, fine-grained lacustrine foundation soils are expansive and may contribute to differential movements of the overlying house as laterally variable changes in foundation soil moisture content occur.
- Lateral variation in foundation soil moisture content was confirmed at several houses in Amherst.
- Post-construction moisture content changes in stiff clayey lacustrine foundation soils are generally controlled by four factors including, 1) concentration and mineralogy of clay in the soil, 2) water availability, 3) confining pressure, and 4) initial moisture content.
- Typical conditions in Amherst promote laterally variable changes in foundation soil moisture content.
- Differential straining of underlying soft lacustrine soils can cause significant differential movements of the overlying house foundation. Three common events may contribute to significant differential straining of the soft stratum including; (1) removal of soil from basement excavations during construction, (2) raising lot elevation with significant amounts of new fill around the perimeter of the house, and (3) long-term lowering of the groundwater level.

4.2 Conclusion

The vast majority of houses in Amherst are apparently performing as expected. Nonetheless, an anomalous number of homeowners (1095) have reported slight to severe foundation-related damage. The majority of houses are located north of Main Street and within lacustrine soils. Lateral pressures and/or settlement are the principal causes of foundation damage. No single causative factor accounts for the variety of damages we observed. Expansive soils, compressible substrata, post-construction hydrologic modification, marginally effective foundation design, poor construction, and inadequate observation/documentation are all potential contributing factors at most sites.

Risks associated with building on expansive soils (and bedrock) have been well known for decades in such western states as Colorado, Texas and California; however, experience with expansive soil in the Northeast is relatively uncommon. Unlike western states, soils in Amherst are generally moister, contain non-smectitic clays (illite and chlorite), and the houses have full basements. In both environments, laterally variable changes to the soil moisture content across the foundation footprint are a primary concern.

We agree with Meehan and Karp (1994) that the design of shallow residential or other lightly-framed foundations on expansive soils is an art which often presents more difficulties than design of foundations for heavy loads. Traditional design criteria, such as bearing capacity, are not relevant. These simple facts may be recognized by only a few in the building business. The importance of proper implementation of design and engineering inspections and other verification during construction cannot be overemphasized (Meehan and Karp, 1994). We concur that improvements in preventative design practices are less a matter of better advanced theory than of information dissemination, development of coherent quality standards, and coordination among practicing professionals and the construction industry.

The Residential Code of New York State (NYSDOS, 2003) does not provide in-depth guidance regarding design, construction, assessment, and repair of foundations in these soils conditions. We conclude the town of Amherst must develop some additional guidelines for design/construction and assessment/repairs.

SECTION 5 – RECOMMENDATIONS

5.1 PRIMARY

In response to problems with residential foundations not unlike Amherst's, the Texas Section of the American Society of Civil Engineers (ASCE) published guidelines entitled, *Recommended Practice for the Design of Residential Foundations* and *Guidelines for the Evaluation and Repair of Residential Foundations* (Texas ASCE, 2002, 2002a). These guidelines, which are not standards, supplement building codes and provide guidance for residential foundation design, construction, and repair. We recommend that Amherst take this holistic approach and develop and adopt similar guidelines. Our primary recommendations are as follows:

1. The Town should develop and adopt new guidelines for the design and construction of residential foundations in Amherst to augment the *Residential Code of New York State* (NYSDOS, 2003). In general, the guidelines should facilitate construction of *engineered foundations* that are designed based on a *site-specific geotechnical engineering evaluation*. Using the findings of the geotechnical engineering evaluation, foundation *design should be performed by a licensed engineer.* The licensed engineer who designs the foundation should be considered *the "engineer of record,"* and should design the foundation to ensure *long-term performance.* Foundation *construction should be observed and documented* to ensure that the foundation is constructed in accordance with the provisions of the foundation design (Texas ASCE, 2002). Appendix 6.7 provides a working draft of the proposed guidelines for Amherst.

2. The Town should develop and adopt new guidelines for the assessment and repair of residential foundation damage in Amherst. In general, the guidelines should facilitate *engineered solutions* that are developed *based on a site-specific engineering evaluation*. Using the findings of the engineering evaluation, design of foundation repairs should be *performed by a licensed engineer*. The licensed engineer who designs the foundation repair should be considered the *"engineer of record,"* and should design the foundation repair to ensure *long-term performance*. Foundation *repair should be observed and documented* to ensure that the foundation repair is constructed in accordance with the provisions of the design. Appendix 6.8 provides a working draft of these guidelines for consideration by Amherst.

3. Homeowners (and homebuyers) north of Main Street should (a) review homeowners guides (Section 1.4), (b) review maps of foundation-related damages (this report or Building Department), (c) be familiar with the aforementioned proposed guidelines (Appendix 6.7 and 6.8), (d) perform bi-annual house inspections (Appendix 6.9), and, when appropriate, retain a licensed engineer to obtain a diagnosis and plan for remediation.

5.2 SECONDARY

Beyond the major recommendations to develop guidelines for design/construction and assessment/repairs, we suggest the Town consider the following secondary recommendations:

1. *Town of Amherst Soils Workshop* – the Town should sponsor a workshop/workgroup to promote dialogue, education, training, and the continued development of the design/construction and assessment/repairs guidelines.

2. *Homeowners' Website* – the Town should develop/sponsor a clearinghouse website dedicated to information exchange about foundation-related topics in Amherst. This site would be the warehouse of new information and allow "one-stop" shopping. Homeowner concern could be greatly reduced, if for example, homeowners with related problems/concerns could exchange information about engineers, contractors, cost estimates, remedial solutions, and alternative methods.

3. *Standardize Data* – the Town should standardize its terminology and methodology for data collection. The Town collects data (foundation inquiries, home inspections, foundation repair permits, plumbing inspections, utility repairs, etc.) that would be more useful if terminology and methodology were standardized and keyed to extent, scope, and causative factors.

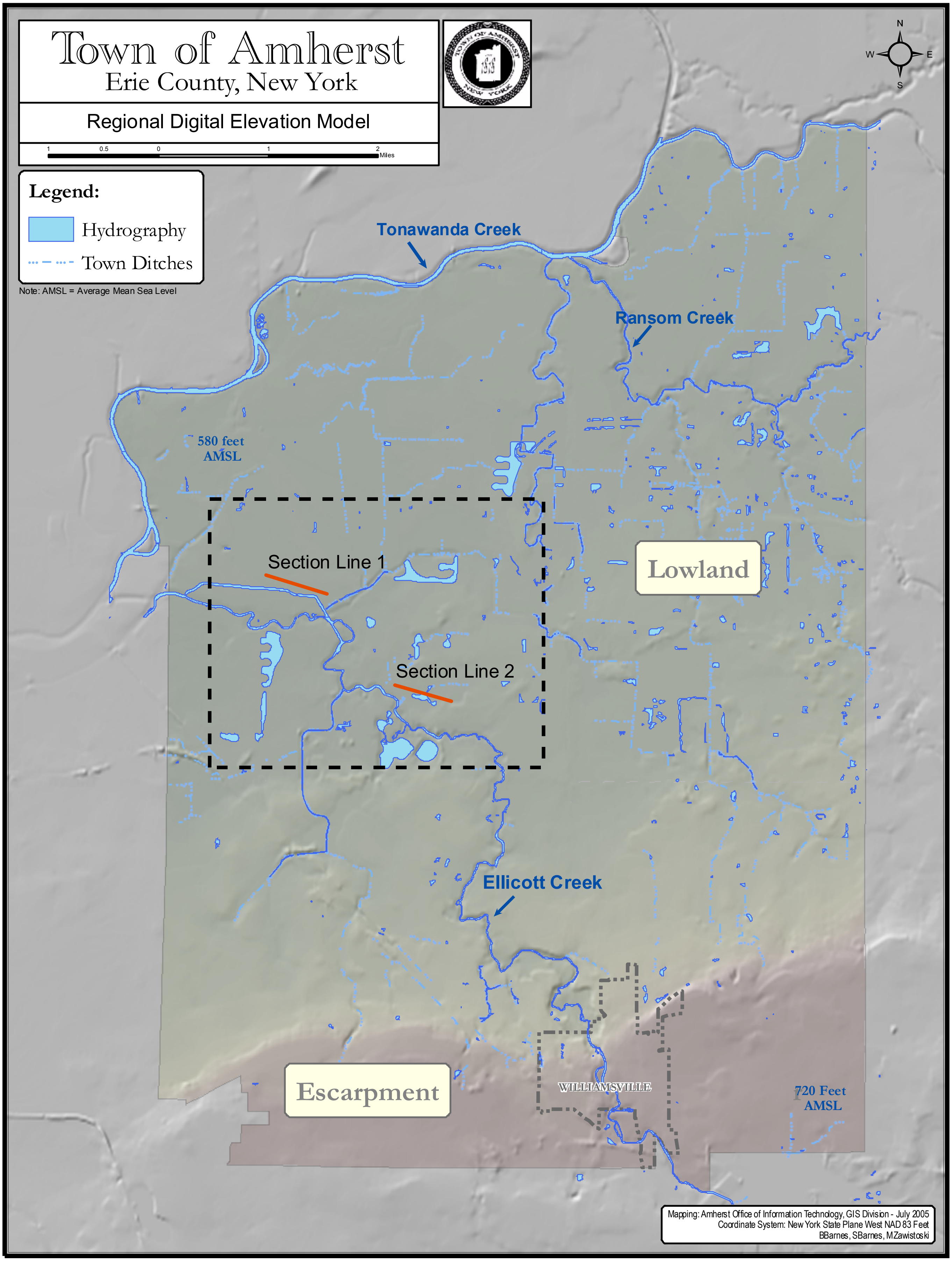
4. *Homeowners/homebuyers Guide* – the Town should develop a homeowners/homebuyers guide specific to Amherst and the surrounding communities. The guide might be used in conjunction with a local ordinance that alerts homebuyers to the problems of buying and maintaining a house built on expansive/compressible soils.

5. *Knowledge-based Systems* – the Town should consider a knowledge-based system such as the Subsidence Case Management System (SCAMS), which is used by the UK to provide guidance for engineers dealing with subsidence cases at all stages – from initial diagnosis to remedial measures (Anumba and Scott, 2001).

6. *Repair evaluation* – the Town should consider evaluating the performance of foundation repair methods commonly used in Amherst.

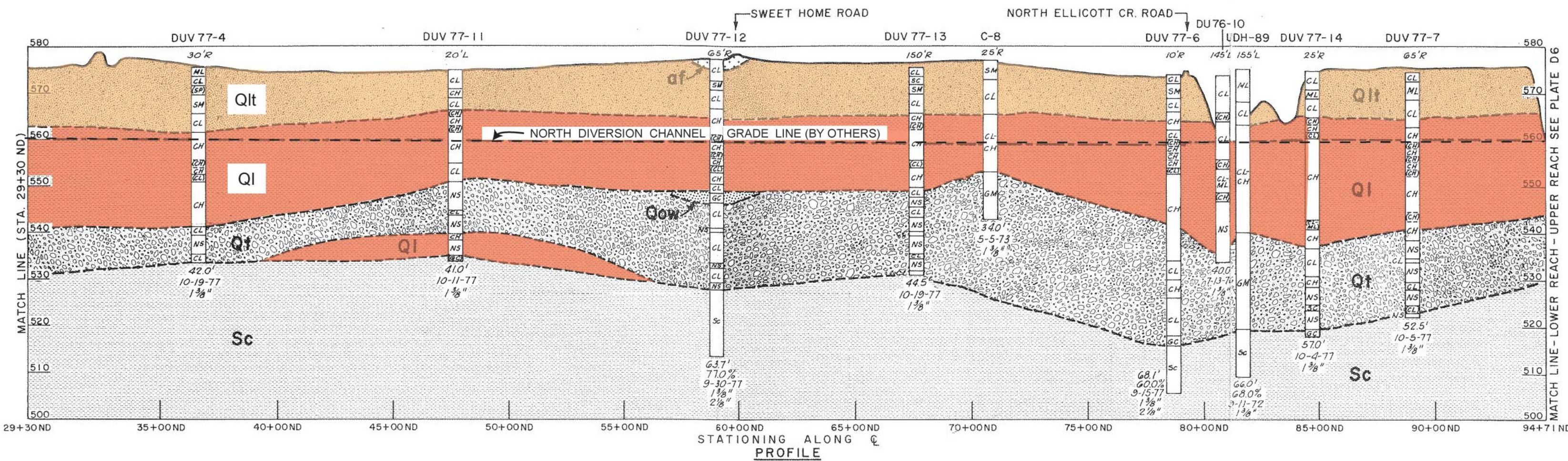
SECTION 6 – APPENDICES

6.1 GEOLOGIC CROSS SECTION

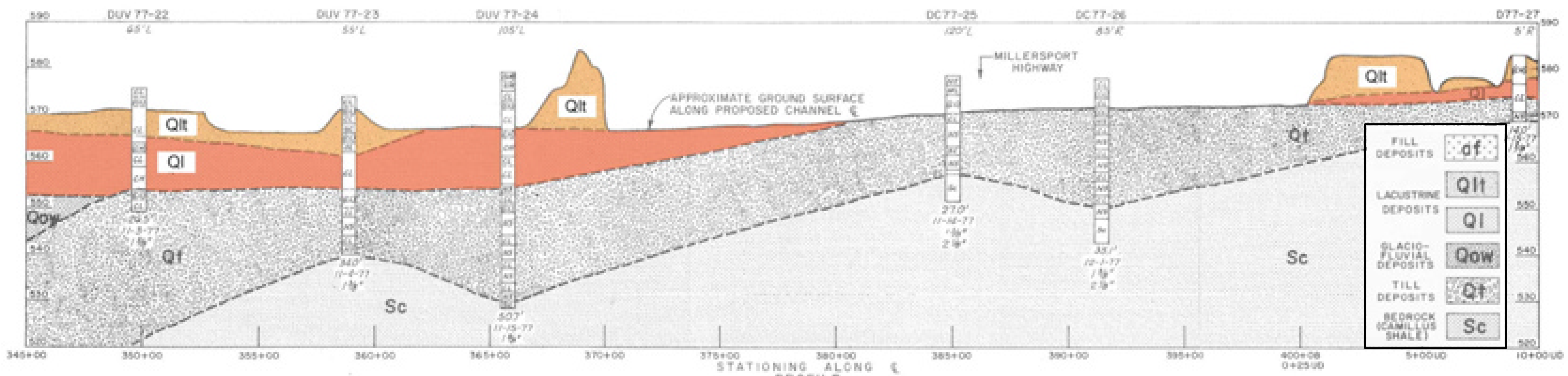


Appendix 6.1.1: Approximate location of cross sections shown in Appendix 6.1.2.

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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January 24, 2005
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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

					%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**

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)**
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.

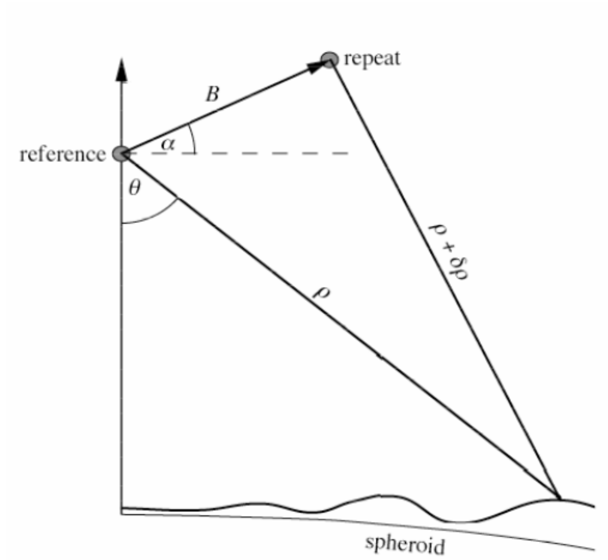


Figure 1: Geometry of Repeat Pass Interferometry [*Sandwell and Price*, 1998]

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.

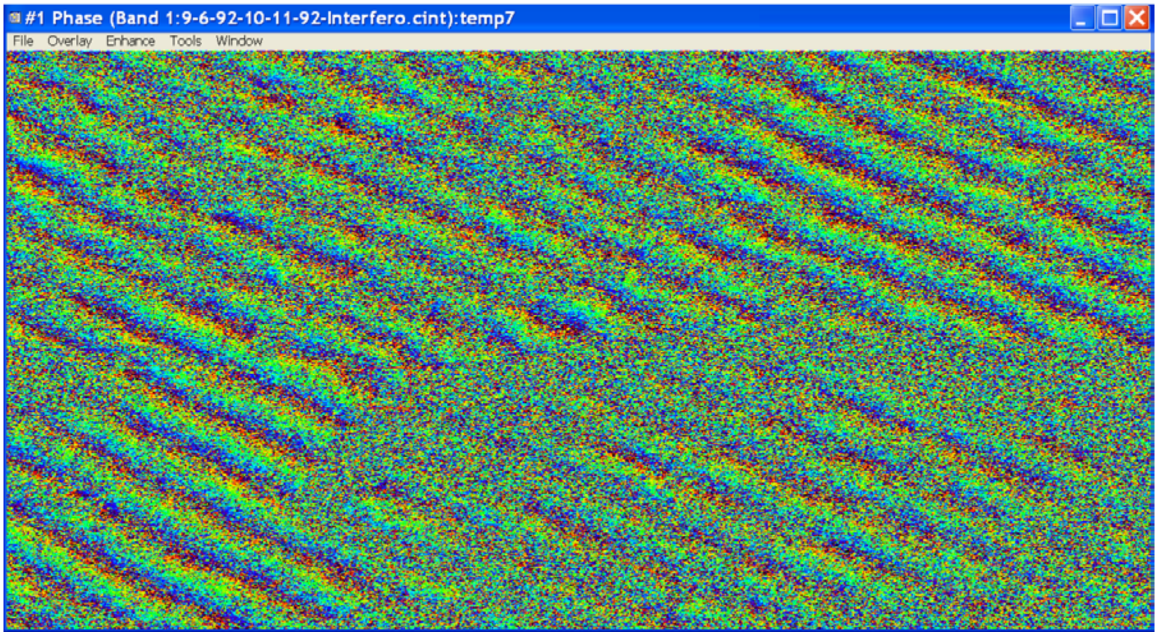


Figure 2: Sample interferogram generated from a pair of radr images acquired on 9-2-92 and 10-31-92.

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.

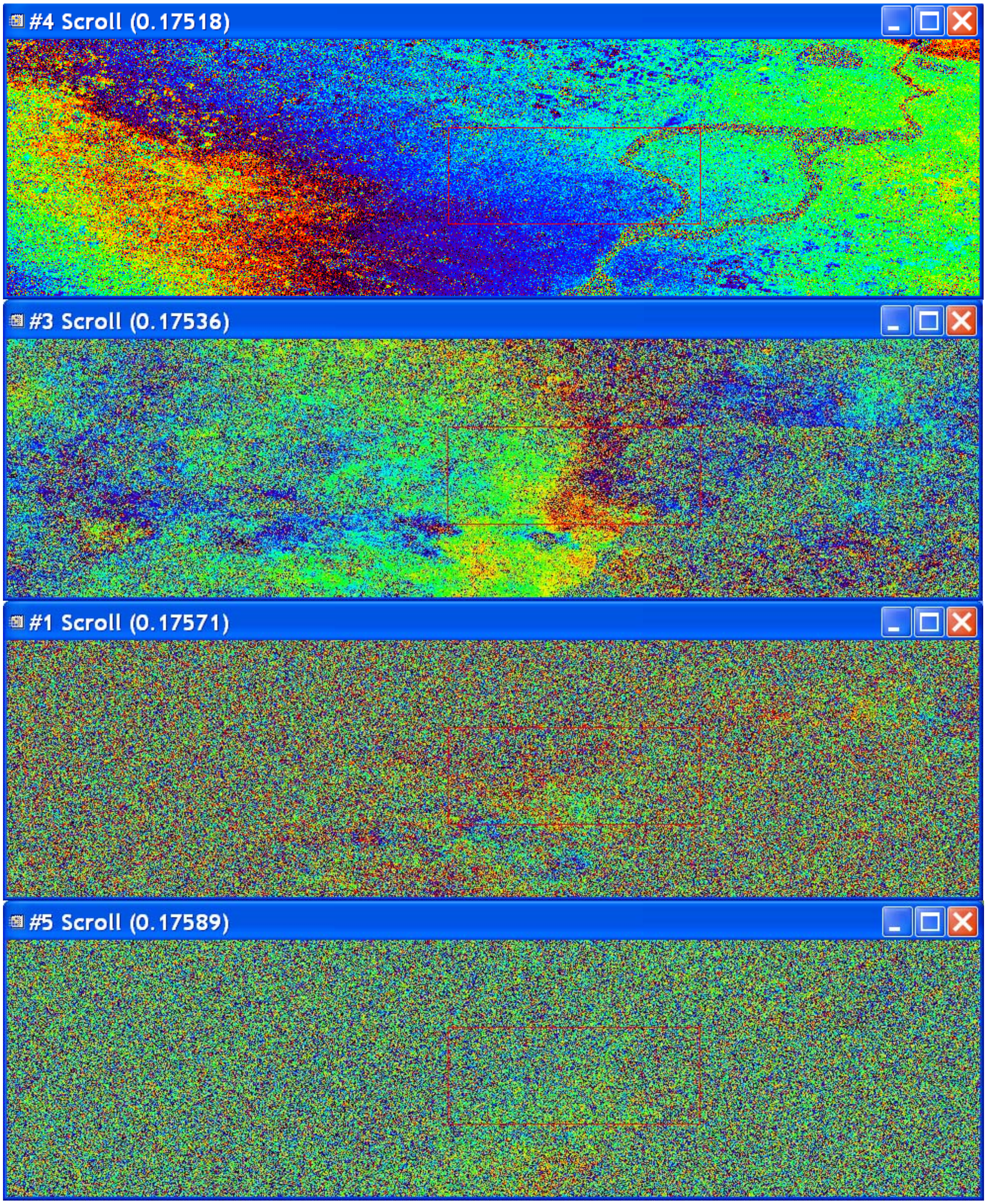


Figure: 3 Series of interferograms showing loss of coherence with increased temporal separation, even with good spatial basline. (From Top, Master:11-28-95 Slave:11-29-95, $B_{perp=}$43m; M: 10-11-92, S:9-6-92, $B_{perp=}$-17m; M:10-11-92, S:10-24-95 $B_{perp=}$273m; M: 10-11-92, 3-12-03, $B_{perp=}$54m) . 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.

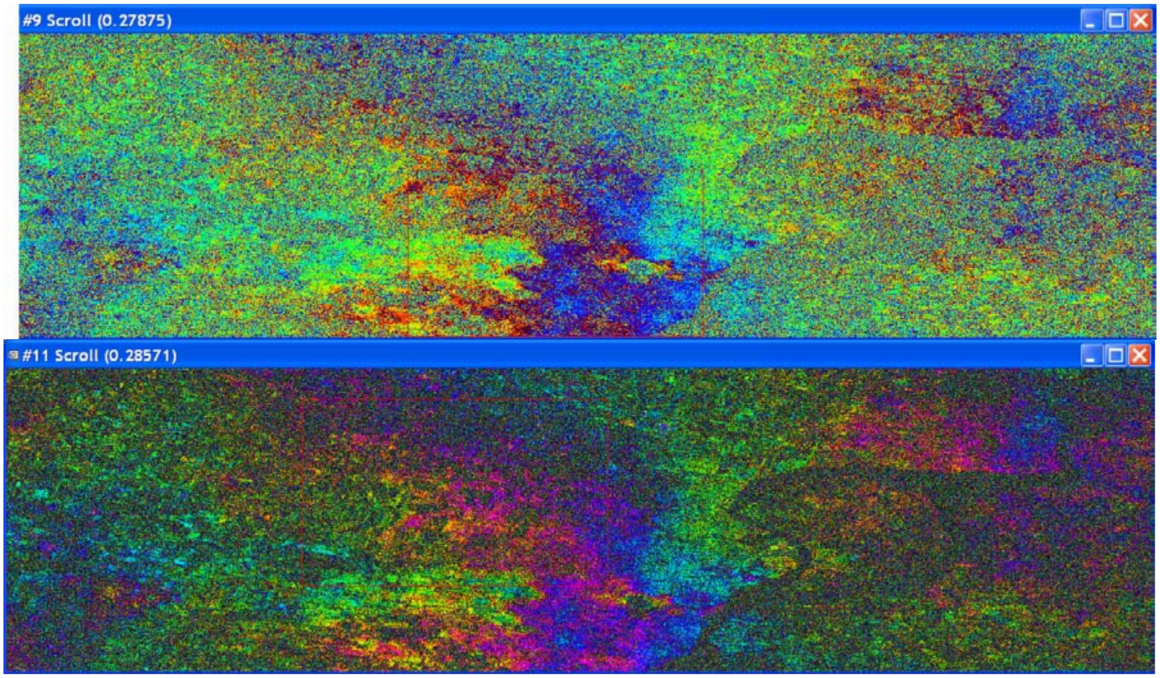


Figure 4: Results from the 4-Pass DINSAR, unfiltered (above), and filtered based on coherence (below)

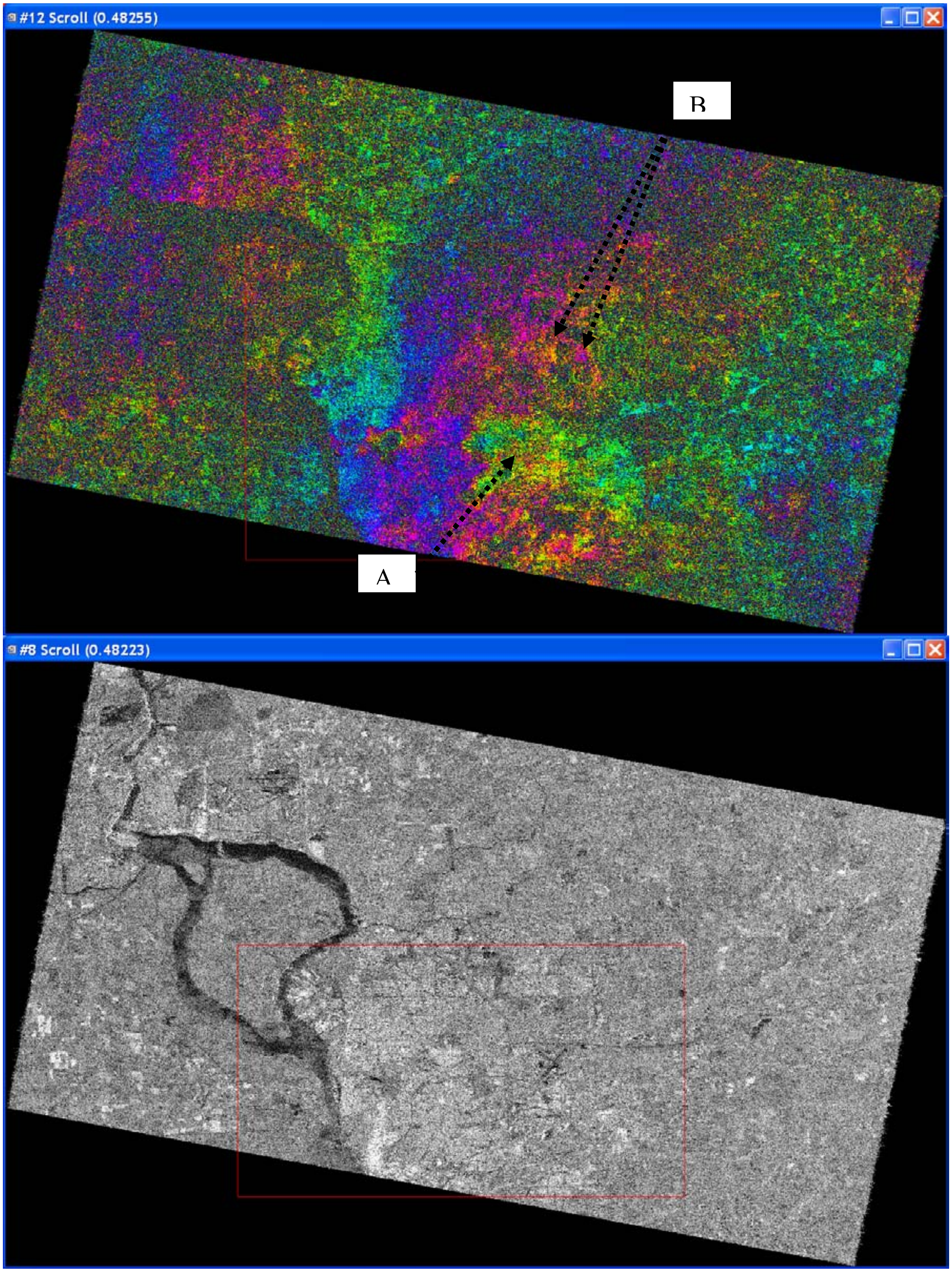


Figure 5: 4-Pass DINSAR result (phase), resampled to UTM (above) and reference image (below) showing areas of interest in DINSAR result.

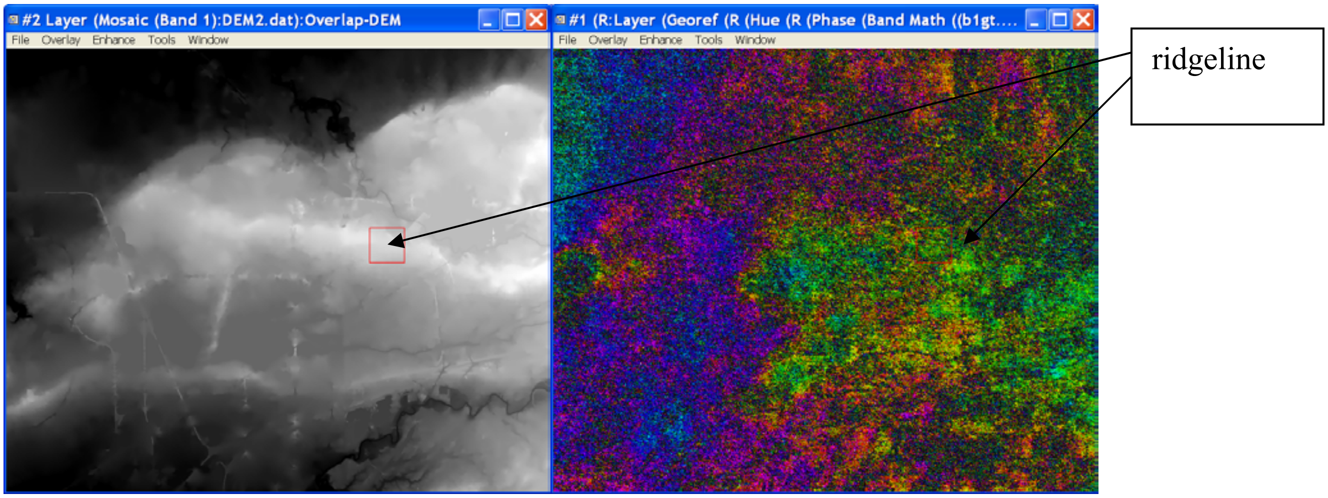


Figure 6: Comparison between DEM mode (left) and DINSAR (right) shows that ridgeline aligns in part with the interferometric feature.

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.

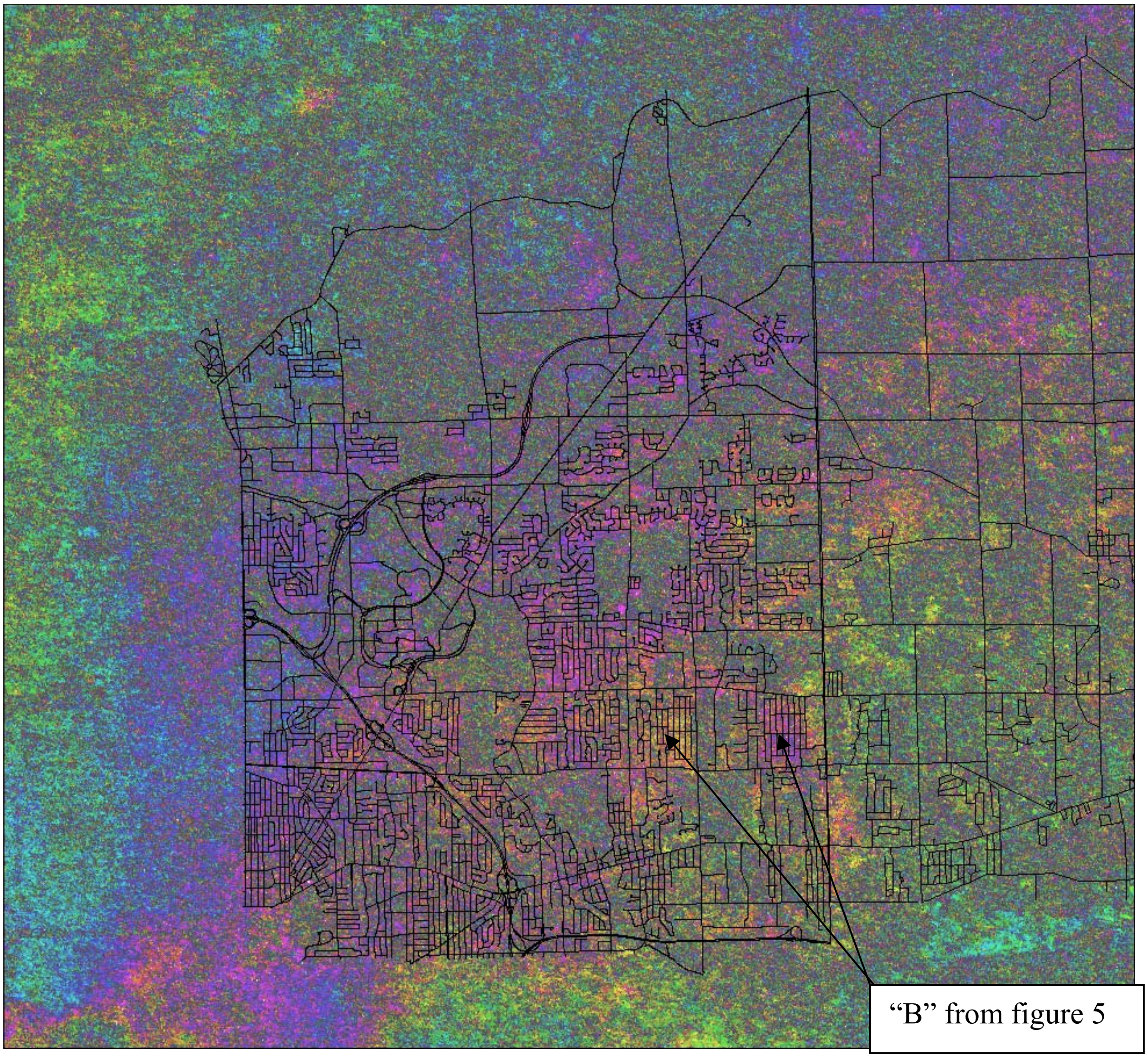


Fig. 7. Four-pass DINSAR deformation result over Amherst, NY, with street map overlain for reference

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:

Amelung, F., and S. Day, InSAR observations of the 1995 Fogo, Cape Verde, eruption: Implications for the effects of collapse events upon island volcanoes, *Geophysical Research Letters*, *29* (12), 2002.

Burbey, T.J., Use of time-subsidence data during pumping to characterize specific storage and hydraulic conductivity of semi-confining units, *Journal of Hydrology*, *281* (1-2), 3-22, 2003.

Carnec, C., and C. Delacourt, Three years of mining subsidence monitored by SAR interferometry, near Gardanne, France, *Journal of Applied Geophysics*, *43* (1), 43-54, 2000.

Chen, C.W., and H.A. Zebker, Phase unwrapping for large SAR interferograms: Statistical segmentation and generalized network models, *Ieee Transactions on Geoscience and Remote Sensing*, *40* (8), 1709-1719, 2002.

Ferretti, A., C. Prati, and F. Rocca, Nonlinear subsidence rate estimation using permanent scatterers in differential SAR interferometry, *Ieee Transactions on Geoscience and Remote Sensing*, *38* (5), 2202-2212, 2000.

Ferretti, A., C. Prati, and F. Rocca, Permanent scatterers in SAR interferometry, *Ieee Transactions on Geoscience and Remote Sensing*, *39* (1), 8-20, 2001.

Gabriel, A.K., R.M. Goldstein, and H.A. Zebker, Mapping Small Elevation Changes over Large Areas - Differential Radar Interferometry, *Journal of Geophysical Research-Solid Earth and Planets*, *94* (B7), 9183-9191, 1989.

Hoffmann, J., H.A. Zebker, D.L. Galloway, and F. Amelung, Seasonal subsidence and rebound in Las Vegas Valley, Nevada, observed by synthetic aperture radar interferometry, *Water Resources Research*, *37* (6), 1551-1566, 2001.

Kampes, B.M., R. Hanssen, and Z. Perski, Radar Interferometry with Public Domain Tools, in *Fringe 2003*, Frascati, Italy, 2003.

Lanari, R., O. Mora, M. Manunta, J.J. Mallorqui, P. Berardino, and E. Sansosti, A small-baseline approach for investigating deformations on full-resolution differential SAR interferograms, *Ieee Transactions on Geoscience and Remote Sensing*, *42* (7), 1377-1386, 2004a.

Lanari, R., G. Zeni, M. Manunta, S. Guarino, P. Berardino, and E. Sansosti, An integrated SAR/GIS approach for investigating urban deformation phenomena: a case study of the city of Naples, Italy, *International Journal of Remote Sensing*, *25* (14), 2855-2862, 2004b.

Massonnet, D., and K.L. Feigl, Radar interferometry and its application to changes in the earth's surface, *Reviews of Geophysics*, *36* (4), 441-500, 1998.

Refice, A., F. Bovenga, and R. Nutricato, Stepwise Approach to INSAR Processing of Multitemporal Datasets, in *Fringe 20003*, ESA, Frascati, Italy, 2003.

Sandwell, D.T., and E.J. Price, Phase gradient approach to stacking interferograms, *Journal of Geophysical Research-Solid Earth*, *103* (B12), 30183-30204, 1998.

Sandwell, D.T., L. Sichoix, and B. Smith, The 1999 Hector Mine earthquake, southern California: Vector near-field displacements from ERS InSAR, *Bulletin of the Seismological Society of America*, *92* (4), 1341-1354, 2002.

Scharroo, R., and P. Visser, Precise orbit determination and gravity field improvement for the ERS satellites, *Journal of Geophysical Research-Oceans*, *103* (C4), 8113-8127, 1998.

Zebker, H.A., and Y.P. Lu, Phase unwrapping algorithms for radar interferometry: Residue-cut, least-squares, and synthesis algorithms, *Journal of the Optical Society of America a-Optics Image Science and Vision*, *15* (3), 586-598, 1998.

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)

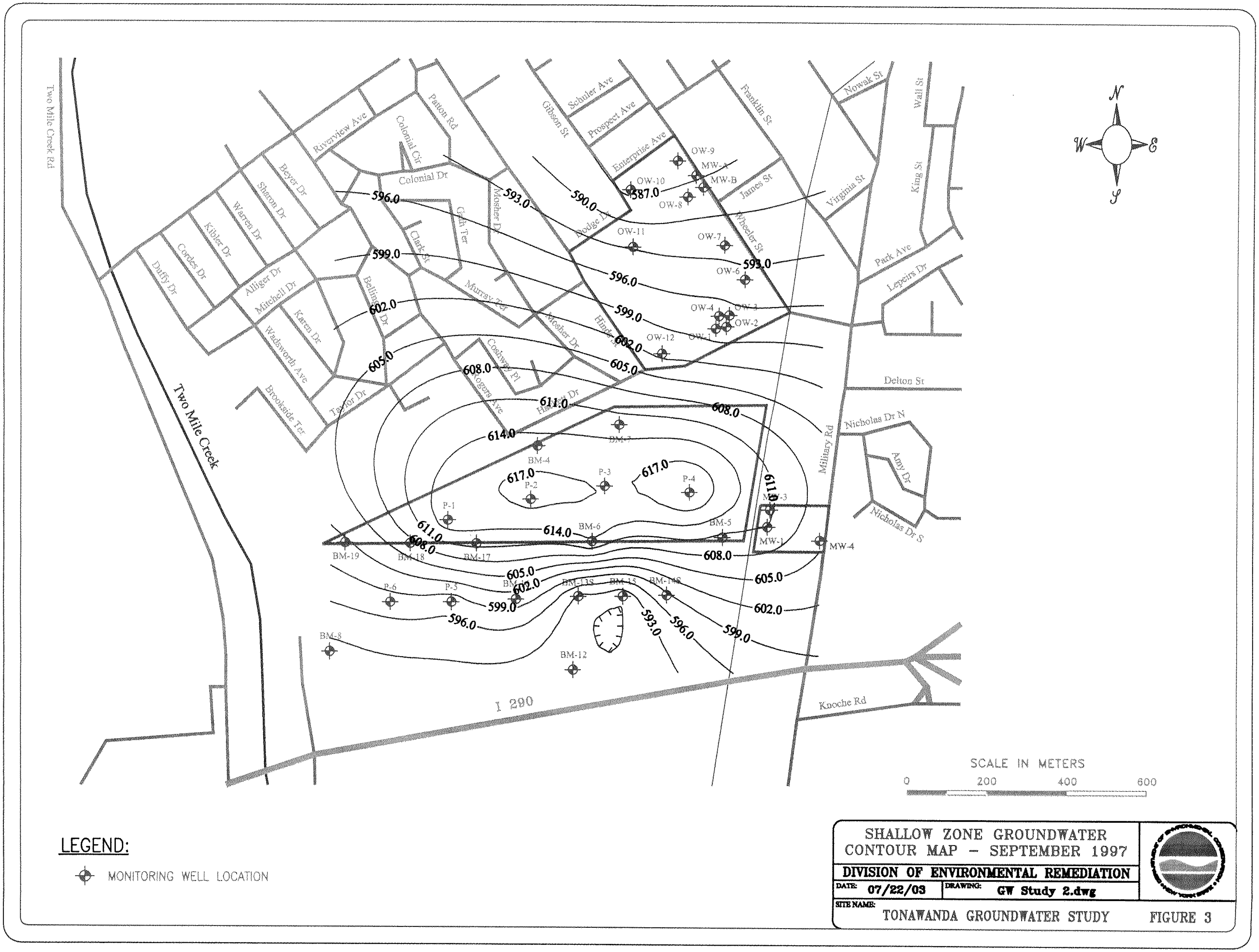
Based on long-term statewide weather records, Ohio receives an average of 38 inches of precipitation. These values would approximate much of Western New York.

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).



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
LEGEND:
MONITORING WELL LOCATION
SCALE IN METERS
0
200
400
600
N
E
S
W
Two Mile Creek Rd
Two Mile Creek
I 290
Military Rd
Knoche Rd
Riverview Ave
Colonial Cir
Colonial Dr
Patton Rd
Mosher Dr
Gibson St
Schuler Ave
Prospect Ave
Enterprise Ave
Dodge Ln
Franklin St
James St
Wheeler St
Virginia St
Nowak St
Wall St
King St
Park Ave
Lepeirs Dr
Delton St
Nicholas Dr N
Nicholas Dr S
Amy Dr
Beyer Dr
Sharon Dr
Warren Dr
Kibler Dr
Cordes Dr
Duffy Dr
Alliger Dr
Mitchell Dr
Karen Dr
Wadsworth Ave
Brookside Ter
Taylor Dr
Bellings Dr
Clark St
Glen Ter
Murray Ter
Coshway Pl
Rogers Ave
Hackett Dr
596.0
593.0
590.0
587.0
599.0
602.0
605.0
608.0
611.0
614.0
617.0
OW-9
MW-A
MW-B
OW-10
OW-8
OW-11
OW-7
OW-6
OW-4
OW-3
OW-2
OW-1
OW-12
BM-7
BM-4
P-1
P-2
P-3
P-4
MW-3
BM-5
BM-6
MW-1
MW-4
BM-19
BM-18
BM-17
BM-13S
BM-15
BM-14S
P-6
P-5
BM-8
BM-12

				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

depth below footing (feet)	thickness of interval (feet)	initial conditions for perimeter ftg. interior footing moisture content	perimeter footing moisture content	S_v 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

$$\Delta h = h_0 S_v \Delta w$$

$$h_0 = 12''$$

$$\Delta w = \frac{(29.7+28.4)}{2} - \frac{(25.8+27.5)}{2} = 2.4\% = 0.024$$

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

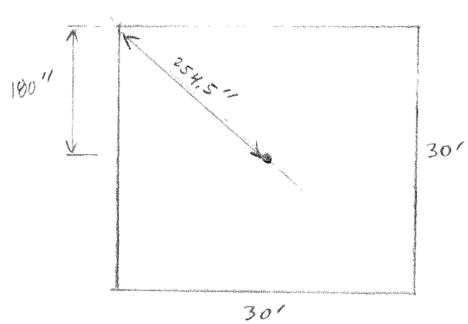
Site 7 - Perimeter Footing Shrinkage Calculations

depth below footing (feet)	thickness of interval (feet)	initial conditions for perimeter ftg. 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)	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)	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	$= 0.015\,(\log 1936 - \log 1957)$	-0.004	-0.050
				total		$\boxed{-0.043}$ → 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

$\underbrace{\qquad\qquad}_{\text{Average For 12' Thick Stratum} = 41.6\ PSF}$

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		$\boxed{-0.006}$

$\boxed{-0.006} \rightarrow$ 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	$\boxed{0.035}$			$\boxed{0.098}$

$1793.8 + 250 = 2043.8$ PSF

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

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

$1961.7 + 250 = 2211.7$

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

Final net compression 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 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	-0.015

→ 100% rebound

$L = 30'$

$B = 30'$

$z_{upper} = 9'$

$z_{lower} = 20'$

$B/z_{upper} = 3.33$

$B/z_{lower} = 1.5$

$L/z_{upper} = 3.33$

$L/z_{lower} = 1.5$

$I_{upper} = 0.24$

$I_{lower} = 0.215$

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

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

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

$\varepsilon_{lower} = 0.015(\log 1954 - \log 1957)$
$= 0.0003$

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		$\boxed{0.033}$

$\rightarrow$ Final net settlement due to excavation and perimeter filling

$1853.8 + 230 - 43.7 = 2040\ PSF$

$L = 30'$

$B = 30'$

$Z_{upper} = 15'$

$Z_{lower} = 26'$

$B/Z_{upper} = 2$

$B/Z_{lower} = 1.15$

$L/Z_{upper} = 2$

$L/Z_{lower} = 1.15$

$I_{upper} = 0.23$

$I_{lower} = 0.19$

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

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

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

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

	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
				0.063			0.132

$1853.8 + 250 = 2103.8$ PSF

$2040.1 + 250 = 2290.1$ PSF

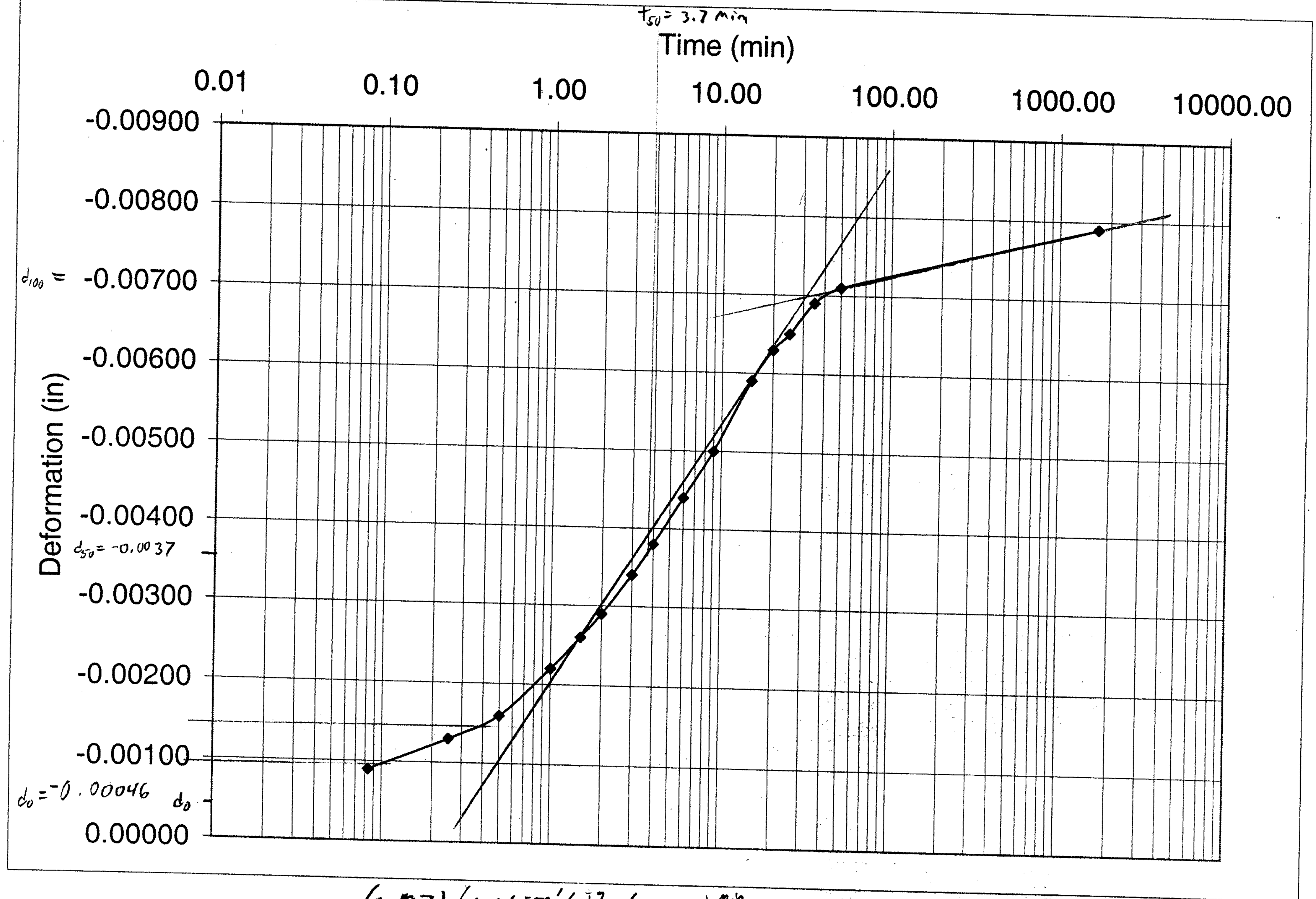
$\to$ Final net compression due to excavation, perimeter filling, and water table drop

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

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

Final net compression due to excavation & water table drop

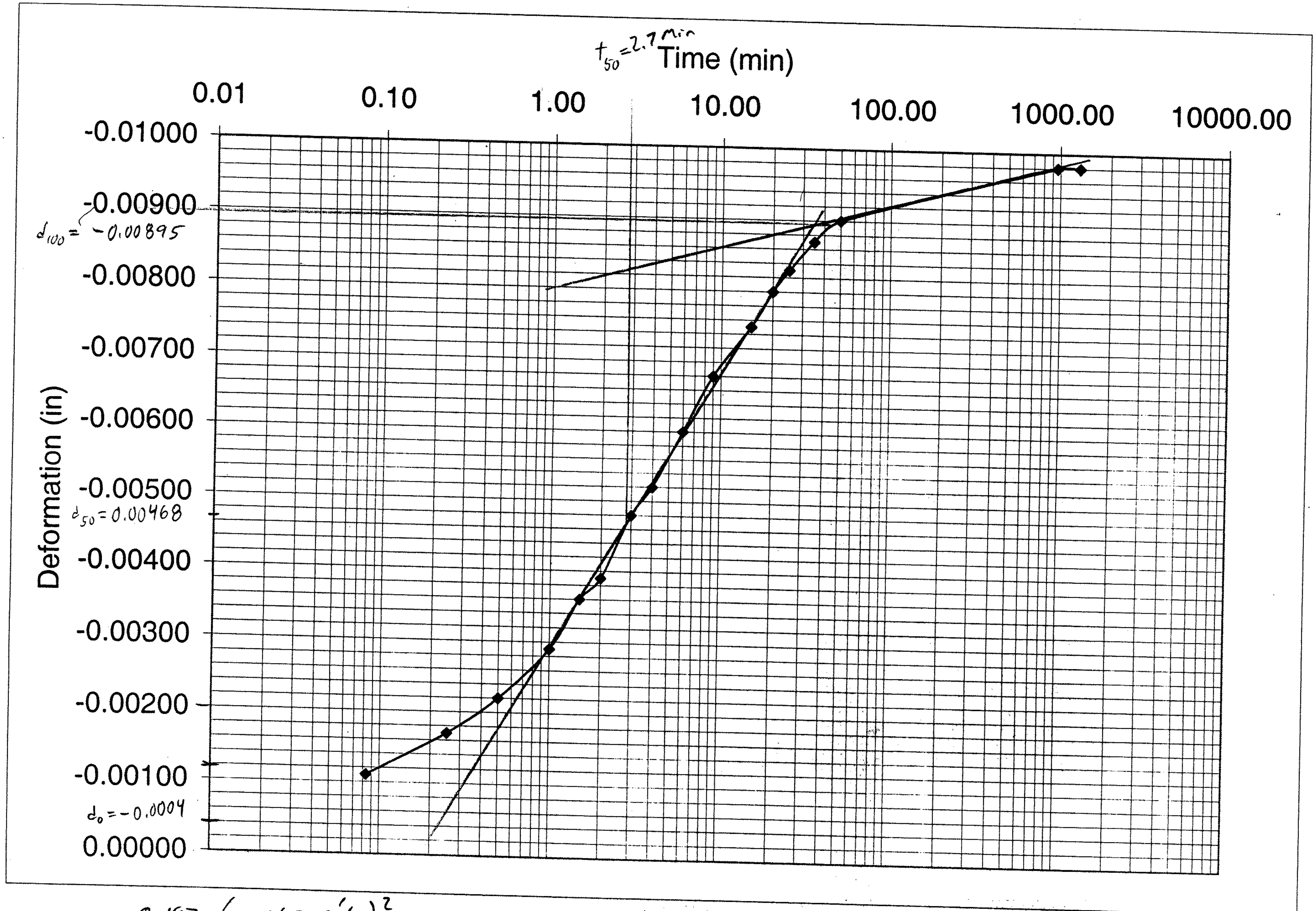
1500 PSF TO 500 PSF



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

$h_0 = 0.06268$

2000 PSF TO 500 PSF



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

US Army Corps of Engineers Buffalo District

Originator: ______ Page ____ of ____
Date: ______
Project: ______
Subject: ______

Time - Rate consolidation

-9

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

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

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

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

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

-20

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

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

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

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

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

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

90% compression

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

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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).

SECTION 7 – LITERATURE CITED

Adib, M.E., and Paratore, W.G. (1994). "Settlement of an Air Cargo Structure on Soft Clay." *Proc., Settlement '94*, ASCE, College Station, Texas, 875-886.

Amherst IDA, Industrial Development Agency (2005). *Amherst Population Statistics*, Amherst, N.Y., http://www.amherstida.com.

Anderson, J.N. and Lade, P.V. (1981). "The Expansion Index Test." *The American Society for Testing and Materials.*

Anumba, C.J. and Scott, D. (2001). "Intelligent Pathological Assessment of Housing Subsidence." *Proc. of the Institutions of Civil Engineers Structures & Buildings* 146, (2), 183-193.

ASCE, American Society of Civil Engineers (1995). *So Your Home is Built on Expansive Soils*, ASCE, New York.

Barron & Associates (1999). *Subsurface Explorations and Geotechnical Engineering Recommendations for Proposed Senior Citizens Center 370 John James Audubon Parkway Amherst, New York.* Consultant report to the Town of Amherst.

Biddle, G. (2001). "Tree Root Damage to Buildings" in Vipulanandan, C., Addison, M.B. and Hasen, M., *Expansive Clay Soils and Vegetative Influence on Shallow Foundations*. The Geo-Institute of the American Society of Civil Engineers, Houston, TX.

Brevik, E., Fenton, T., and Jaynes, D. (2003). "Evaluation of the Accuracy of a Central Iowa Soil Survey and Implications for Precision Soil Management." *Precision Agriculture*, (4) 331-342.

Briaud, J., Zhang, X., and Moon, S. (2003). "Shrink Test-Water Content Method for Shrink and Swell Predictions." *J. Geotechnical and Geoenvironmental Eng.*, 590-600.

Brooker, E.W. and Ireland, H.O. (1965). "Earth Pressures at Rest Related to Stress History." *Canadian Geotechnical Journal*, II(1), 1-15.

CMHC, Canada Mortgage and Housing Corporation (2004). "Understanding and Dealing with Interactions Between Trees, Sensitive Clay Soils and Foundations." Canada Mortgage and Housing Corporation, http://www.cmhc-schl.gc.ca (May 28, 2004).

D'Agostino, J.P. (1958). "Lake Tonawanda: History and Development." Masters Thesis, State University of New York at Buffalo, Amherst, N.Y.

Daigler, J.A. & Associates (2004). "Geotechnical Report: Saint Brendan's Court Subdivision." Consultant report to the Town of Amherst.

Daigler, J. A. & Associates (2004a). "Supplemental Geotechnical Report: Saint Brendan's Court Subdivision." Consultant report to the Town of Amherst.

Day, R.W. (1992). "Damage to Two Apartment Buildings Due To Moisture Variation of Expansive Soil." *J. Perf. Constr. Facil*., 6 (3) 169-176.

Diaz , C.F., Hadipriono, F.C. and Pasternack, S. (1994). "Failures of Residential Building Basements in Ohio." *J. Perf. Constr. Facil.*, 8(1), 65-80.

DPWES, Department of Public Works & Environmental Services (2002). "Homeowner's Guide to Overcoming Problems with Marine Clay in Fairfax County," Fairfax, Virginia, http://www.co.fairfax.va.us/dpwes.

Duncan, M.J. (1993). "Limitations of Conventional Analysis of Consolidation Settlement." *J. Geotech. Eng*., 119 (9), 1333-1359.

Earth Dimensions, Inc. (1981). *Soils Report: Step I Waste Water Facilities Report Southeast Planning Area Town of Amherst*. Consultant report to Town of Amherst.

ECSWCD, Erie County Soil and Water Conservation District (1972). *Soil Inventory and Interpretive Study for Town of Amherst.*

Feld, J. (1965). "Tolerance of Structures to Settlement: *J. Soil Mech. And Found. Div*., 91 (SM3), 555-569.

FHA, Federal Highway Administration (1975). *A Review of Engineering Experiences With Expansive Soils in Highway Subgrades, Interim Report No. FHWA-RD-75-48.* U.S. Army Engineer Waterways Experiment Station.

Freeman, T.J., Littlejohn, G.S. and Driscoll, R.M.C. (1994). *Has your house got Cracks?; A guide to subsidence and heave of buildings on clay,* Thomas Telford Services Ltd, London, England.

Ghably, G.Y. (1998). "Investigation of Foundation Failure Due to the Variations in Soil Moisture in South Texas." Unpublished Masters thesis, Texas A&M University, Kingsville, TX.

Giese, R.F. and Juul, K. (2005). "Report to USACE: Mineralogy of Amherst Drill Core Samples." University at Buffalo, Department of Geology, Amherst, N.Y.

GIT, Georgia Institute of Technology (2002). *GT STRUDL Release Guide Version 26,* Georgia Tech Research Corporation, Atlanta, GA.

Jalla, R. (1999). "Basement Wall Design: Geotechnical Aspects." *J. of Architectural Engineering,* 5(3), 89-91.

Jones, D.E. Jr., and Holtz, W.G. (1973). "Expansive soils – the hidden disaster." *Civ. Engrg*., ASCE, 43(8), 49-51.

Kappel, W.M. and Miller, T.S. (1996). *Geology, Hydrology, and Ground-Water Flow near the Akron Municipal Well, Erie County, New York.* U.S. Geological Survey Water-Resources Investigations Report 96-4193.

Kyfor, Z.G., and Gemme, R.L. (1994). "Compacted Clay Embankment Failures.: *Transportation Research Record 1462, National Academy of Sciences*, 17-27.

La Sala, A.M. (1968). Ground-Water Resources of the Erie-Niagara Basin, New York, New York State Conservation Department, Water Resources Commission, Basin Planning Report ENB-3.

MacClennan, P. (25 July 1974). "HUD Maps Show Amherst Projects Are Located in 'Flood Hazard Areas'". *The Buffalo News.*

McGuffey, V., Iori, J., Kyfor, Z., and Athanasiou-Grivas, D. (1981). "Statistical Geotechnical Properties of Lockport Clays." *Transportation Research Record 809, National Academy of Sciences*, 54-60.

Meehan, R.L., and Karp, L.B. (1994). "California Housing Damage Related to Expansive Soils." *J. Perf. Constr. Facil*., 8(2), 139-157.

Miller, T.S., and Staubitz, W.W. (1985). Hydrogeologic Appraisal of Five Selected Aquifers in Erie County; U.S. Geological Survey Water-Resources Investigations Report 84-4334.

Moore, W.W., and Chryssafopoulos, N. (1972). "Sample Approaches to Optimum Use of Marginal Lands." *J. Soil Mech. And Found. Div*., March 1972, 243-263.

Murray, R.S., and Quirk, J.P. (1980). "Clay-Water Interactions and the Mechanism of Soil Swelling." *Colloids and Surfaces* , (1), 17-31.

Nelson, J.D., Overton, D.D. and Durkee, D.B. (2001). "Depth of Wetting and the Active Zone" in Vipulanandan, C., Addison, M.B. and Hasen, M., *Expansive Clay Soils and Vegetative Influence on Shallow Foundations*, The Geo-Institute of the American Society of Civil Engineers, Houston, TX.

NJDEP, New Jersey Department of Environmental Protection (1999). *Planning for Clean Water: The Municipal Guide*.

Noe, D.C. Jochim, C.L., and Rogers, W.P. (1997). *A Guide to Swelling Soils for Colorado Homebuyers and Homeowners*, Colorado Geological Survey Department of Natural Resources.

NRCC, National Regional Climate Center (2005). "New York Drought Periods". http://www.nrcc.cornell.edu.

NWS, National Weather Service (2005). "Buffalo Monthly Precipitation". http://www.erh.noaa.gov.

NYSDOS, New York State Department of State Division of Code Enforcement and Administration (2003), *New York State Residential Code*, NYS Department of State and International Code Council.

NYSGA, New York State Geological Association (1966). "Geology of Western New York", Guidebook for NYSGA 38th Annual Meeting, Department of Geological Sciences State University of New York at Buffalo.

NYSGA, New York State Geological Association (1982). "Geology of the Northern Appalachian Basin Western New York", Guidebook for NYSGA 54th Annual Meeting, Amherst, NY.

PMP, Project Management Plan (2004). *Town of Amherst Soil and Structural Stability Study Project Management Plan*, U.S. Army Corps of Engineers, Buffalo District.

Poulos, H.G., Carter, J.P. and Small, J.C. (2002), "Foundations and Retaining Structures - Research and Practice", in Proceedings of the 15th International Conference on Soil Mechanics and Geotechnical Engineering, volume 4: pp. 2527-2606. Published by A.A. Balkema Publishers: Lisse, Netherlands. Held in Istanbul, Turkey, August 27-31, 2001.

Preene, M. (2000). "Assessment of Settlements Caused by Groundwater Control." *Proc. Instn. Civ. Engrs. Geotech. Engng. 2000*, 143, Oct., 177-190.

Senapathy, H., Davie, J.R. and Boehm, D. (2000). "Improving Deep-Seated Soft Clays Using Super-Jet Grouting." *Grouting and Ground Treatment Proceedings of the Third International Conference American Society of Civil Engineers*, New Orleans, LA., 440-451.

Silvestri, V. and Tabib, C. (1994). "Settlement of Building Foundation On Clay Soil Caused by Evapotranspiration," *Proc., Settlement '94*, ASCE, College Station, Texas, 1494-1503.

Simons, K.B. (1991). "Limitations of Residential Structures on Expansive Soils." *J. Perf. Constr. Facil.*, 5(4), 258-270.

Sridharan, A. and Prakash, K. (2000). "Classification Procedures for Expansive Soils." *Proc. Instn Civ Engrs Geotech. Engng*, Oct., 2000, 143, 235-240.

Staubitz, W.W., and Miller, T.S. (1987). Geology and Hydrology of the Onondaga Aquifer in Eastern Erie County, New York, with emphasis on ground-water-level declines since 1982: U.S. Geological Survey Water Resources Investigations Report 86-4317.

Steinberg, M.L. (1999). "Swelling Soils and Geomembrane Barriers." *Proc., Texas Section American Society of Civil Engineers*, Midland TX

Sultan, M. and Becker, R. (2005). Report to USACE: "Radar Interferometry Task: Amherst, NY." State University of New York at Buffalo, Amherst, N.Y.

Texas ASCE, Texas Section – American Society of Civil Engineers (2002). “Recommended Practice for the Design of Residential Foundations.”

Texas ASCE, Texas Section – American Society of Civil Engineers (2002a). “Guidelines for the Evaluation and Repair of Residential Foundations.”

Ulrich, E.J. (1994). “Mat Foundation Design: an Historical Perspective.” *Proc., Settlement ’94*, ASCE, College Station, Texas, 107-120.

USACE, U.S. Army Corps of Engineers (1973). *Review of Reports on Lake Erie – Lake Ontario Waterway*. Buffalo, New York.

USACE, U.S. Army Corps of Engineers (1979). *Ellicott Creek, New York General Design Memorandum,* Buffalo, New York.

USACE, U.S. Army Corps of Engineers (1992). “Bearing Capacity of Soils.” *Engineering Manual 1110-1-1905*

USDA, United States Department of Agriculture (1986). *Soil Survey of Erie County, New York.*

USDOA, U.S. Department of the Army (1983). “Foundations in Expansive Soils.” *Technical Manual 5-818-7.*

Wahls, H.E. (1981), "Tolerable Settlement of Buildings." *Proc., ASCE, Vol. 107, No. GT11, Nov. 1981*, 1489-1505,

Waltham, T. (2002). “Sinking Cities.” *Geology Today*, 18(3), 95.

Ward, J.S. & Associates (1971). “Preliminary Report of Soils and Foundation Investigation Expansion District 16 Sewage Treatment Plant, Town of Amherst, New York.

Ward, J.S. & Associates (1973). “Report of Soils and Foundation Investigation District 16 Sewage Treatment Plant Expansion, Town of Amherst, New York.

Whitlock, A.R., and Moosa, S.S. (1996). “Foundation Design Considerations for Construction on Marshlands.” *J. Perf. Constr. Facil.*, 10(1), 15-22.

Willenbrock, J.H., Manbeck, H.B., and Suchar, M.G. (1998). *Residential Building Design and Construction*, Prentice-Hall, Inc., New Jersey.

SECTION 8 – GLOSSARY OF TERMS

(The following definitions are introductory and not for technical citation)

Active Zone – is that zone of soil that is contributing to heave/settlement due to soil expansion/shrinkage at any particular time. The active zone will normally vary with time.

Bearing Capacity – the maximum foundation load that can be applied to a soil.

Continuous Footing – a footing that supports load continuously throughout its whole length, and can in some instances, support three or more columns provided the footer is connected to all columns. For the purposes of this study, all perimeter foundation footings are continuous footings.

Differential Settlement – (or differential foundation movement) a measure of the distortion in a wall based on the vertical displacement of one point with respect to another.

Expansive Soils – a family of soils found in various parts of the country that contain a large portion of highly plastic or moisture sensitive clays. Because these soils are sensitive to water their volumes increase, or swell, or decrease, or shrink, with changes in their moisture content.

Failure – is the unacceptable difference between expected and observed performance, which includes serviceability problems such as distress, excessive deformations, and collapse.

Footer – slang for footing.

Footing – also known as a spread footing, a structure foundation type designed to distribute or spread building loads over a sufficient area of soil to secure adequate bearing capacity.

Foundation Footing – see continuous footing.

Geotechnical Engineer – a registered professional engineer that specializes in the relationship between structure and the earth.

Groundwater – the water under the surface of the ground.

Hand Auger – a boring tool used to excavate about a 4” diameter hole in the ground.

Illite – one of three common clay minerals.

Infiltration – the penetration of water into the surface of the soil, rock, etc.

Interior Footing – see isolated or independent footing.

Isolated or Independent Footing – also known as a column footing, a footing that supports a single column, pier, post or other single concentrated load. For the purposes of this study, all interior footings are column footings.

Lateral Wall Pressure – horizontal force against a basement wall caused by (1) pressure from soil weight, (2) pressure from soil swell, (3) hydrostatic pressure, and (4) pressure from frost.

Liquid Limit – a measure of the minimum moisture content at which a clay looses its “plastic” properties and begins to flow.

Natural Moisture Content – is moisture content of undisturbed sample of soil, or the water content equaling the ratio of the mass of water to dry mass of solids expressed as a percentage.

Permeability – a measure of the rate at which water will flow through a soil.

Pier – a vertical support that provides bearing in the ground.

Piezometer – a hollow pipe inserted into the overburden to measure the groundwater head at that depth.

Pilaster – a projection from the face of the wall that extends the wall’s full height to provide lateral support.

Plastic Limit – a measure of the minimum moisture content at which a clay retains its “plastic” properties and does not break up when moulded.

Plasticity Index – the difference in moisture content between the plastic limit and the liquid limit for a given sample of clay.

Project Delivery Team - members of the Corps, Town, media, citizens and elected official that regulary participated in the review of this study.

Rebar – a steel reinforcing rod with a raised deformations on the surface that interlock with the surrounding concrete.

Subsidence – is the downward movement of the ground (beneath a building) independent of the building load.

Swelling Clay – a clay whose soil volume increase when ambient humidity or water content is increased.

Team – The Corps field and house inspection team that generally consisted of a geotechnical engineer, structural engineer, and a hydrologist.

Wall footing – a footing, which supports a wall by extending along the entire length of the wall.

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SECTION 9 – ABBREVIATIONS

AMSL - above mean sea level

BFE – Base Flood Elevation

CORPS – U.S. Army Corps of Engineers

CMU – cement masonry unit, commonly referred to as cinder block

DEM – Digital elevation model

FIRMs – Flood Insurance Rate Map

NFIP – National Flood Insurance Program

NRCS – Natural Resource Conservation Service

PDT – (Team)

SFHA – Special Flood Hazard Area

TOA – Town of Amherst

UB – State University of New York at Buffalo

USDA – U.S. Department of Agriculture

USGS – U.S. Geologic Survey