Ground Penetrating Radar Characterization of Wood Piles and the Water Table in Back Bay, Boston

by
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B.A., Geology and Environmental Studies
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Submitted to the Department of Earth, Atmospheric, and Planetary Sciences
in Partial Fulfillment of the Requirements for the Degree of

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ABSTRACT

Ground penetrating radar (GPR) surveys are performed to determine the depth to the water table and the tops of wood piles beneath a residential structure at 122 Beacon Street in Back Bay, Boston. The area of Boston known as the Back Bay was once a tidal estuary of the Charles River. During the latter half of the 19th century, the Back Bay was filled to create room for the city's expanding population. Most of the structures built in the Back Bay during this period were residential buildings supported by untreated wood pile foundations.

Submerged beneath the water table, untreated wood piles maintain their structural integrity indefinitely. However, recent groundwater fluctuations throughout the Back Bay have exposed the tops of some of the piles, causing the exposed areas to rot. Rotted wood piles weaken a structure's foundation and often result in differential settlement or cracking in walls or foundations. The current method of investigating suspected pile failure is to excavate a foundation and to physically inspect the piles, noting the elevation of the water table. In many cases, foundations may be stabilized by underpinning: replacing rotted wood piles with steel beams or concrete plugs often at great cost to the owner of the building.

The research presented in this thesis investigates the usefulness of GPR in determining the proximity of the tops of wood piles relative to the water table. Two different types of radar surveys were used in an attempt to estimate the depth to the water table and the tops of the piles. Data collected from several radar surveys is interpreted and compared with ground truth derived from historical references, water level data from monitoring wells, observations

from recent excavations, and the results of a resistivity survey. The results of this study indicate that modifications of this technique may allow more definite interpretation of wood pile foundations than traditional GPR surveys can provide in this type of environment.

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TABLE OF CONTENTS

TABLE OF CONTENTS 7

LIST OF TABLES AND FIGURES 9

1. INTRODUCTION 13

1.1 DEFINING THE PROBLEM 14
1.1.1 Evolution of the City of Boston 14
1.1.2 19th Century Building Practices 19
1.1.3 The Discovery of Groundwater-Related Structural Problems 21
1.1.4 The Role of Groundwater in Pile Deterioration 23
1.1.4.1 Groundwater Studies in Boston 24
1.1.4.2 An Aging Subterranean Infrastructure 26
1.1.5 Current Methods of Investigation 29
1.1.6 A Proposed Alternative: Ground Penetrating Radar 30
1.1.7 Similar Applications and Previous Research 31

2. GROUND PENETRATING RADAR: DEFINITION AND RESEARCH DESIGN 47

2.1 DEFINITION: WHAT IS GROUND PENETRATING RADAR? 47
2.1.1 Principles of Operation 47
2.1.2 The Propagation of Radiowaves 48
2.1.3 Signal Strength and Resolution 50
2.1.4 Data Acquisition 51
2.2 SURVEY DESIGN 52
2.2.1 What is the target depth? 53
2.2.2 What is the target geometry? 54
2.2.3 What are the target electrical properties? 54
2.2.4 What is the host material? 55
2.2.5 What is the survey environment like? 55

3. GROUND TRUTH 65

3.1 BOREHOLE DATA 65
3.2 WELL DATA 66
3.3 TEST PITS 66
3.4 RESISTIVITY SURVEY 66
3.5 HYDROLOGY AND INFRASTRUCTURE 67

4. DATA PROCESSING..... 73

4.1 REAL TIME PROCESSING..... 73
4.2 POST SURVEY PROCESSING..... 74
4.2.1 Gain..... 74
4.2.2 Enhancing Targets: Spatial and Temporal Filtering..... 76
4.2.3 Damping Noise: Median Filtering and Trace Editing..... 76
4.2.4 Normal Moveout Velocity Estimation..... 76

5. RESULTS..... 79

6. ANALYSIS AND CONCLUSIONS..... 91

7. RECOMMENDATIONS FOR FUTURE RESEARCH..... 101

REFERENCES..... 97

APPENDIX I – AN ILLUSTRATIVE CHRONOLOGY OF FILLING IN THE BACK BAY..... 103

APPENDIX II – STRATIGRAPHY OF THE BACK BAY..... 115

APPENDIX III – AQUIFERS OF THE BACK BAY..... 119

APPENDIX IV – MAXWELL'S EQUATIONS..... 121

List of Tables and Figures

Chapter 1

Figure 1.1a	Map of Boston in 1630	32
Figure 1.1b	The Colonial Shoreline Superimposed on a Modern Map	33
Figure 1.2a	The Trimountain	34
Figure 1.2b	Pemberton Hill, Beacon Hill, and Mt. Vernon	34
Figure 1.2c	The Trimountain	34
Figure 1.3a	A Map of Boston and the Surrounding Area circa 1800	35
Figure 1.3b	The Boston Neck	35
Figure 1.4	Early Filling in Boston	36
Figure 1.5	A Hole in Beacon Hill	37
Figure 1.6a	The Mill Dam and the Cross Dam	38
Figure 1.6b	Cross Section of the Mill Dam	38
Figure 1.7	Railroads in the Back Bay	39
Figure 1.8	A Primitive Steam Shovel	39
Figure 1.9a	Typical Back Bay Wood Pile Foundations	40
Figure 1.9b	Transverse Section of a Typical Back Bay Structure	41
Figure 1.9c	A Typical Pile Schedule	42
Figure 1.9d	A Typical Contract Drawing	42
Figure 1.10	Rotted Wood Piles	43
Figure 1.11	The Boston Main Drainage System	43
Figure 1.12	The Back Street Sea Wall	44
Figure 1.13a	Cross Section of the Boston Marginal Conduit	45
Figure 1.13b	Location of the Boston Marginal Conduit	45
Figure 1.14a	Cross Section of the West Side Interceptor	45
Figure 1.14b	Location of the West Side Interceptor	45

Chapter 2

Figure 2.1	A Typical Radar System	57
Figure 2.2	Common Offset Reflection Profiling	58
Figure 2.3	Common Midpoint Velocity Sounding	58
Figure 2.4a	Map of Monitoring Wells	59
Figure 2.4b	Water Level Data: Winter 2002 - Spring2003	59
Figure 2.5a	Soil Conditions	60
Figure 2.5b	Map Showing the Location of Borings	61
Figure 2.6a	Diagram of the Survey Site	62
Figure 2.6b	Survey Line 1	63
Figure 2.6c	Survey Line 2	64

Chapter 3

Figure 3.1a	Resistivity Survey Site	68

Figure 3.1b Diagram of the Resistivity Survey Site 69
Figure 3.2a Resistivity Survey Results (Logarithmic Plot) 69
Figure 3.2b Resistivity Survey Results (Linear Plot) 70
Figure 3.2c Analysis of Resistivity Survey 70
Figure 3.3 The Subterranean Infrastructure 71

Chapter 4

Figure 4.1 Normal Moveout Geometry 78

Chapter 5

Table 5.1 Survey Information 79
Figure 5.1 Diagram of the GPR Survey Site 82
Figure 5.2 Large Data Shifts 83
Figure 5.3 Removing Noise from the Data 84
Figure 5.4a Plot for Calculating NMO Velocity 85
Figure 5.4b Color Rendering of 5.4a 85
Figure 5.5 Effective Noise Reduction through Filtering 86
Figure 5.6 The 225 MHz Antenna Reveals Deeper Reflectors 87
Figure 5.7 Noise Reduction by Median Filtering 88
Figure 5.8 Common Midpoint Data, 225 MHz, Survey Line 2 89
Figure 5.9 Common Midpoint Data, 225 MHz, Survey Line 1 90

Chapter 6

Table 6.1 Summary of Groundwater Level Data 91
Figure 6.1 A Strong Reflector in BCN1_3 97
Figure 6.2 Reflectors at 25ns in BCN2_3 98
Figure 6.3 Strongest Reflector in BCN2_3 99
Figure 6.4 Events (Anomalies) in the Wiggle Plot 100

Appendix I

I.1 Map of Back Bay – 1814 107
I.2 Map of Back Bay – 1836 107
I.3 Map of Back Bay – 1851 108
I.4 Westerly View of the Back Bay – Late 1850s 108
I.5 Easterly View of the Back Bay – Late 1850s 109
I.6 Southerly View of the Back Bay – Late 1850s 109
I.7 Map of Back Bay – 1861 110
I.8 Map of Back Bay – 1871 110
I.9 Map of Back Bay – 1882 111
I.10 Map of Back Bay – 1888 111
I.11 Westerly View of the Back Bay – 1900 112
I.12 Map of Boston – 1902 112

I.13 The Progression of Filling 113
I.14 Current View of the Back Bay 113

Appendix II

II.1 Stratigraphic Column 117

Appendix III

III.1 Three Back Bay Aquifers 119

Appendix IV

CHAPTER 1: INTRODUCTION

An alternative title for this thesis might be: "Why Buildings Fall Down: Trying to See Through Dirt." The research presented investigates one small piece of a very large, very pervasive problem: the failure of untreated wood piles due to local groundwater fluctuations. It is a problem of not knowing, of not being able to see what is happening underground. The current solution is to move away the dirt and the fill, to clear everything out so that we may look upon the piles and the water table with our own eyes and make our own judgments. Yet this is often cumbersome, time consuming, invasive, and expensive.

In a society of quick answers and real time data streams, we find a possible solution in sensing technologies. Ground penetrating radar (GPR) is one such technology. GPR is widely used in geotechnical investigations to locate buried utilities, subterranean cavities, archaeological relics, forensic evidence, as well as for other applications. The objective of this research is to test whether GPR is a useful tool for identifying specific subsurface characteristics in Boston's Back Bay.

Chapter one provides an introduction to the problem, explaining how much of the city of Boston came to be built on fill and how the characteristics of the filled land affected the construction of buildings and the subterranean urban infrastructure. It also describes the role of groundwater in foundation stability by tracing the history of groundwater evolution and pile failure in the Back Bay. Chapter two explains how GPR works and how best to design a survey to maximize the chances of success.

Chapter three describes the ground truth: soil conditions gathered from boring data, water levels measured in monitoring wells, pile heights observed in recent excavations, and a review of subsurface features which might impact the local hydrology. Chapter four

describes the methods by which interpretation and meaning can be read from a raw radargram, and chapter five presents the results of the field work undertaken to support this effort. Chapter six provides an analysis of the information presented in chapters one through five and the conclusions drawn from this research. Chapter seven presents suggestions for future research. Appendices offer additional explanation and provide collateral information.

1.1 Defining the Problem

When the city of Boston was first settled in the early 1600s, the Back Bay was little more than a tidal estuary of the Charles River (figures 1.1a, 1.1b). As the population grew steadily, it became necessary to expand the city's land area by filling the vast expanses of surrounding marshes and coves. Although the filling of the Back Bay was a relatively well-planned operation, inconsistencies in methods of construction and the variable content of the fill make it nearly impossible to accurately predict contemporary subsurface conditions. The situation is worsened by a nearly complete lack of building records and conflicting data. Diagnosing the deterioration of wood piles is a complicated issue.

1.1.1 Evolution of the City of Boston.

To the casual observer, the city of Boston is a classic colonial seaport. The oldest city in the United States, credit for her founding is given to John Winthrop, who sailed to her shores in 1630 seeking freedom from religious persecution and desiring to found a "city upon a hill." Winthrop and his followers arrived at Boston, then the Shawmut Peninsula, to find that their "city upon a hill" was a city upon three hills. The Shawmut Peninsula, in the 1600s, was dominated by the Trimountain (or, Tremont), a collection of three hills joined together

along a ridge connecting Mt. Vernon to the west, Beacon Hill in the center, and Pemberton Hill (aka Cotton Hill) to the east (figures 1.2a, 1.2b, 1.2c). These were flanked by two smaller hills, Copp's Hill to the north and Fort Hill to the south. Today, only one of the five hills remains. Even the shoreline mapped by Captain John Smith in 1614 no longer exists, owing not to the dynamical forces of nature, of time and tide, but rather to the effects of man.

Woodhouse (1989) describes the major geological influences that affected the founding, and later, the growth of the city of Boston. A seafaring people, Winthrop and the early settlers sought a port with a safe harbor. The city and its immediate suburbs lie in a topographic depression known as the Boston Basin. Rocks of the Boston Basin are softer than then granites that outcrop along its periphery. Pleistocene glaciers easily carved a large indentation in the Massachusetts coastline, and rising seas flooded the low areas, forming Boston harbor and creating dozens of small islands.

Until the early 1800s, Boston was essentially a high tide island with an area just slightly greater than one square mile (783 acres) (figure 1.3a). The only connection to the mainland was the Neck, a low, narrow causeway connecting Boston to Roxbury and points south (figure 1.3b). Water provided protection from enemy invaders, and the city's hills and knolls served as natural fortifications. Most important to the early settlers, however, was the availability of fresh groundwater. Settlements in Charlestown and other low-lying areas had limited freshwater supplies that were usually brackish at best. Settlers in Boston, however, found several springs in the city's hills, allowing them to install artesian wells that provided a seemingly limitless source of fresh water. Nearly 400 years later, groundwater again plays a critical role in the city's survival, though for a very different reason.

The settlement of Massachusetts Bay, and of Boston in particular, increased steadily during the 19th century. According to the first nationwide census, the city's population numbered about 18,320 in 1790 (Whitehill, 1963). By 1800, the city's population had grown to 25,000. By 1825 it had doubled to 58,277, and by 1850 it was nearly five fold its 1800 level at 136,881 (Bunting, 1967). Thus the early Bostonians, long before the advent of mechanized equipment, began to modify their landscape to accommodate an expanding population. They looked to the city's hills as a proximal source of fill and to her marshes and coves as future streets and squares.

The filling of the city of Boston began as a rather haphazard operation (figure 1.4). An early tidal dam was constructed in 1643, enclosing the marshy North Cove at the mouth of the Charles River. By the early 1700s, much of the Inner Harbor had been developed, with Long Wharf as the center of the thriving seaport. Beacon Hill was mined for gravel beginning in 1790, at which time cows still grazed on the Boston Common. The Public Gardens were nothing but a gray mudflat until 1794 when filling began at the foot of the Boston Common. In 1795 a large hole was gouged in the side of Beacon Hill to create space for the new state house (figure 1.5). From 1799-1803, 15-18 meters (50-60 feet) of Mt. Vernon was excavated, and the cove at its base, West Cove, was filled to create Charles Street. North Cove was filled intermittently between 1805-1835 with material from Copp's Hill and the Beacon Hill. Throughout most of the 19th century, extensive filling was carried out in the South End, and fill slowly encroached on the Inner Harbor until it was declared off limits for further development. This study, however, is largely concerned with the filling that occurred in the Back Bay.

The first major construction in the Back Bay occurred between 1818-1821 with the building of the Mill Dam, an earth dike running essentially the length of Beacon Street from the foot of Charles Street to Sewall's Point in Brookline, near present day Kenmore Square (figure 1.6a). The dam consisted of two parallel rubble masonry walls, about 15 feet high and 50 feet apart, resting on timber supports bearing on organic soils. The walls were ballasted with small stones, and the remaining space between the walls was backfilled with mud, sand and road base (figure 1.6b). Relatively impervious to the flow of water across its width, it was likely to have been quite pervious along its length (Lambrechts et al, 1985). A second dam was built in 1821. Known as the Cross Dam (figure 1.6a), it divided the Back Bay into two basins, extending from Gravelly Point in Roxbury and intersecting the Mill Dam just east of where Massachusetts Avenue is today. A differential in water height was maintained between the two basins to power a series of mills located along the dam[1].

The second major encroachment on the Back Bay, after construction of the dams, was the railroads. Between 1831-1835 two railroad causeways were constructed on gravel-filled, pile-supported rail beds that crisscrossed the Back Bay (figure 1.7). The Boston & Worcester line (1834) traversed the Back Bay along the tracks now serviced by Conrail. In 1835 the Boston & Providence line opened with tracks crossing the Back Bay southwest-northeast from Roxbury to Park Square. The two lines intersected near where Back Bay station is located today. The railroads greatly interfered with the flow of water in the Back Bay basins, reducing the usefulness of the area for power generation and limiting the ability to flush the city's sewage out with the tide as it had done for so many years.

---

[1] Aldrich (1970) provides a description: "At high tide, water was admitted into the full basin, located in the Fens west of the cross dam, powering machinery in mills located along the cross dam on Gravelly Point, discharging into the easterly receiving basin, and at low tide water was sluiced back into the Charles river through the main dam near the present Exeter St."

Filling continued through the middle of the 19th century. The filling of the Public Gardens, begun in 1794, was finally completed by 1826. In 1835 the top of Pemberton Hill was shaved off to fill and develop land north of Causeway Street, the area today known as Pemberton Square. The remaining ridge connecting the former peaks of Beacon and Pemberton Hills was leveled in 1845, and by 1850 the city's land area had increased to 1131 acres. The most significant filling in the Back Bay had yet to occur, and the dams and railroads had transformed a once pleasant tidal basin with cool sea breezes into an offensive open sewer with a horrendous stench and detrimental health effects[2]. The city had little choice but to extend the filling into the Back Bay. (See Appendix I for an illustrative sequence of filling.)

Filling of the lower basin began in 1857. Having already flattened and developed the land where the city's hills once stood, sand and gravel fill was carted by rail from a site in Needham near where present day Route 128 intersects Needham Avenue. This massive undertaking involved laying a temporary rail line along what would later become Commonwealth Avenue. The project employed 145 rail cars, 80 men, and two very primitive steam shovels. Three 35-car trains ran continuously, with one arriving at the Back Bay every 45 minutes.

After depositing a load of fill and returning to the borrow pit, the train was split in half and each half was filled by one of the steam shovels (figure 1.8). It took ten minutes to load one car, as two shovel-fulls filled one car. Some of the sand hills in Needham were 50 feet

[2] Lambrechts et al (1985) provide a description of the problem: "Most house drains and sewers were below basement level, and when minimum slopes to street sewers and interceptors were provided, the outfalls were rarely above low tide. As a result, contents of the sewers were dammed up by the tide during the greater part of every day. (Tide gates were commonly adopted to prevent salt water from flooding lower reaches of the sewers.) Settlement of the filled land caused numerous breaks in sewer connections and reversals of slope. Soon there were deposits of sludge and debris within the sewers and upon the tidal lands, with attendant health and odor problems."

high, and it is estimated that in the first year alone about 12 acres were leveled to create enough fill for 14. Filling occurred at a rate of approximately 2500 cubic yards per day. The fill was usually placed to approximately El. 12, Boston City Base datum[3], although the streets were built up to El. 18 (Aldrich, 1970).

By the start of the Civil War in 1861, fill had been placed as far west as Clarendon Street. By 1870 the filling had advanced to Dartmouth Street, and during the next 15 years the land along the Charles River was transformed into a system of parks under the direction of the famed landscape architect, Frederic Law Olmstead. Filling in the Back Bay had advanced to the Fens by 1880, and by 1890 the filling reached its furthest extent at Sewall's Point in Brookline near present day Kenmore Square. A total of 450 acres had been added to the city's land area, bringing the total acreage to 1581. A few more minor areas were filled as late as the early 1950s. These included: the Esplanade (1893), a 100 foot wide promenade on the southern banks of the Charles River; the Charles River dam (1910) to stabilize water levels in the Charles River Basin; the Storrow drive embankment (1929-1931); and Storrow Drive (1951).

1.1.2 19th Century Building Practices

Construction in the Back Bay followed rapidly after the filling. Most of the early structures were 4-5 story residential buildings, although many of Boston's cultural institutions, including the original Museum of Fine Arts, the original Massachusetts Institute of Technology Building, Symphony Hall, the Public Library and Trinity Church at Copley

[3] All elevations are referenced to the Boston City Base Datum, which is 5.65 feet below the National Geodetic Vertical Datum (formerly the U.S. Coast and Geodetic Survey Mean Sea Level Datum of 1929). Elevation 0.0 BCB datum is therefore 5.65 feet below mean sea level.

Square, and the Christian Science Mother Church, were also built during this time. Sewers and drains were laid beneath city streets, later followed by the subways.

The standard foundation practice was to support structures on untreated wood piles. Piles were typically 7.5-12 meters (25-40 feet) in length and approximately 6 inches in diameter (Johnson, 1989). Most frequently, the piles were spruce or oak trees stripped of their branches, inverted, and driven into the ground to a suitable bearing stratum beneath the fill, most commonly the sand and gravel outwash or the yellow clay layer (Lambrechts et al, 1985). (See also Appendix II.) Drop hammers were used to drive the piles. These commonly had weights ranging from 1800-2300 pounds and were dropped from heights of 10-25 feet (Aldrich, 1970). Piles were spaced 2-3 feet on centers and capped with dry-laid blocks of rock, frequently granite, upon which brick or stone foundations were built (figures 1.9a-d). Safe loads were estimated to be approximately ten tons (Lambrechts et al, 1985).

Wood piles are the oldest type of pile foundation, and under favorable environmental conditions they maintain their structural integrity over long periods of time. Submerged beneath the water table, untreated piles remain structurally stable and intact. Deterioration, however, can be caused by any combination of factors, including fungi, insect attack, or mechanical wear (Chellis, 1961). In the Back Bay, decay of untreated wood piles is caused by the growth of fungi which break down the cellular structure of the wood when the tops of the piles are exposed to air due to fluctuations in the local water table (figure 1.10).

Prior to 1900, it was common practice to cut off the tops of the wood piles at El. 5, which was roughly mean tide level and approximately three feet below the groundwater table (Lambrechts et al, 1985). By as late as 1915 and certainly even earlier, some civil engineers argued that piles could be cut off at El. 7 or 8 with certain safety, as a result of the fact that the

groundwater level in the Back Bay during the latter part of the 19th century was approximately El. 8 (Aldrich, 1970). However, there was significant discussion about what was an acceptable cutoff elevation. Recent inspections show that many wood piles throughout the Back Bay were indeed cut off higher than El. 5 (Lambrechts, 2003).

There are several structural failures attributed to problems with wood pile foundations, though the two that directly affect buildings in the Back Bay include settlement and cracking. Many heavy buildings in the Back Bay settled despite, or perhaps due to, a large number of closely spaced piles. An example is Trinity Church, which settled nearly one foot in the 30-40 years following its construction in 1876. Similarly, the towers of the Old South Church on Boylston Street suffered differential settlement due to consolidation of the Boston blue clay and subsequently had to be dismantled and reassembled (Lambrechts et al, 1985).

The most drastic pile failure in the Back Bay, however, was discovered in 1929 when cracks in the Boston Public Library building and settlement of the structure's stone platform facing Copley Square were observed. Investigations revealed that the tops of many wood piles supporting the library were badly rotted and in some cases completely decayed. The piles had originally been cut off at El. 5.0, and at the time of the investigation the groundwater level was observed to be a full foot lower at El. 4.0. Unsound piles beneath approximately 40 percent of the library had to be underpinned (Aldrich and Lambrechts, 1986).

1.1.3 The Discovery of Groundwater-Related Structural Problems

Prior to the discovery of rotted piles beneath the library, two other occurrences of rotted piles had been reported in Boston. In July, 1921, 4 Appleton Street in the South End required underpinning where piles cut off at El. 3.96 had rotted due to exposure above the

groundwater table, which had dropped to El. 3.3. In June, 1933, rotted piles at El. 8.13 were reported at 12 Hereford St., where the water table was found to be at El. 6.50 (Aldrich and Lambrechts, 1986).

Following the discovery of rotted piles beneath the Public Library, officials at Trinity Church, situated just across Copley Square only a few hundred yards away, became concerned that their foundation might also be affected. Excavations, however, revealed that the church's piles, cut off from El. 5.0-5.5, showed no signs of significant decay, most likely due to the fact that settlement of the structure had driven the tops of the piles deeper into the fill. Monitoring is ongoing at Trinity Church, as well as several other Back Bay institutions, including the Christian Science Church, Prudential Center, Public Library, Massachusetts Turnpike Extension, and Church of the Advent (Aldrich and Lambrechts, 1986).

The lower Beacon Hill area, from Charles Street to Embankment road, has been a problem area since the late 1920s. A report by the Boston Inspectional Services Department indicated that repairs to wood piles have been made at 38 of 188 residences and commercial buildings in this 10-block area, with some repairs being made every decade since the 1920s. This area received a great deal of attention during the early 1980s when residents along the waterside of Brimmer Street between Pinckney and Mt. Vernon Streets noticed cracks developing in interior and exterior walls as well as other evidence of differential settlement. The piles in this area had been cut off at El. 7.0, and in many cases the top 1-3 feet of the piles had severely decayed. Groundwater levels were measured several feet below the tops of the piles and as great as six feet below the mean water level in the nearby Charles River (Aldrich and Lambrechts, 1986).

There was a series of groundwater-related problems, and extensive publicity, in the 1970s and 1980s. Apartments on Hemenway Street in the Fenway were demolished in the 1970s due to rotting piles (Hawkins, 2000). In the spring of 1980, four residential buildings on Belvedere Street were demolished following the discovery of piles which had deteriorated beyond repair (Brown, 1980). Similar conditions were reported on Hudson Street in Chinatown, and a survey by the city Inspectional Services Department revealed that of 160 Chinatown buildings, 90 percent showed damage characteristic of rotting piles (Ranalli, 198*). A recent article in the Boston Globe reports that Chute Hall Construction, a company specializing in foundation repair, sees about two dozen damaged buildings each year where the underlying cause of the damage is rotted pilings (Cook, 2002).

1.1.4 The Role of Groundwater in Pile Deterioration

Neither the problem of wood pile deterioration nor the conditions under which it frequently occurs are unique to Back Bay, Boston. The following case studies document groundwater-related wood pile failure in other U.S. cities[4].

- New York, NY, 1910: A deep, spring-fed, freshwater pond in New York City once maintained local groundwater levels in the vicinity of Tombs prison, saturating wood piles supporting prison buildings and surrounding structures. Within two years after water was pumped and drains were installed for construction of a subway tunnel, a seven-story brick warehouse in the vicinity showed evidence of settlement. Excavations revealed that the tops of the piles supporting the building had begun to decay. The decayed piles were cut and the building's footings extended to maintain the structure.

- New York, NY, 1924: Wood piles exposed at the corner of Bethune and Washington Streets suffered severe decay due to a 14 foot drop in water table elevation. Piles exposed above the water table had the consistency of peat moss and could be pulled apart by hand.

[4] All case studies are taken from Chellis (1961).

- Brooklyn, NY: Extensive wood pile decay due to localized lowering of the groundwater table over a period of several years was discovered during the modernization of a large power station. The piles were removed and replaced. Also in Brooklyn, NY, lowered ground water levels due to pumping for industrial uses in Long Island caused buildings in the Navy Yard experienced settlement due to decay of untreated wood piles.

- Chicago, IL, 1932: A grain elevator on the banks of the Chicago River was observed tilting toward the river. Inspection revealed that the top six inches of the untreated piles supporting the building had rotted away. Repair required exposing the entire foundation top, cutting off the rotted sections of the piles, and underpinning.

- Lake Washington, Seattle, 1932: A timber trestle founded on untreated wood piles was refitted for a heavier deck. It was discovered that the piles were badly decayed at ground level but perfectly preserved 1-2 feet below the surface where the ground was permanently saturated. The tops of the piles were cut off and replaced with concrete piers.

1.1.4.1 Groundwater Studies in Boston

Concern over groundwater levels in the city has initiated several groundwater studies since the late 1800s. The earliest grew out of the concerns of city engineers commissioned to design what would become the Boston Main Drainage System. They feared that a system of intercepting sewers would eliminate the semi-daily tidal damming which occurred in the existing sewers. This could cause groundwater levels to drop below the tops of the piles, inducing decay and endangering the stability of many newly built structures.

In the late 1870s to early 1880s, an experiment simulating this condition was set up in the Back Bay by means of a steam pump in the Berkeley Street sewer. Continuous pumping ensured that the sewage level was kept to a minimum, as if discharging into an intercepting sewer. The groundwater level was measured in 20 wells, basically pipes, drilled below the water table for the experiment. Some were placed within a few feet to a few hundred feet of

the Berkeley Street sewer while others were placed several blocks away. Water height in the pipes was measured twice daily as the pumping continued.

Results of this experiment determined the water level to be 7.7 feet above the mean low water line, independent of the conditions imposed on the sewers but slightly affected by local topography. It was also observed that groundwater rose and fell uniformly throughout the Back Bay in quick response to any rain or melting snow. Most importantly, the pumping, over a period of 53 days, affected the groundwater levels within a 100-foot radius of the Berkeley Street Sewer only slightly. When the pumping ceased, the groundwater levels responded to their previous levels and began to fluctuate with the normal water levels observed in the rest of the Back Bay (Clark, 1885).

Groundwater studies were commissioned again in 1894 when proposals for construction of the Charles River Dam were being considered. Water levels, in general, were found to be similar to those measured nearly a decade earlier. Leaky sewers, however, were blamed for a few localized water table depressions. As a result of these studies, it was recommended that the new dam maintain the Charles River at El. 8.0 (Lambrechts et al, 1985).

Following the discovery of rotted piles beneath the Public Library in 1929, observation wells were installed in Copley Square and several local institutions took it upon themselves to monitor local groundwater levels. Another formal survey of the entire Back Bay was conducted from 1936-1940 by the Works Progress Administration (WPA). Approximately 700 wells throughout the Boston peninsula were monitored, of which approximately 300 were located in the Back Bay. The highest and lowest water levels measured during the four year monitoring period were recorded, and although complete

records of this data no longer exist, most wells in the Back Bay experienced water levels below El. 5.0. Precipitation in the city during this period was about average (Lambrechts et al, 1986).

Water levels were not measured again until the United States Geological Survey (USGS) conducted well readings during 1967-1968. Though sparse (groundwater levels were measured only twice, September 1967 and March 1968, respectively), the data appeared to indicate higher groundwater levels compared to the WPA data. However, some of the areas measured below El 5.0 from 1936-1940 remained as low in 1967-1968, despite precipitation levels six inches above average during 1967. In addition, five inches of rain fell over the two day period during which data were collected in March, 1968 (Lambrechts et al, 1986).

Monitoring of groundwater levels was sporadic from 1970 through the late 1990s. Of the observation wells monitored at some point during this period, data were rarely collected for periods greater than a year since, in most cases, monitoring was to track the effects of local construction projects (Lambrechts et al, 1986). During the 1980s, heightened publicity over groundwater-related foundation problems led to the creation of the Boston Groundwater Trust[5], an organization which began monitoring a network of wells throughout the Back Bay beginning in 1999. This most recent monitoring effort indicates that there are 'hot spots' throughout the Back Bay where groundwater levels are dangerously low, where dangerously low indicates water levels below the suspected pile cutoff of El. 5.

1.1.4.2 An Aging Subterranean Infrastructure

Many of the localized groundwater depressions in the Back Bay can be linked to dewatering for construction projects, leaky sewers, basements, and drains, and local barriers

[5] http://www.bostongroundwater.org

to flow. A summary of some of the more important projects is presented (Aldrich and Lambrechts, 1986).

Dewatering for construction:

- Construction of the Boston Main Drainage System from 1877-1884 involved connecting the existing sewers in the Back Bay to a system of intercepting sewers built along the margins of the city (figure 1.11). The West Side Interceptor was constructed along Beacon Street and connected to existing sewers at Beaver, Berkeley, Dartmouth, Fairfield and Hereford Streets, which formerly discharged into the Charles River. The invert grade at Beacon and Arlington Streets is El. 0.0, and at Beacon and Hereford Streets it is El. –2.4.

  Estimates suggest that excavating and dewatering for construction of the West Side Interceptor would have been 2 feet below these invert grades, thus potentially causing draw down in the fill and organic layers (upper aquifer) at Arlington/Beacon and in the outwash layer (middle aquifer – Appendix III) at Beacon/Hereford. During construction of the intercepting sewers, underdrain pipes (8-12 inch diameter) were used to control groundwater, but they were never removed. Within ten years of construction, groundwater levels in some areas of the Back Bay had dropped to El. 5 or lower, suggesting groundwater leakage into sewers.

- Temporary draw downs in both the fill and the outwash strata are also thought to have occurred during construction of the Boylston Street Subway (1912-1914) and the Huntington Avenue Subway (1937-1940). In particular, construction of the Huntington Avenue subway required the most extensive and prolonged dewatering of any Back Bay construction project before or since. Drains installed in the tunnels of both subways are also believed to be local groundwater sinks.

- In the last 65 years the construction of buildings with deep basements and the construction of underground parking garages has also caused temporary water table fluctuation due to dewatering. Built in the late 1930s, the earliest Back Bay buildings to have deep basements are the Liberty Mutual and New England Life buildings in the insurance district. Other deep-seated buildings in the Back Bay include the John Hancock Berkeley Building (1946), John Hancock Tower (1968), Prudential Center Tower (1960), and Copley Place (1981).

Leaks:

- Other sewers constructed and modified during this time period were undoubtedly leaky, and it is thought that groundwater levels throughout the Back Bay were widely

affected by this leakage. In particular, the St. James Avenue sewer is historically known as a source of local groundwater lowering.

- The construction of Storrow Drive included an underpass and interchange between Embankment Road and Berkeley Street. The road surface descends to approximately El. –4 at its lowest point. The underpass was designed to prevent groundwater lowering, but soon after completion groundwater began infiltrating through leaks in the concrete walls. These leaks were never plugged despite attempts to repair them, and in 1986 it was reported that about 20,000 gallons per day was being pumped into the Charles River from a series of wells in the vicinity of the underpass.

Local barriers to flow:

- Two of the Back Bay's first construction projects were the Mill Dam and its complement, the Cross Dam. After its completion in 1881 and until the major Back Bay filling was completed in 1880, water levels east of the Cross Dam were generally below mean tide[6]. The Mill Dam lies beneath present-day Beacon Street, and while it is thought to freely conduct water longitudinally, except where it has been breached by construction it is likely impervious along its width.

- As filling advanced north of Beacon Street, a sea wall was constructed along the Charles River running parallel to present day Back Street. The top of the wall remains exposed today along portions of Back Street (figure 1.12). The wall was constructed of dry-laid granite placed on a timber platform supported by wood piles, and it is likely that the walls were ballasted with stone or gravel similar to the walls of the Mill Dam.

- The Boston Main Drainage System was designed to handle dry-weather flow and a small volume of storm water. In times of heavy rain, excess water and sewage from the West Side Interceptor was discharged into the Charles River at a number of overflow outlets. This quickly became a nuisance for residents along the river, and as part of the 1910 Charles River Dam project, the Boston Marginal Conduit (figures 1.13a, 1.13b) was constructed along the Boston side of the Charles River basin (presently beneath Storrow drive) to collect overflows from the West Side Interceptor (figures 1.14a, 1.14b). Like the Mill Dam, the Boston Marginal Conduit and West Side Interceptor are relatively impervious along their widths but conduct groundwater easily along their long axes. Thus, all three impede the flow of water from the Charles River into the Back Bay.

[6] Mean tide in Boston Harbor is approximately El. 5.

In several instances, construction projects have been designed to minimize the effect of dewatering, though these efforts have not always been successful. The Massachusetts Turnpike Extension (1963-1966), a six-lane highway which crosses the Back Bay just north of the Conrail tracks, was designed to prevent a permanent lowering of groundwater levels below El. 6.5-El. 8.5, depending on location. Where underdrains were used, steel sheetpiling was driven into the clay layer to prevent the leakage of groundwater into the underdrains (Aldrich and Lambrechts, 1986).

The Southwest Corridor Project (1981-1985), the construction of three tracks servicing the Massachusetts Bay Transportation Authority (MBTA) Commuter Rail and Amtrak and two tracks for the relocated MBTA Orange Line, cuts through the Back Bay along parts of the two original railroad embankments in the old receiving basin. In some places, construction required excavations approaching depths of nearly 40 feet below ground surface (approximately El. –20). In order that the new tunnels not impede the flow of groundwater in the Back Bay, a groundwater equalization underdrain was installed (Aldrich and Lambrechts, 1986). However, not long after its construction was complete, the Southwest Corridor was observed to leak in several areas, prompting concern over local groundwater draw down in the vicinity of Holyoke Street in the South End (Cook, 2002).

1.1.5 Current Methods of Investigation

There exists in the Back Bay an interesting dichotomy of perception regarding wood piles and groundwater. Some residents could not care less about what goes on beneath their basement floors. Others, however, take proactive measures to ensure that their foundations are sound, regardless of whether or not there is physical evidence that proves otherwise. The

most direct means of determining the current state of a building's wood piles is to dig a test pit. Pits are 10-15 feet deep and usually extend to two feet below the tops of the piles. Excavating the foundation allows for two key observations: the elevation and physical condition of the tops of the piles, and the elevation of the local groundwater table.

An excavation of this sort costs between $1,200-$4,000 per building, depending mostly on accessibility constraints (Lambrechts, 2003). The urban environment of the Back Bay presents several challenges. Where possible, a backhoe may be used to dig the pit at costs near the lower end of the price range. However, more often it is the case that a backhoe cannot be used and the pit must be dug by hand at greater expense. In addition, a quick scan of Back Bay streets immediately reveals that all utilities are underground, further complicating excavations. Finally, the buildings are spaced so tightly that many have shared party walls. In certain cases the owners of the buildings may agree to split to cost of digging a test pit (Sherin, 2003). However, one can readily imagine circumstances in which cooperation may be difficult to achieve.

1.1.6 A Proposed Alternative: Ground Penetrating Radar

This thesis proposes that ground penetrating radar (GPR) may be a feasible alternative to excavation intended to provide information about wood pile and groundwater conditions. Although GPR does not, as employed in this study, reveal information about the physical condition of the wood piles, its usefulness is in determining the relative proximity of wood piles and the water table. In this sense, GPR could be used to determine whether a structure is at potential risk of pile failure.

GPR is a widely used method in near surface geophysics. It is based on the principle that an electromagnetic (EM) pulse will be reflected by dielectric contrasts, "reflectors" in the subsurface, and the depth to a reflector may then be inferred if the velocity of the EM pulse and the travel time of the reflected pulse are known. In contrast to digging, GPR is a noninvasive technique. Estimated costs for a commercial GPR survey range from $1,500-$2,000 (Richter, 2003; Bechtel, 2003). As with excavating, using GPR in an urban environment does present certain challenges. In many cases, however, these may be overcome by appropriate choice of equipment, survey design considerations, and data processing techniques.

1.1.7 Similar Applications and Previous Research

There are a variety of hydrological applications for GPR, and it is frequently used to determine depth to the water table (Nakashima et al., 2001; Pilon et al., 1994; Smith et. al, 1990). An exhaustive literature search, however, suggests that this is the first study which attempts to use GPR to locate wood pile foundations and their relative proximity to the water table. While one would not expect to "see" piles which remain safely submerged, rotted piles exposed above local water levels should provide sufficient dielectric contrast to the surrounding fill. The fill generally consists of sand and gravel which has a lower water content, and hence a lower dielectric capacity, than rotted wood, which, in its most advanced stages has the consistency of peat moss.

Figure 1.1a – A Map of Boston in 1630. The Back Bay is a tidal estuary.
(http://omega.cc.umb.edu/~conne/wendy/History.htm)

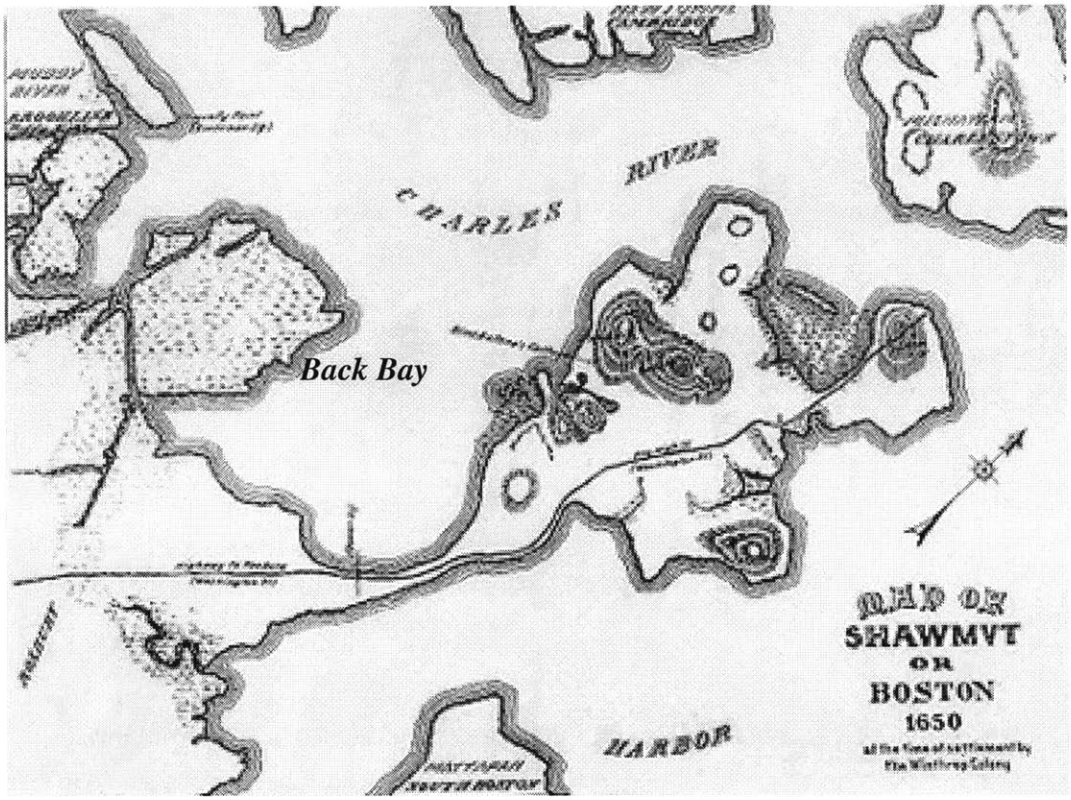


Figure 1.1b – The Colonial Shoreline Superimposed on a Modern Map
(Aldrich, 1970)

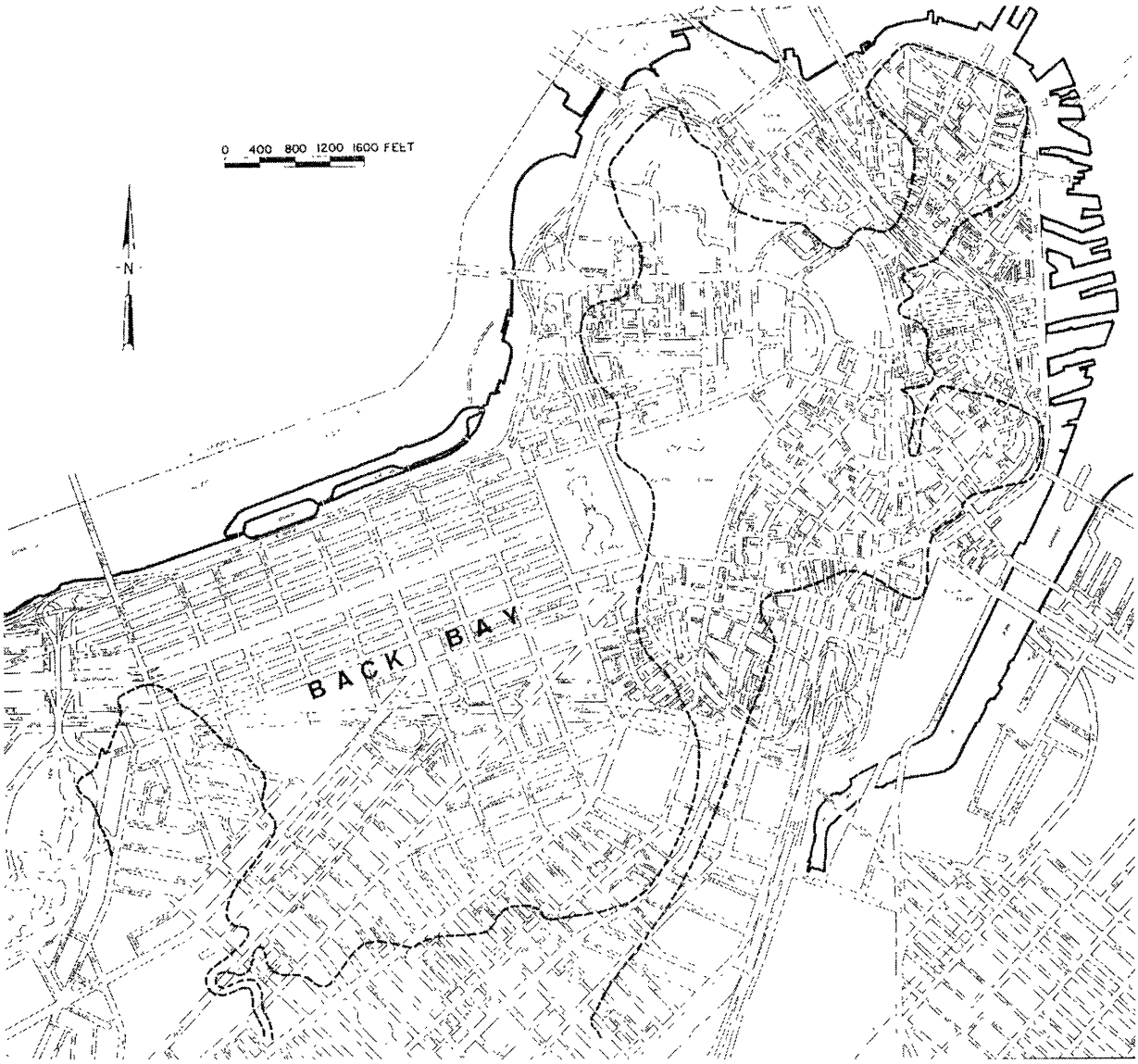
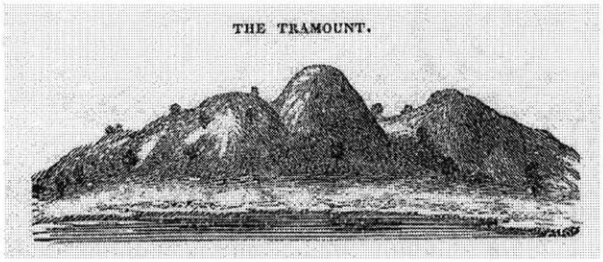
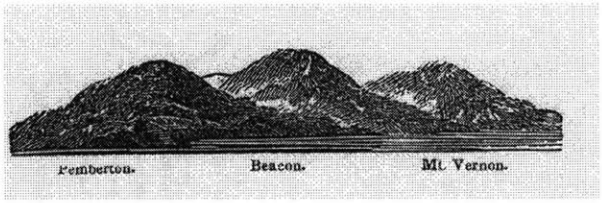


Figure 1.2 – The Trimountain
(Whitehill, 1963)

1.2a



1.2b



1.2c – The Trimountain, as seen from the Cambridge shore of the Charles River, 1776.

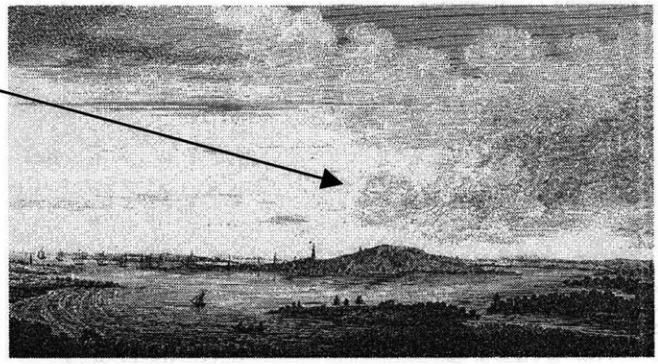
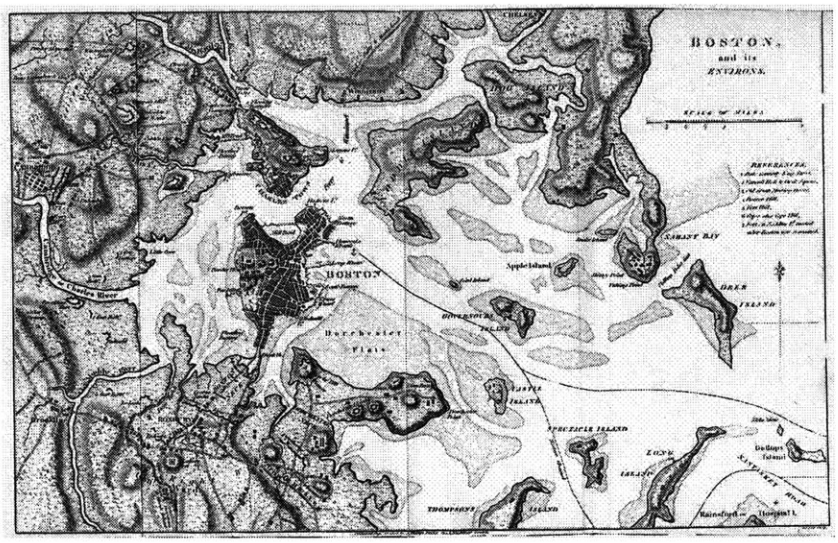


Figure 1.3 – In the late 1700s / early 1800s, Boston was essentially a high tide island, connected to the mainland by the Boston Neck

1.3a – Boston, a high tide island, circa 1800.
(http://earlyamerica.com/earlyamerica/maps/bostonmap/bostonmap.jpeg)



1.3b – The Boston Neck, 1775 (Kaye, 1976)

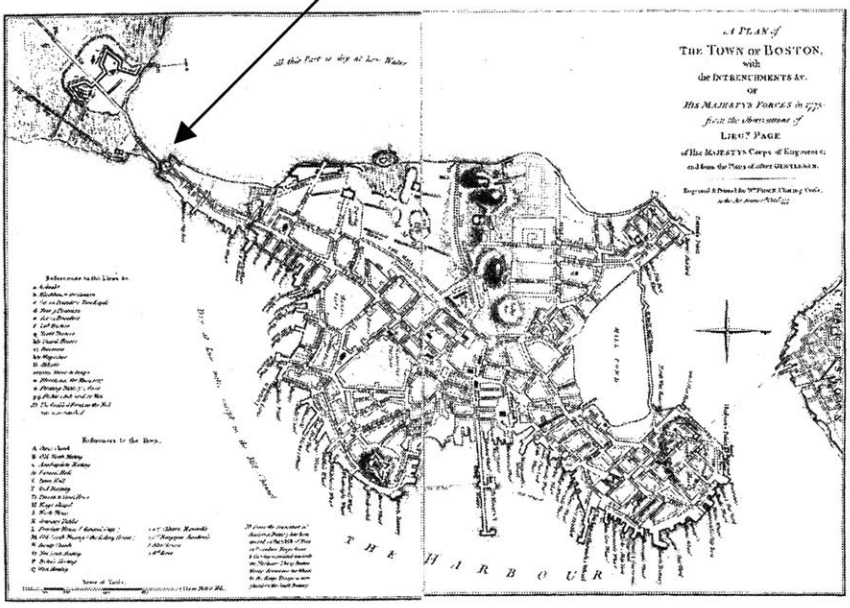


Figure 1.4 – Early filling in Boston (Adapted from Bunting, 1967)

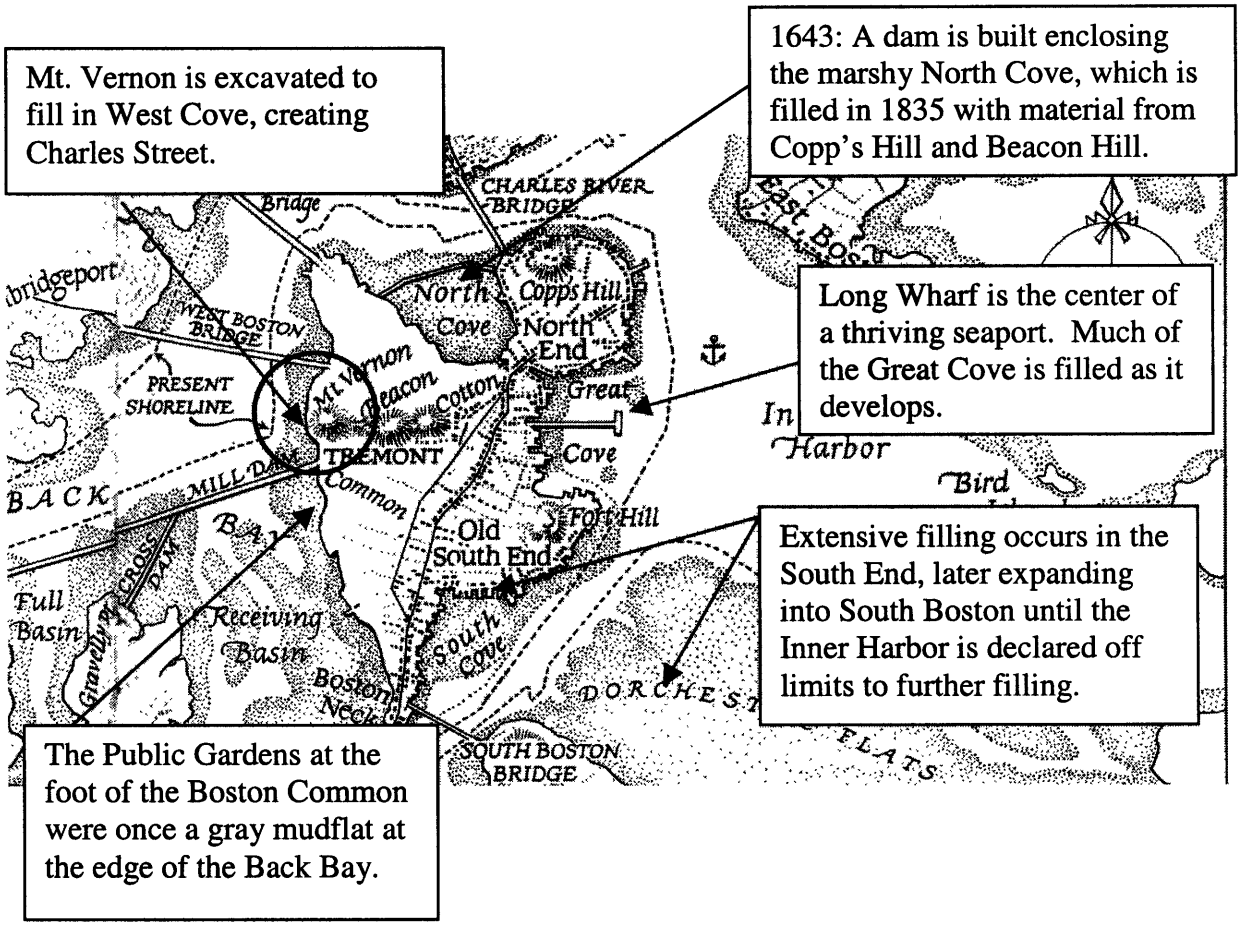


Figure 1.5 – A hole is gouged in Beacon Hill to make way for the new State House in 1755. (Bunting, 1967)

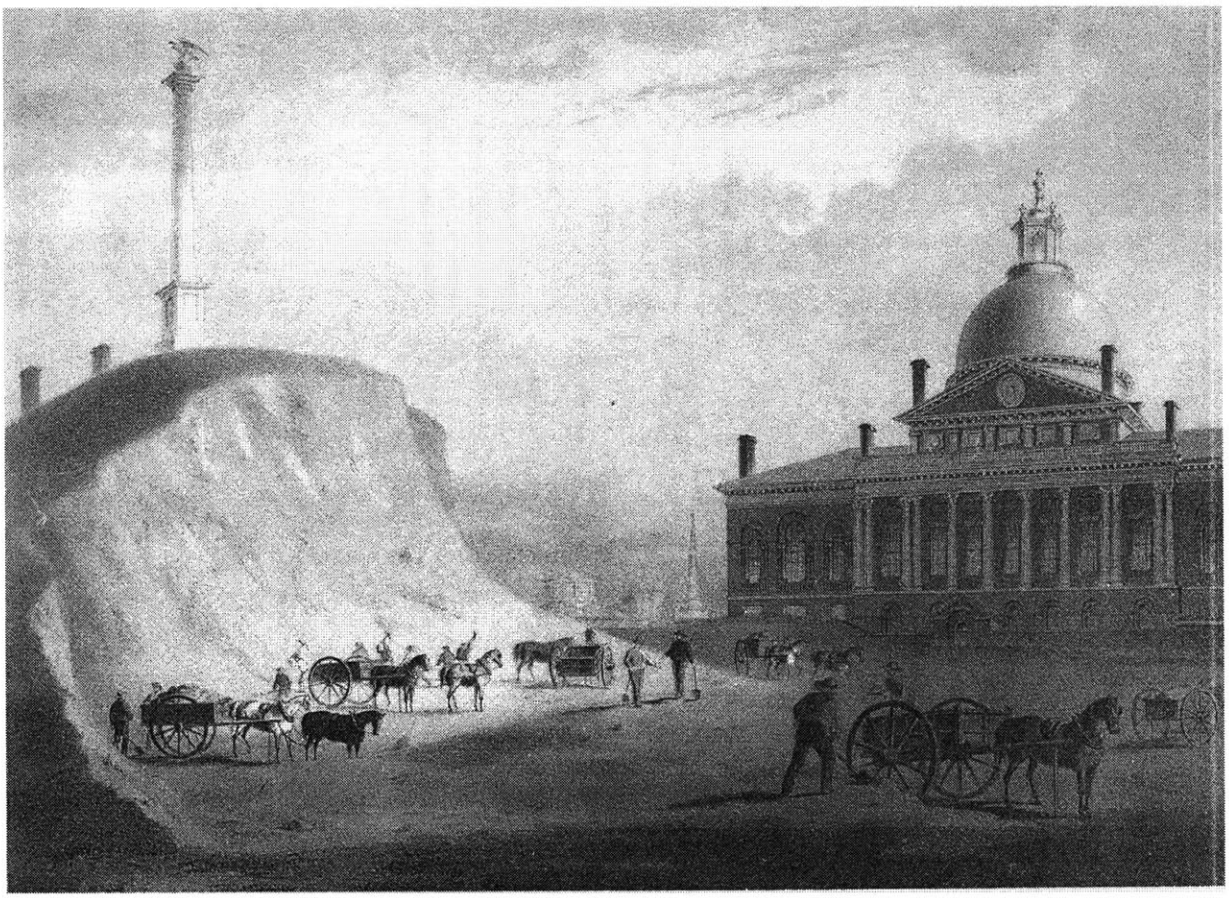
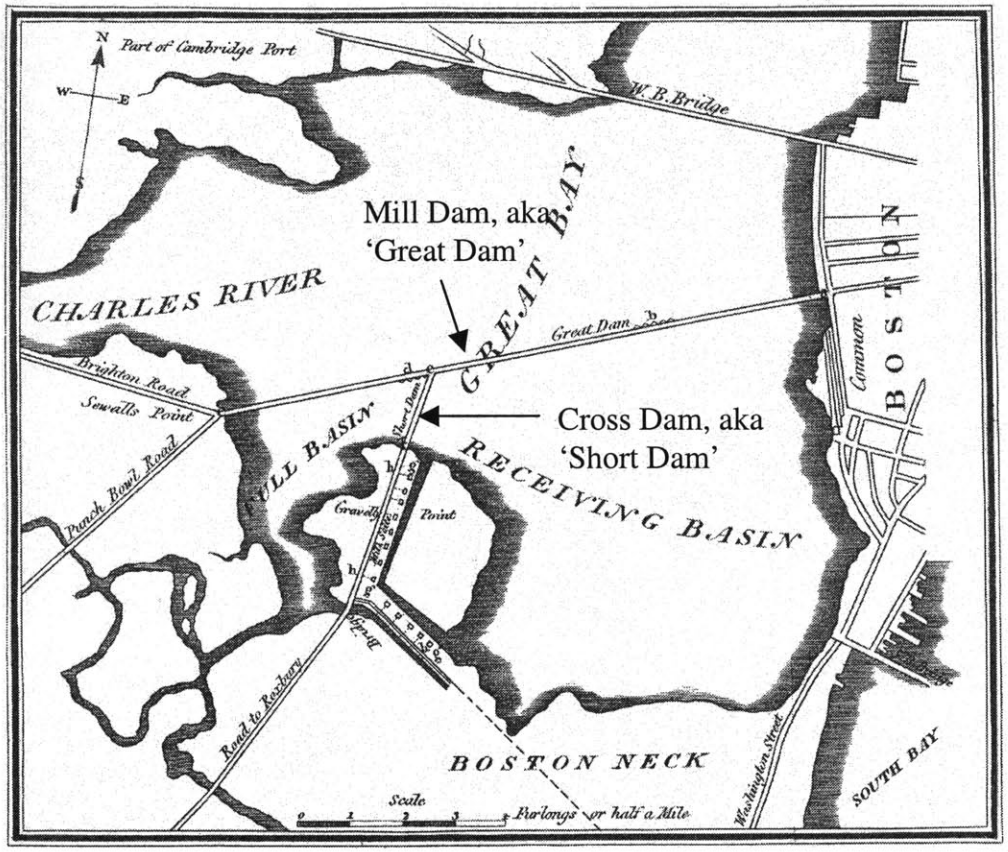


Figure 1.6a – The Mill Dam and the Cross Dam (Whitehall, 1963)



1.6b – A cross section of the Mill Dam (Aldrich and Lambrechts, 1986)

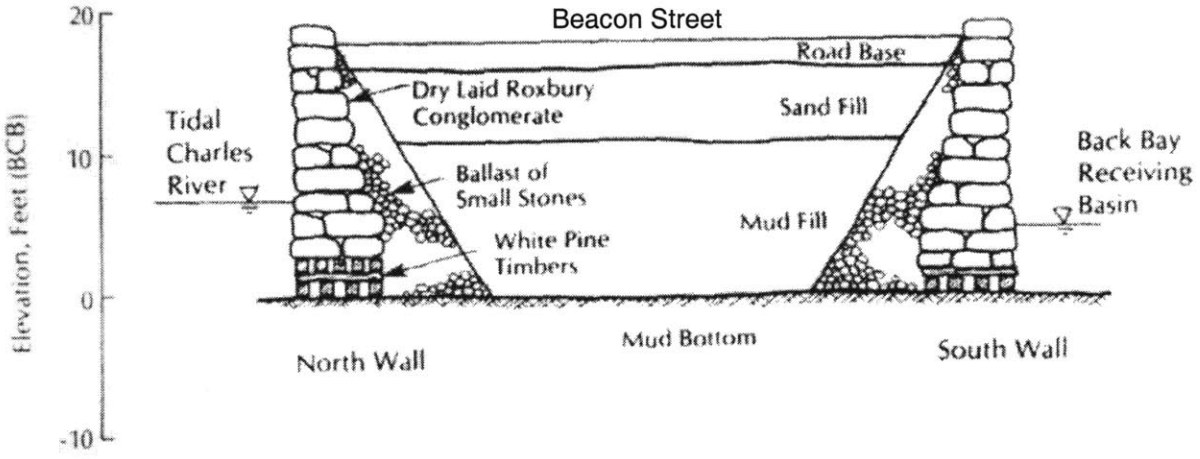


Figure 1.7 – The First Railroads in the Back Bay (Adapted from Aldrich, 1970)

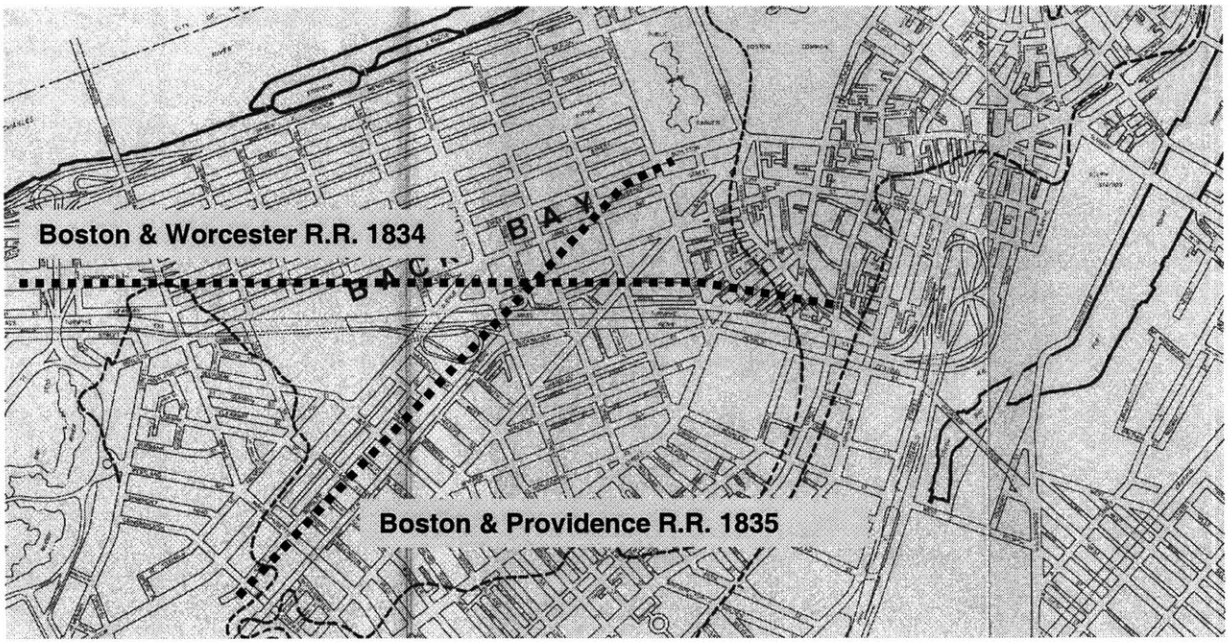


Figure 1.8 – Mining the sand hills in Needham and filling the rail cars using a very primitive steam shovel (Whitehill, 1963).

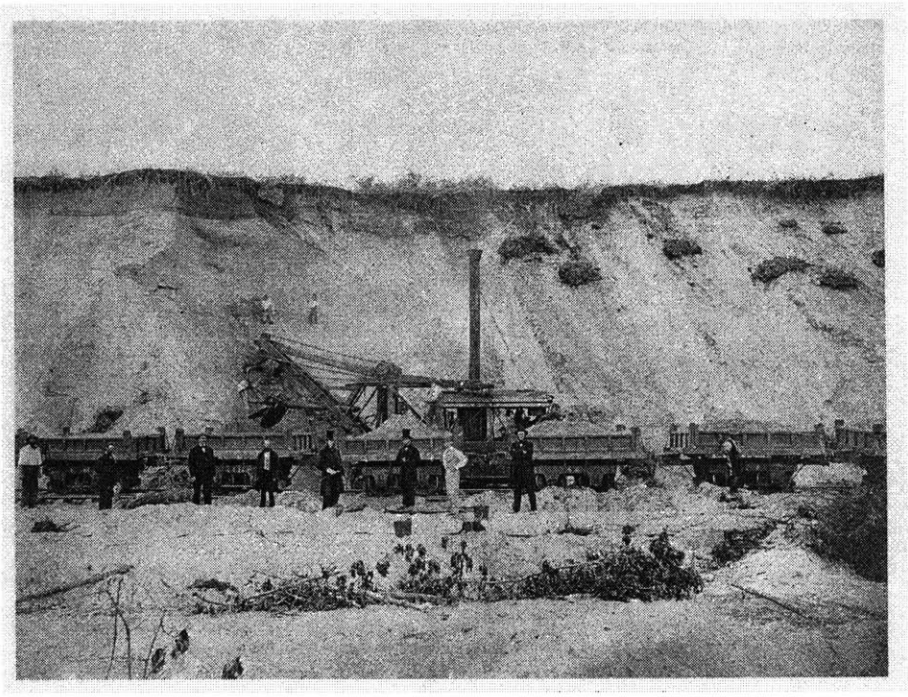


Figure 1.9a – Typical Back Bay Pile Foundations

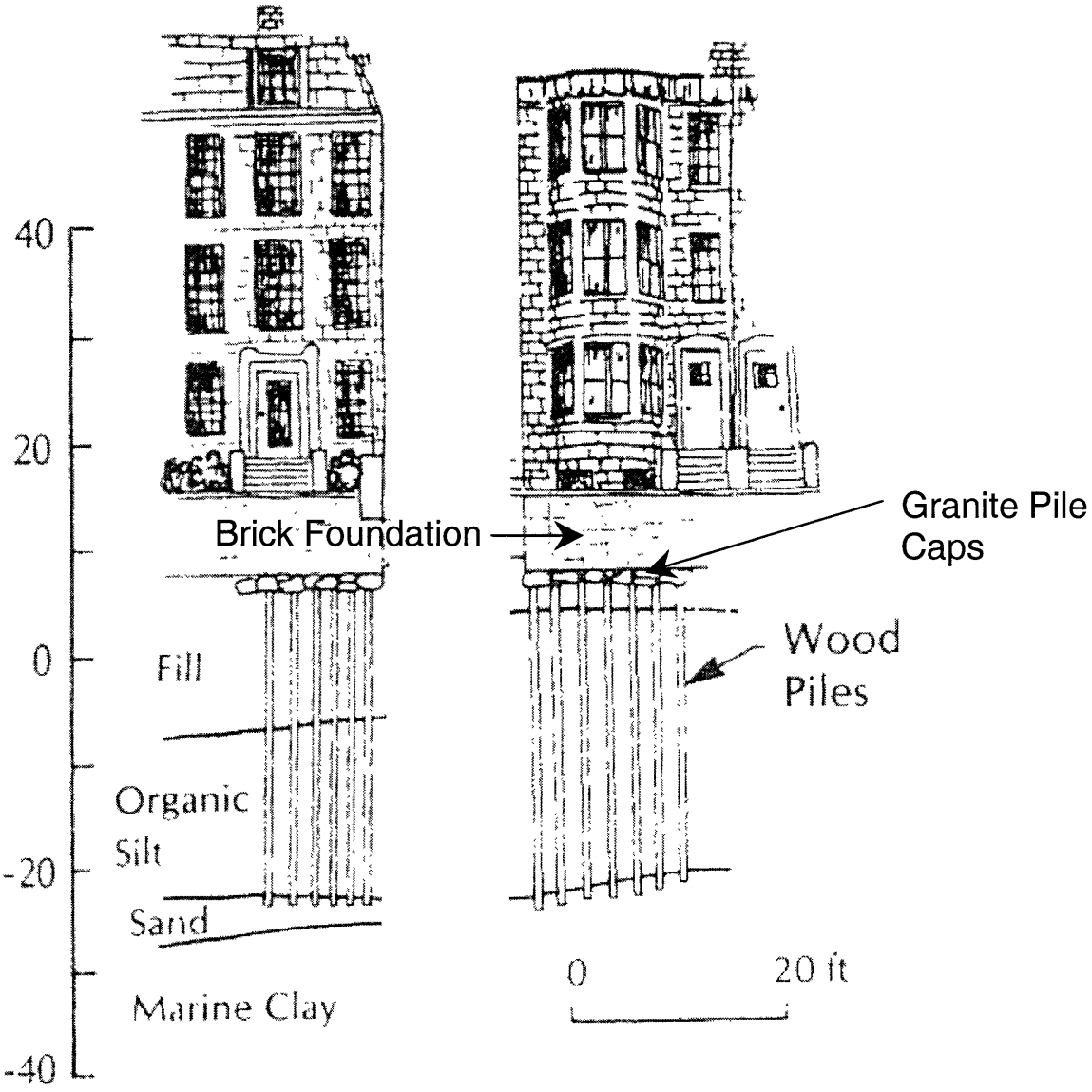


Figure 1.9b – Transverse section of a typical Back Bay house.
(Bunting, 1967)

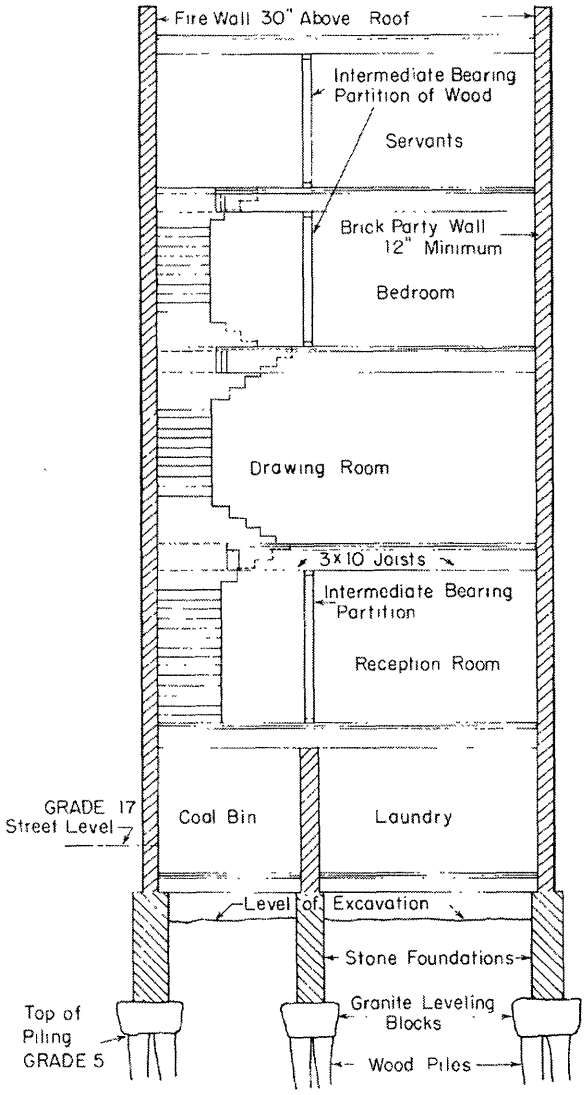


Figure 1.9c – The pile schedule for 92-93 Beacon Street: each pile is numbered individually
(Bunting, 1967)

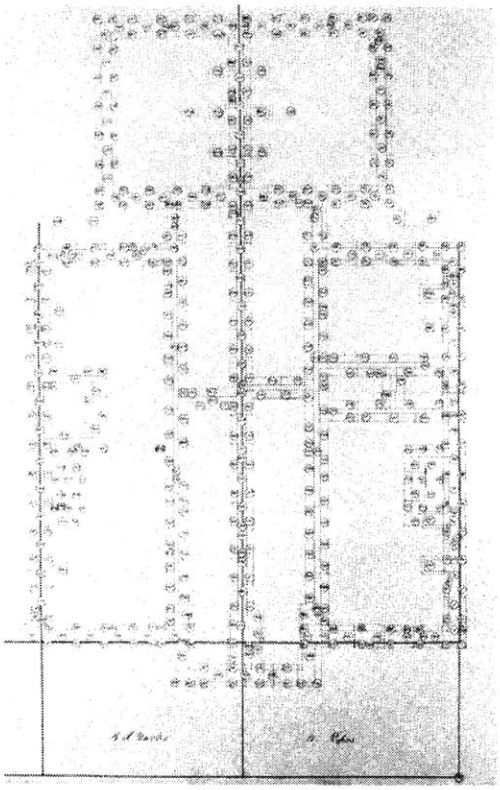


Figure 1.9d – The contract drawing for 92-93 Beacon Street
(Bunting, 1967)

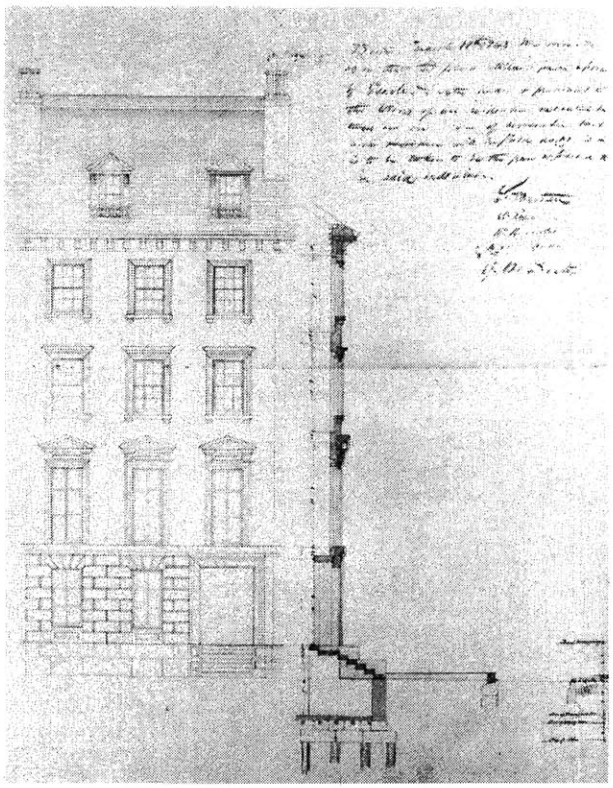


Figure 1.10 – Decayed Wood Piles
(Aldrich and Lambrechts, 1986)

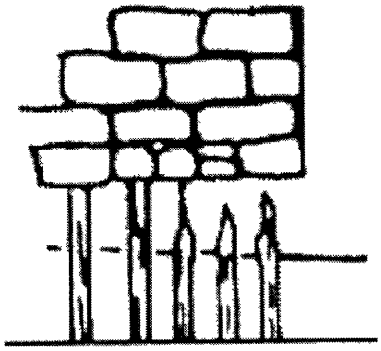


Figure 1.11 – A Plan for the design of the Boston Main Drainage System
(Clark, 1885)

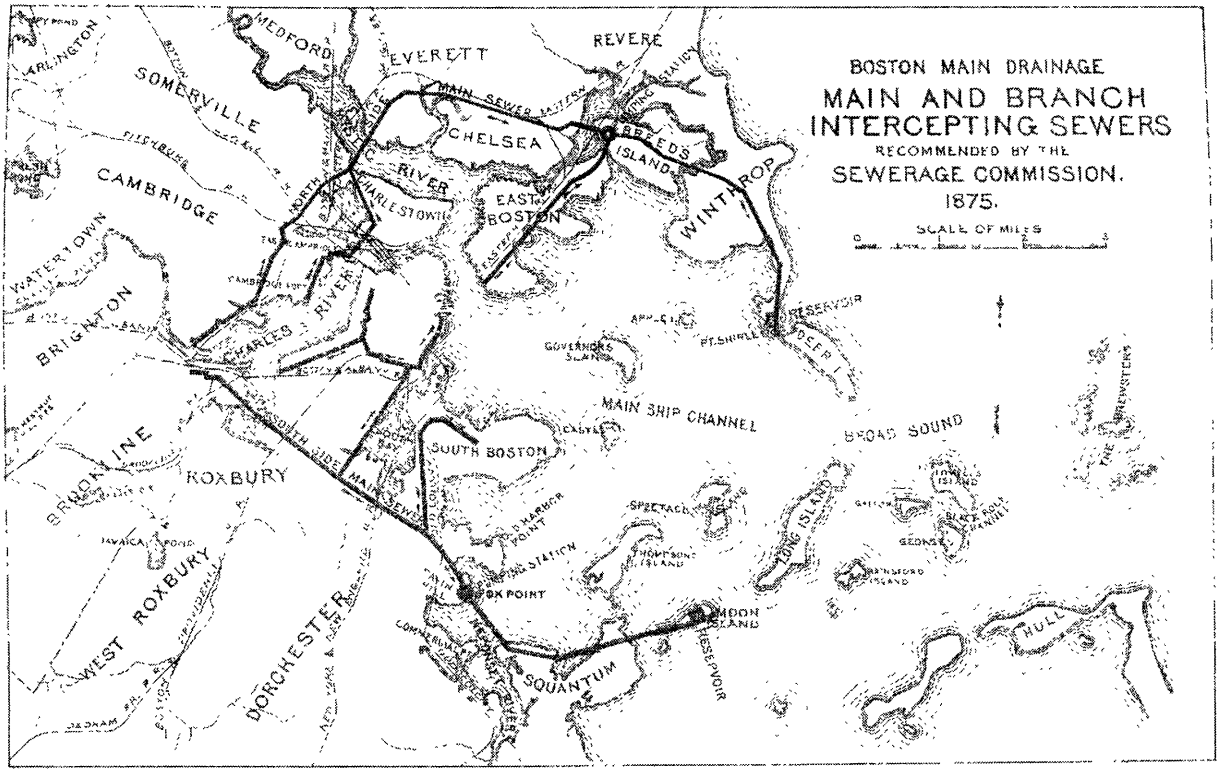
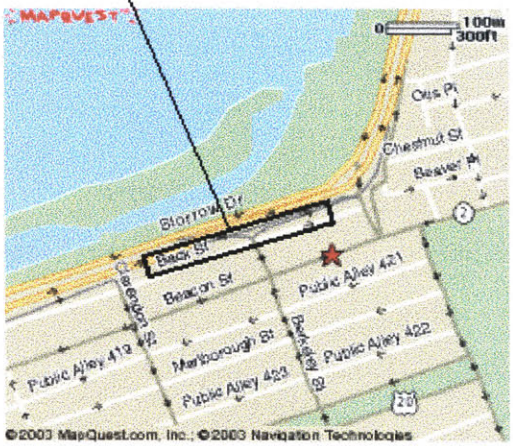


Figure 1.12 – The sea wall constructed along the north side of Back Street.



A cross section of the sea wall
(Aldrich and Lambrechts, 1986)

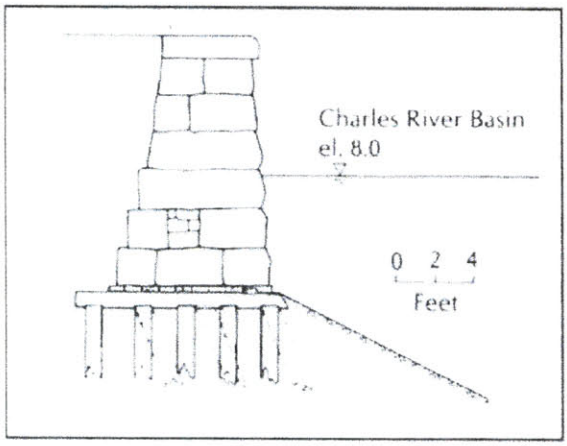


Figure 1.13a – A cross section of the Boston Marginal Conduit (Aldrich and Lambrechts, 1986)

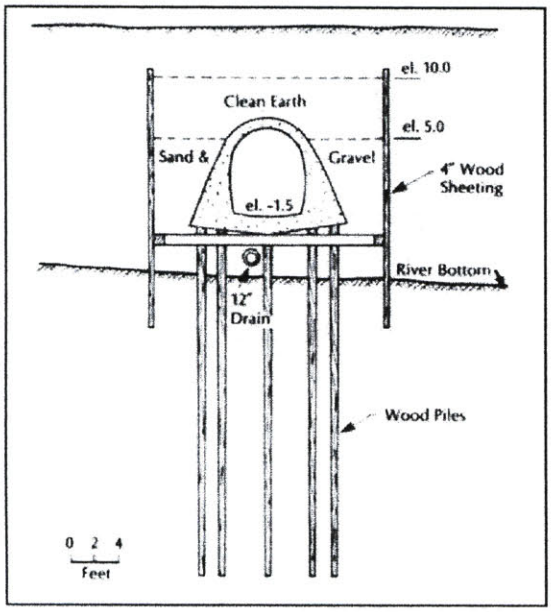


Figure 1.13b – Location of the Boston Marginal Conduit (http://www.mapquest.com)

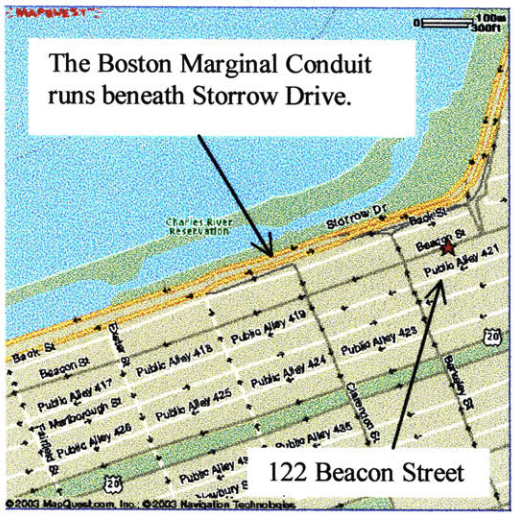


Figure 1.14a – A cross section of the West Side Interceptor (Aldrich and Lambrechts, 1986)

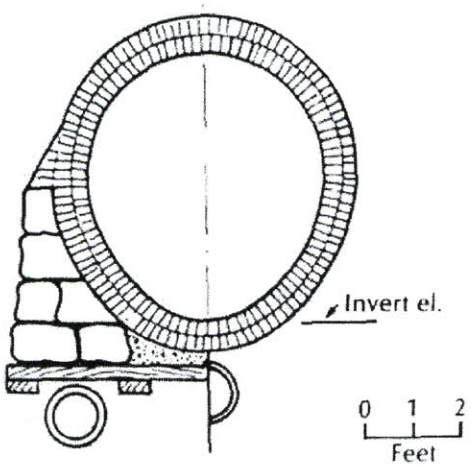


Figure 1.14b – Location of the West Side Interceptor (http://www.mapquest.com)

CHAPTER 2: GROUND PENETRATING RADAR: DEFINITION AND RESEARCH DESIGN

2.1 Definition: What is ground penetrating radar?

RADAR (**Ra**dio **D**etection **a**nd **R**anging) works by transmitting short duration pulses of electromagnetic energy into the environment and timing the return echo from a reflector. There are many common radar applications, including air traffic control, satellite surveillance and remote sensing, meteorology, and highway speed traps. For conventional radar applications, air is the carrier medium for the electromagnetic (EM) signal. For radar investigations of the subsurface, the earth is the carrier medium. Ground-coupled radar applications are collectively termed ground penetrating radar.

The idea of using EM signals to locate remote buried objects can be traced back to several German patents granted in the early 1900s[1]. However, substantial development of pulsed radar technology did not occur until the early 1970s, following closely after the revolution in electrical engineering which occurred during the 1960s. Ground penetrating Radar (GPR) has become a very popular and widely used technique in the geotechnical community since the mid-1980s. It is used for a variety of geological, environmental, engineering, archaeological and forensic applications.

2.1.1 Principles of Operation

In principle ground penetrating radar is similar to reflection seismology. A radar system consists of five basic components: a control console, a power source, a transmitter

[1] "The first use of electromagnetic (EM) signals to locate remote buried objects is attributed to Hülsmeyer in a German patent in 1904, but the first published description of such investigations was by Leimbach and Löwy (1910), also in German patents. The systems used in these investigations employed continuous wave (CW) transmission. Hülsenbeck (1926) developed the first use of radar to investigate buried features" (Reynolds, 1997).

antenna, a receiver antenna, and a computer to record and store the data (figure 2.1). The transmitter antenna generates a succession of electromagnetic pulses (radiowaves) at a given frequency and repetition rate determined by the characteristics of the antenna. The receiver scans for reflected signals at a fixed rate for the duration of the time window of the transmitted pulse. Signals picked up by the receiver are recorded and stored in the computer for real-time viewing and data processing. As the antennas are moved along a survey line, data are gathered at discrete station intervals. At each station, the two-way travel time of the EM pulse is recorded and plotted against distance.

Since many data processing methods used for GPR are borrowed from seismic imaging techniques, the data output, a radargram, is very similar to a seismogram. The radargram, often called a 'wiggle plot,' is nothing more than a series of vertical traces stacked side by side. Each trace, or 'wiggle,' records the amplitude of the reflected pulse in time. The greater the depth to a reflector, the longer the time delay for the return pulse echo, and the further down trace the amplitude inflection.

2.1.2 The Propagation of Radiowaves

The propagation of electromagnetic fields in the ground is governed by a set of four equations developed by James Clerk Maxwell in 1873 (Appendix IV). These equations describe all electromagnetic phenomena in terms of three material constants: magnetic permeability, electrical conductivity, and the electrical permittivity. The success of GPR depends on the relative contrasts of these properties in subsurface media. Since most earth materials are nonmagnetic, GPR is most sensitive to contrasts in electrical conductivity and permittivity.

We must also consider that the velocity of an EM wave and the attenuation of EM energy in earth materials depends on composition and water content. The velocity of radiowaves in any medium depends on the speed of light in free space ($c = 0.3m/ns$), relative dielectric constant (ε_r), and the relative magnetic permeability (μ_r). The velocity of radiowaves in a material (V_m) is given by the following equation (Reynolds, 1997):

$$V_m = \frac{c}{\sqrt{(\varepsilon_r \mu_r / 2)[(1+P^2)+1]}} \qquad (1).$$

P is the loss factor, defined such that $P = \sigma / \omega\varepsilon$, where σ is the conductivity and $\omega = 2\pi f$. Frequency is given by f, ε is the permittivity such that $\varepsilon = \varepsilon_r \varepsilon_0$, and ε_0 is the permittivity of free space (8.854×10^{-12} F/m). As previously stated, most earth materials are nonmagnetic, and $\mu_r = 1$ for nonmagnetic materials. In low loss materials we may consider that $P \approx 0$ so equation (1) may be simplified to (Reynolds, 1997):

$$V_m = \frac{c}{\sqrt{\varepsilon_r}} = \frac{0.3}{\sqrt{\varepsilon_r}} \qquad (2).$$

The contrast in dielectric constant between adjacent layers in the ground is what governs the reflection of incident EM radiation. The greater the contrast, the greater the amount of energy reflected. The proportion of energy reflected is given by the amplitude reflection coefficient, R, which is determined by the contrast in radiowave velocities in adjacent media. More fundamentally, it is determined by the dielectric contrast at the boundary between adjacent media. The magnitude of the reflection coefficient always lies in the range ± 1, and the proportion of energy transmitted equals $1 - R$. The amplitude reflection coefficient is given by (Reynolds, 1997):

$$R = \frac{V_1 - V_2}{V_1 + V_2} \qquad (3),$$

where V_1 and V_2 are the radiowave velocities in layers 1 and 2, and $V_1 < V_2$. Equations 3 and 4 (below) apply to situations where EM incidence occurs normal to a planar reflector, assuming no other signal dissipation. Thus R may also be expressed as (Reynolds, 1997):

$$R = \frac{\sqrt{\varepsilon_2} - \sqrt{\varepsilon_1}}{\sqrt{\varepsilon_2} + \sqrt{\varepsilon_1}} \qquad (4),$$

where ε_1 and ε_2 are the relative dielectric constants (ε_r) of layers 1 and 2. Equation 4 is valid as long as there are no unusually highly conductive reflectors (e.g. metal objects, highly conductive sulfide or graphite ores, water-saturated soils, salt-freshwater interface, etc.) present in the subsurface (Parasnis, 1997).

2.1.3 Signal Strength and Resolution

Although it is a popular and attractive technique for probing the subsurface, GPR is only as good as its ability to resolve the target in question. There are several factors which cause a decrease in signal strength and therefore poor resolution. Whenever a radiowave passes through a dielectric boundary in the subsurface, part of the energy is reflected back to the transmitter and part is transmitted across the boundary with a lower energy. Energy is also lost by absorption when electromagnetic energy is converted to heat. Geometrical spreading accounts for additional loss of signal strength. The radar signal is transmitted as a cone-shaped beam, and as the signal spreads out there is a reduction in energy per unit area at a rate of $1/r^2$, where r is the distance traveled.

Another important factor governing signal strength is the skin depth. Skin depth is the depth by which the signal has decreased in amplitude to $1/e$ $(1/e = 37\%)$ of its initial value. Finally, a fundamental cause of energy loss is due to attenuation, which is a complex function of the dielectric and electrical properties of the media through which the signal is propagating as well as the frequency of the signal itself [2]. In general, total signal loss can be attributed to five factors: antenna losses, transmission losses between the air and the ground, losses caused by the geometrical spreading of the radar beam, losses due to scattering of the radar signal from the target, and attenuation within the ground as a function of the material properties (Reynolds, 1997).

Resolution is partially governed by an unavoidable natural phenomenon: the earth eats high frequencies. This is the tradeoff with GPR: resolution is sacrificed for depth. Antennas which transmit at high frequencies have excellent resolution but cannot resolve beyond shallow depths. Antennas which transmit at low frequencies probe greater depths but at the expense of good resolution.

2.1.4 Data Acquisition

Radar systems are generally deployed in three modes: common-offset reflection

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[2] From Reynolds (1997): If E_0 is the peak electric field strength on transmission, and if at a distance x it has reduced to E_x, the ratio of the two amplitudes is given by: $E_0/E_x = e^{-\alpha x}$, where α is the attenuation coefficient: $\alpha = \omega\left\{\left(\frac{\mu\varepsilon}{2}\right)\left[\left(1+\frac{\sigma^2}{\omega^2\varepsilon^2}\right)^{1/2}-1\right]\right\}^{1/2}$, where $\omega = 2\pi f$ where f is the frequency in Hz, μ is the magnetic permeability (4π x 10^{-7} H/m), σ is the bulk conductivity at the given frequency (S/m), and ε is the dielectric permittivity where $\varepsilon = \varepsilon_r$ x 8.85 x 10^{-12} F/m and ε_r is the bulk relative dielectric constant. This formula is only valid for non-magnetic materials. The loss factor P is equivalent to the term $(\sigma/\omega\varepsilon)$, such that: $P = \sigma/\omega\varepsilon = \tan D$. In addition, skin depth can be expressed: $\delta = 1/\alpha$. When $\tan D \ll 1$, $\delta = (2/\sigma)(\varepsilon/\mu)^{1/2}$. Or, numerically: $\delta = \left(5.31\sqrt{\varepsilon_r}\right)/\sigma$, where σ is in mS/m.

profiling, common-midpoint (CMP) or wide-angle reflection and refraction (WARR), and transillumiation or radar tomography. This thesis uses common-offset reflection profiling and common-midpoint velocity sounding to acquire data. Figure 2.2 is an example of common-offset reflection profiling. The receiver and transmitter are maintained at a fixed distance while moved along a survey line to produce a 2-D profile. Figure 2.3 is an example of common-midpoint profiling. The receiver and transmitter are stepped out at fixed intervals from the midpoint of a survey line.

The antennas may be arranged in either monostatic or bistatic mode. In monostatic mode, one antenna is used as both the transmitter and the receiver. In bistatic mode, two separate antennas are used, one to transmit and one to receive. Additionally, the antennas may be shielded or unshielded. Unshielded antennas radiate electromagnetic energy spherically, and thus reflections from objects other than those underground will appear in the radargram as noise in the data. Shielded antennas radiate electromagnetic energy hemispherically, significantly lessening the amount of noise caused by objects on the surface. Only shielded, bistatic antennas were used in this study.

2.2 Survey Design

The success of a GPR survey, or of any near surface geophysical investigation, depends on a clear definition of the problem. Annan and Cosway (1992) outline five important questions to be addressed to ensure an effective survey: What is the target depth? What is the target geometry? What are the target electrical properties? What is the host material? What is the survey environment like? These five questions are discussed below.

2.2.1 What is the target depth?

The depth to the target must be within the range of GPR operating in less than ideal conditions. Though nature may be perfect, it is rarely ideal for geophysics. It is usually best to integrate some factor of safety when making any survey-related estimations. The survey presented in this thesis, which was conducted at 122 Beacon St, Boston, MA, has several targets that may be encountered, based on what is known about 19th century foundations. Targets may include brick, granite or other rock, water-soaked or perhaps rotted wood, and water.

There are several pieces of evidence to consider when estimating the target depth. Historical records indicate that when the Back Bay was filled, the streets were built up to El. 18 but building lots were filled only to El. 12. Knowledge of past building practices suggests that the tops of the piles could be as high as El. 7 or as low as El. 5 or even El. 3. If the tops of the piles are exposed above the water table, their depth beneath street level could vary between 5-9 feet (1.5-2.7 meters), and similarly the depth to the tops of the pile caps can be estimated at 3-7 feet (0.9-2.1 meters) below street level. Finally, data from wells in the vicinity of 122 Beacon indicate that water table could be as low as El. 3 or as high as El. 6.5-7 (figures 2.4a, 2.4b) (Lambrechts, 2003).

A pulseEKKO 1000 radar system, manufactured by Sensors and Software, Ltd[3]., was used for data collection. There were three antenna frequencies available: 225 MHz, 450 MHz, and 900 MHz. Depth penetration estimates for these center frequencies are approximately 2.0, 1.0, and 0.5 meters respectively. These estimates are based on the assumption that spatial resolution of the target is about 25% of the target depth (Sensors & Software, 1996). Since a high frequency means better resolution, the ideal survey design uses

[3] http://www.sensoft.ca

the highest frequency that adequately penetrates to the target depth. For this study we used antennas of 225 MHz and 450 MHz frequencies. It was found that the 225 MHz antennas provided adequate resolution for the required depth of penetration. The 450 MHz antennas generally did not penetrate to sufficient depth.

2.2.2 What is the target geometry?

It is important to geometrically qualify the targets to be detected. Size, in three dimensions, is the most obvious and important factor. In this study, there are multiple target geometries. The easiest is the water table, which should approximate a simple planar surface. If the piles are submerged beneath the water table, the GPR will not "see" them and their geometry is moot. However, if the tops of the piles are exposed and not significantly rotted, descriptions of 19th century building practices indicate that the piles were generally six inches in diameter and capped with blocks of rock. Thus, we can anticipate rectangular blocks of rock, and perhaps small, circular pile tops. The rectangular blocks will approximate a planar reflector, like the water table, and the circular pile tops might be expected to produce a hyperbolic or perhaps parabolic reflection. If the piles are exposed and significantly rotted, however, their geometry will be complex and nearly impossible to predict.

2.2.3 What are the target electrical properties?

As stated previously, the two most important electrical properties when using GPR are the relative permittivity and electrical conductivity. Relative permittivity (expressed as the dielectric constant), or the ability of a material to hold charge, is the more important of the two. The success of GPR depends on the magnitude of the contrast between the target and the

host environment, since the contrast is what causes the signal to be scattered and reflected. For this study, the host environment is mostly fill and silty sand (figures 2.5a, 2.5b), for which we may approximate ε_r=120-170. Dielectric constants for most of the other materials are: brick, ε = 4 (Svechnikov, 2003); granite, ε = 5-8 (Reynolds, 1997); water, ε = 81 (Reynolds, 1997); wood, ε = 10-30 (ASI, 2003).

2.2.4 What is the host material?

There are two ways in which the host material must be qualified. First, the electrical properties must be evaluated (above). Second, the degree and spatial scale of heterogeneity in the host material must be considered. If the host material exhibits variations similar to the contrast and scale of the target, detecting the target may become nearly impossible. In this study, the degree and scale of heterogeneity of the host is significantly different from any of the targets (bricks, rock blocks, water table, wood piles). In this case, host/target heterogeneity scales will not render the target undetectable.

2.2.5 What is the survey environment like?

GPR is sensitive to its surroundings, especially in an urban environment like the Back Bay. The presence of extensive metal structures and radio frequency sources or transmitters are two key factors, although something as simple as a wall or a door could have an equally confounding effect. Figure 2.6a is a diagram of our survey site. Though we benefited from the use of shielded antennas, noise was not masked completely. Accessibility is another consideration, in terms of safety, economy, and exposure to unusual conditions or hazards.

As figures 2.6a, 2.6b, and 2.6c illustrate, the survey lines for this study were hallways, and though the workspace was narrow, accessibility constraints did not hamper our work.

Figure 2.1 – The five basic components of a radar system
(Sensors & Software, 1996)

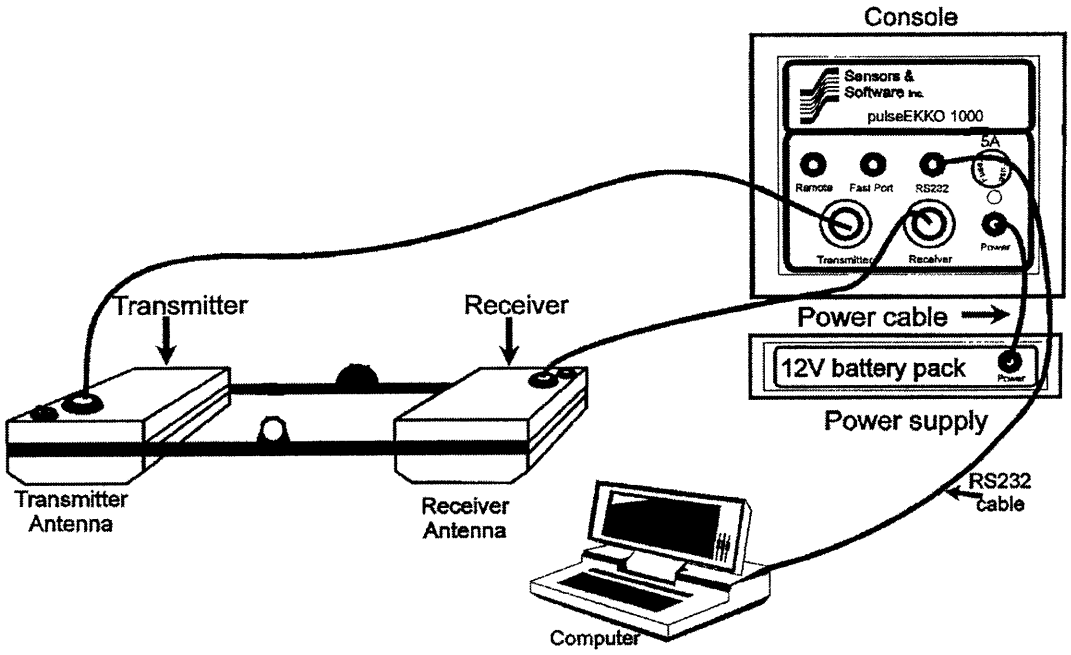


Figure 2.2 – Common Offset Reflection Profiling
(Annan and Cosway, 1992)

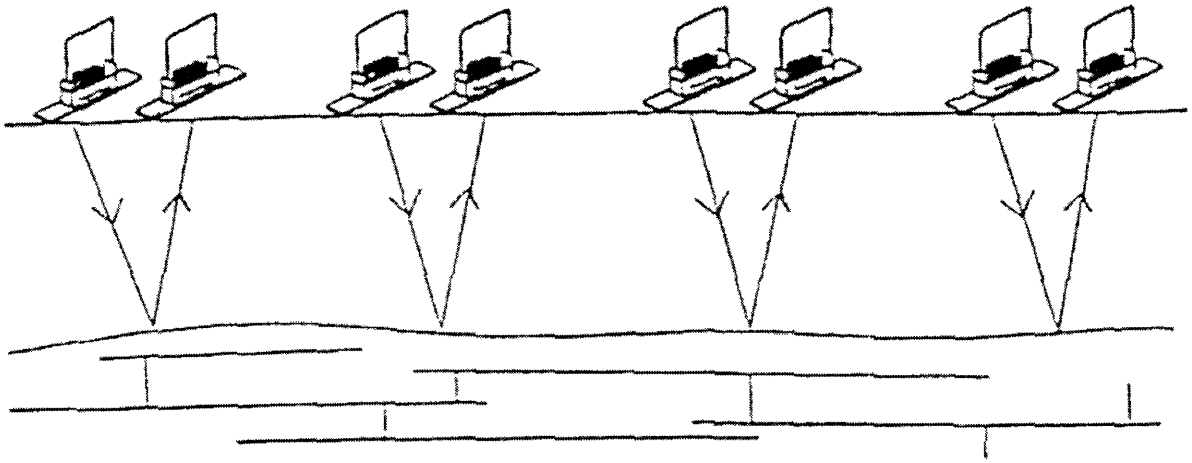


Figure 2.3 – Common-Midpoint Velocity Sounding
(Reynolds, 1997)

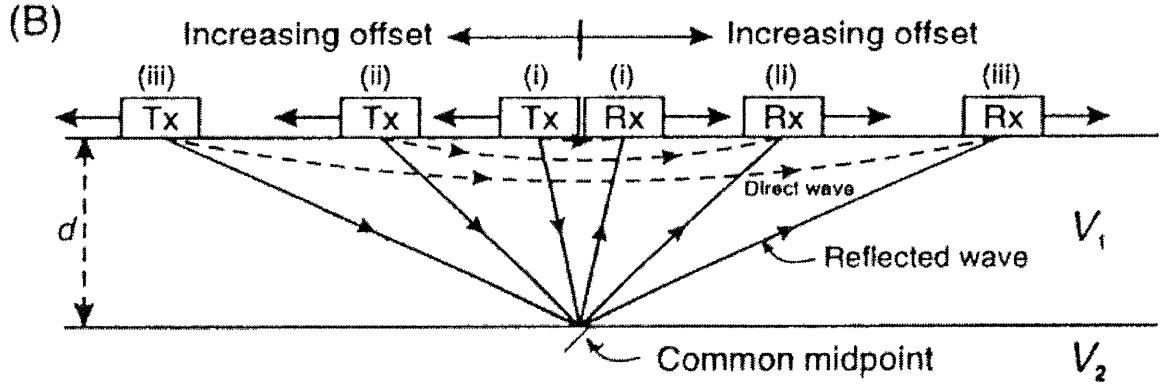
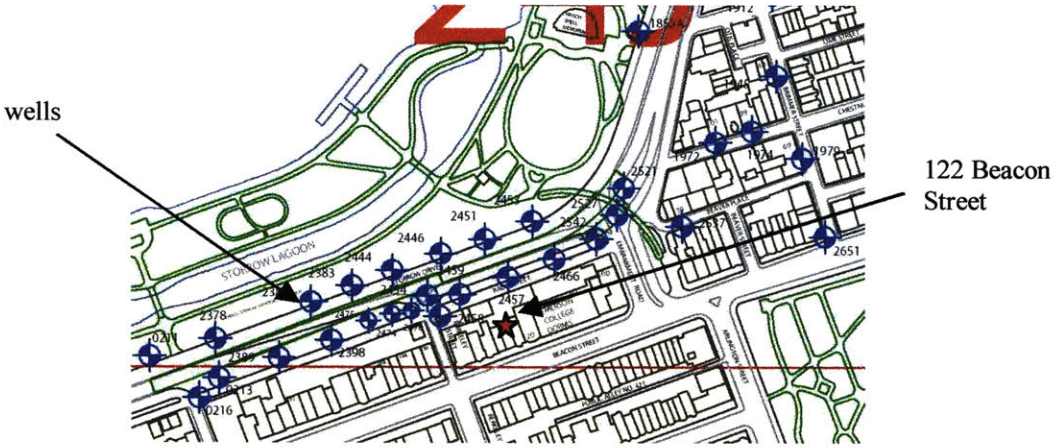


Figure 2.4a – A map of monitoring wells near 122 Beacon Street (http://bostongroundwater.org)



2.4b – Available water level data for wells near 122 Beacon Street: Winter 2002 – Spring 2003 (http://bostongroundwater.org)

well #	win 2002	spr 2002	sum 2002	sum 2002	fall 2002	win 2003	spr 2003
24j-0211		3.99	4.28	4.18	3.54		3.35
24j-2378	3.22	4.77	4.46	4.26	2.73		4.26
24j-2389	2.58	0.47	4.27	3.02	2.62		2.52
24j-2385	3.92	3.91	3.71	3.65	3.14		3.25
24j-2398	2.54	2.32	2.56	2.60	2.17		1.82
24j-2383	2.44	4.66	4.57	4.46	4.06		4.06
24j-2475					1.74		1.77
24j-2474	2.54	0.79	-0.05	2.28	1.24		1.15
24j-2476					2.06		2.16
24j-2444	2.57	4.62	4.14	4.14	4.07		3.87
24j-2454			2.06	1.26	0.66		0.78
24j-2458	4.30	3.62	4.49	3.87	3.60		3.84
24j-2446	3.61	3.13	2.55	2.76	2.35		2.15
24j-2459	2.94	3.25	2.29	1.80	1.43		1.64
24j-2451	4.19	5.95	5.64	5.45	5.31		5.20
24j-2457	2.82	4.62	3.32	2.52	2.22		2.62
24j-2453	3.25	5.35	4.99	4.84	4.65		4.64
24j-2466	2.57	3.12	5.85	2.40	2.08		2.60
24j-2542	2.73	2.62	3.27	2.37	2.03		2.46
24j-2527	2.72	3.07	3.30	2.02	1.74		2.53
24j-2521	2.43	2.42	2.99	2.31	1.93		2.04
24j-2537	6.91	6.33	6.54	4.98	< 6.18	plugged	plugged
24j-1972	3.27	3.77	4.12	2.18	1.67	1.62	3.22
24j-1974	3.38	3.79	4.15	2.10	2.48	2.39	3.39
24j-1945				2.47	2.77		
24j-1979	5.14	5.58			5.75	6.52	6.37
24j-2651	4.55	4.71	4.51	4.46	4.30	4.34	4.33

Figure 2.5a – Soil conditions interpolated from borehole data
(adapted from Committee on Subsoils of Boston, 1969)

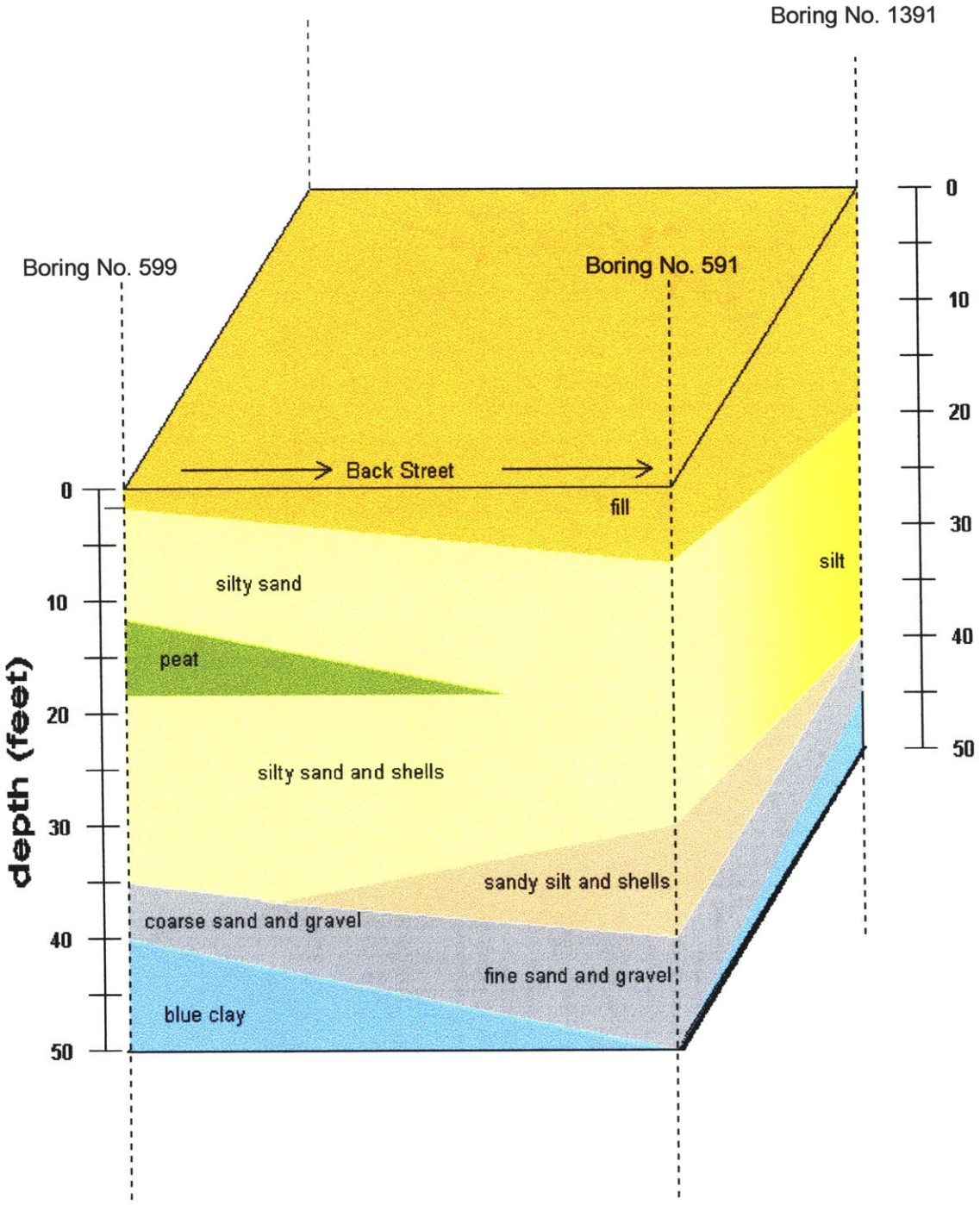


Figure 2.5b – Map showing the location of borings 591, 599, and 1391
(Committee on Subsoils of Boston, 1969)

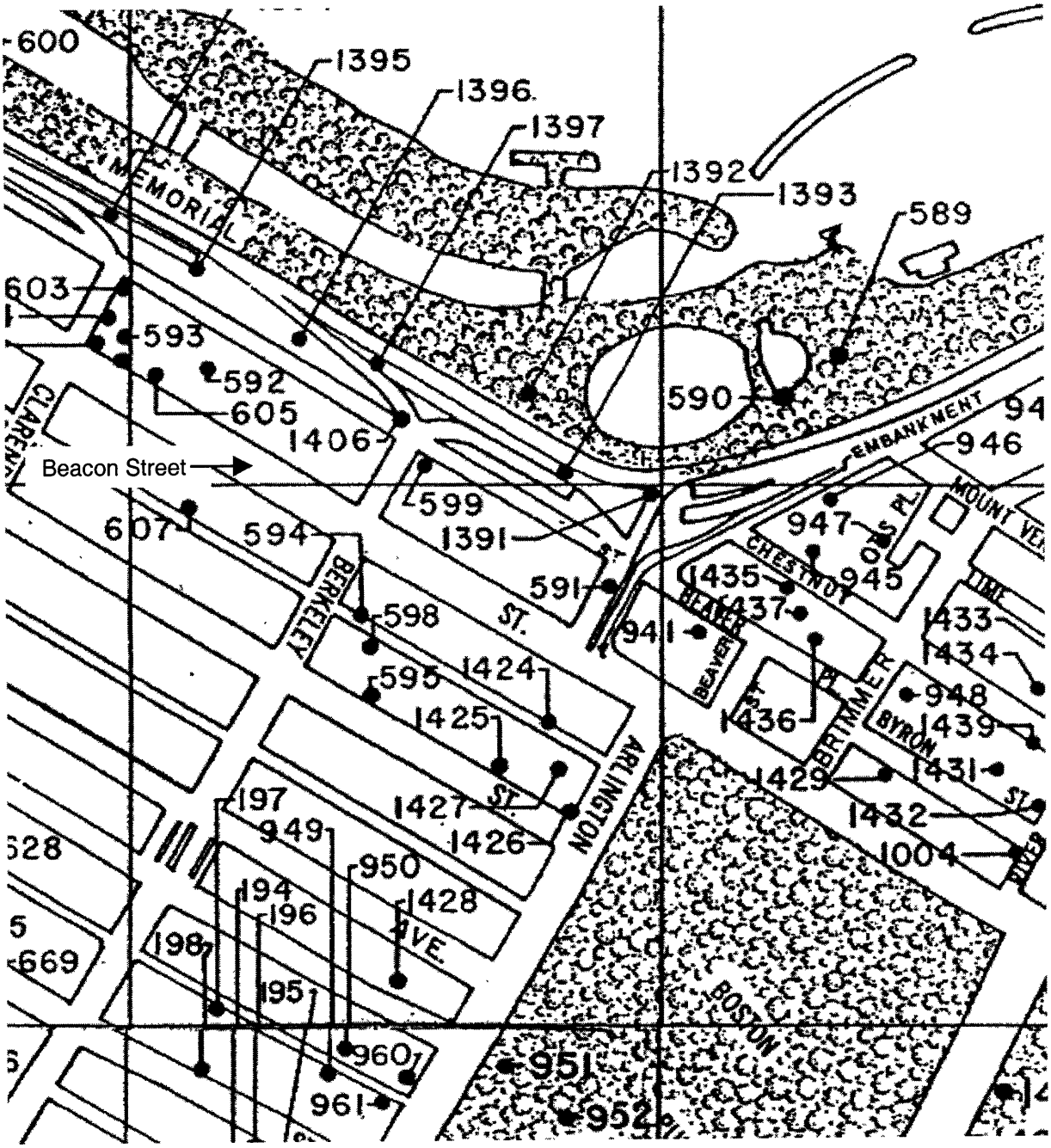


Figure 2.6a – A diagram of the study site, 122 Beacon Street

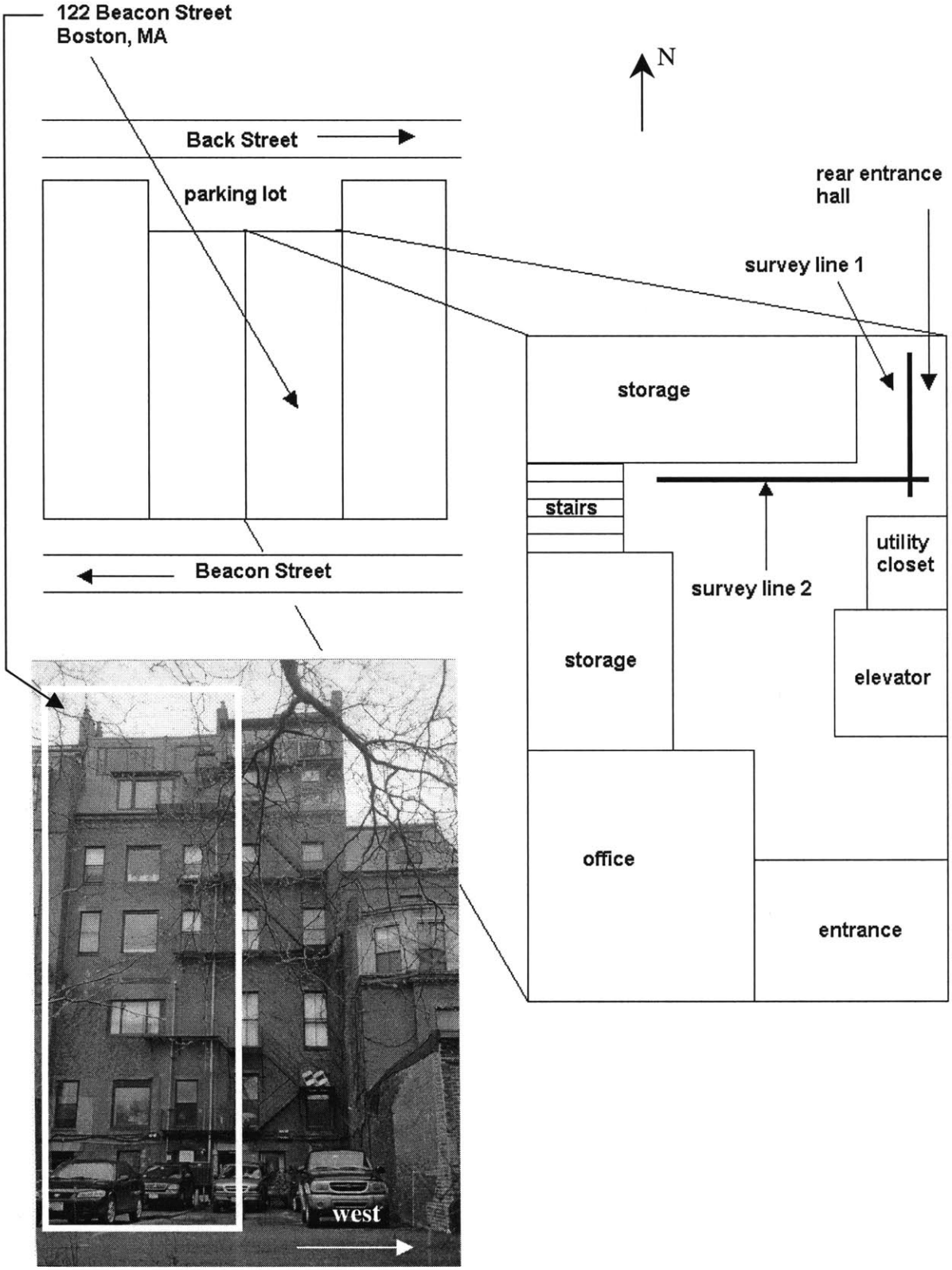
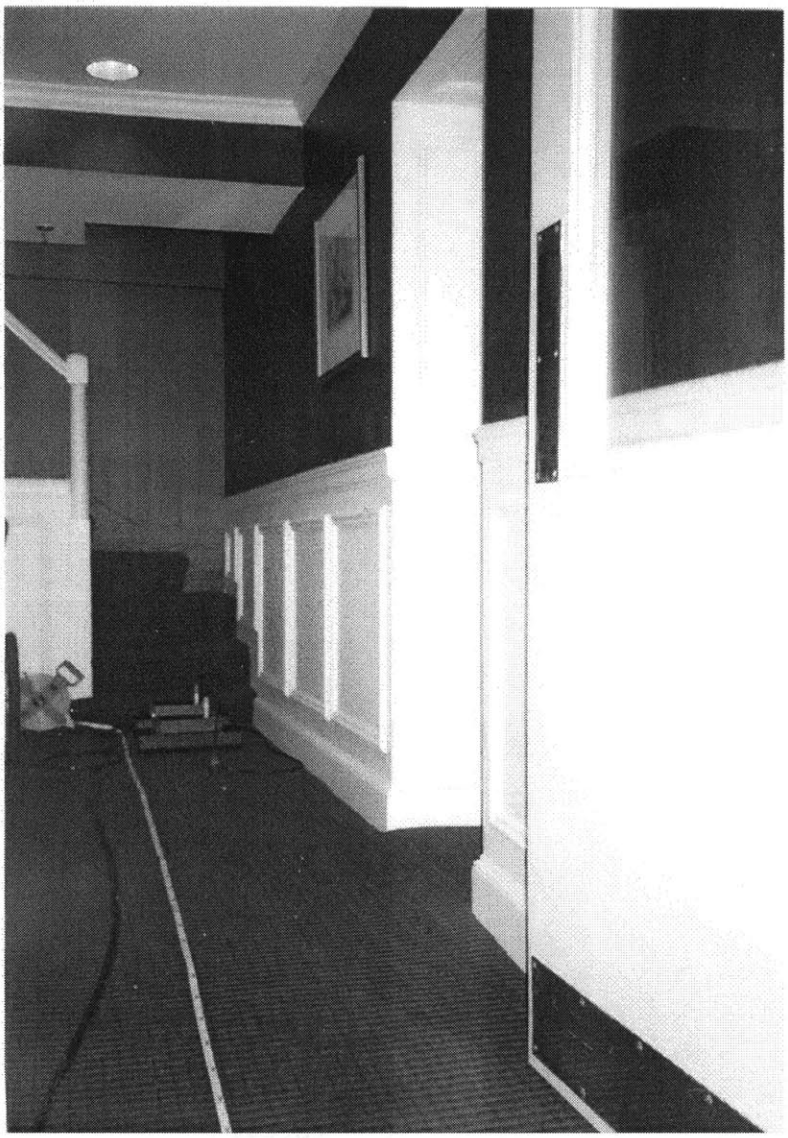


Figure 2.6b – Survey Line 1, looking due north towards Back Street

Figure 2.6c – Survey Line 2, looking due west



CHAPTER 3: GROUND TRUTH

The interpretation of geophysical data is an art. The subtleties and nuances of a radargram may well go unnoticed by the untrained eye. As significant as the data is, corroborative evidence in the form of ground truth and simple observations is equally important. In this study, ground truth comes from three sources: well data, observations from recent excavations within one city block of the study site, and the results of a resistivity survey. We also make inferences about the local hydrology from historical as well as recent evidence.

3.1 Borehole Data

Since 1920 the Committee on Subsoils of Boston, in cooperation with the USGS, has been collecting boring data from Boston and surrounding areas. The data is accumulated from the Boston Society of Civil Engineers, consultants, contractors, government organizations, and others and is periodically compiled and published. There are several boreholes in the vicinity of 122 Beacon Street for which data has been published (figure 2.5b). Figure 2.5a is a three dimensional interpolation of the stratigraphy at our study site based on the borehole data. Although historical estimates suggest that the fill was placed to a depth of approximately 20 feet, the borehole data suggests that the soil beneath the block of Beacon Street between Embankment Road / Arlington Street and Berkeley Street is approximately 0-6 feet of fill and 6-30 feet of silty sand and shells with beds of peat and fine silty sand interfingering or pinching out within the silt. Both the fill and the silt layers taper northward into thicker, more homogenous deposits.

3.2 Well Data

The Boston Groundwater Trust monitors a series of wells throughout the Back Bay. Wells are monitored 4-5 times each year, although some of the data is sporadic because wells are frequently inaccessible, damaged, or plugged. There are approximately 30 wells within a two-block radius of our study site (figure 2.4a). All have been monitored consistently since the winter of 2002 (figure 2.4b). In general, water levels have been relatively low during this period. With the exception of well 24J-2451, all wells show water levels below El. 5.

3.3 Test Pits

Observations from recently dug test pits provide a second source of ground truth. An excavation at 146 Beacon Street in the spring of 2003 determined that the tops of the piles had been cut off at El. 6.5-7.0. The water table was at El. 6.5, just high enough to submerge the tops of the piles, which were found to be in good condition. A second excavation at 130 Beacon Street, also in the spring of 2003, revealed similar conditions (Lambrechts, 2003).

3.4 Resistivity Survey

A resistivity survey was performed behind 122 Beacon Street along a narrow strip of grass between Back Street (on the north side of the seawall) and Storrow Drive near several of the actively monitored wells (figures 3.1a, 3.1b). The results (figures 3.2a, 3.2b, 3.2c) indicate that the water table is at a depth of approximately 1 meter (3 feet) below ground surface. These results contradict the most recent well readings for the wells near the survey line, which indicate that the water table is at a depth of approximately 3 meters (10 feet)

below ground surface. However, a meaningful comparison would be best attained if the wells were measured the same day the resistivity survey was performed.

3.5 Hydrology and Infrastructure

The hydrological setting can be inferred from both historical and recent evidence. 122 Beacon Street is located near four potential groundwater sources/sinks: the Mill Dam (figure 1.7b), the West Side Interceptor, the Storrow Drive underpass, and the Boston Marginal Conduit (figure 3.3). Both the Mill Dam and the West Side Interceptor run beneath Beacon Street. Based on descriptions of how the West Side Interceptor was constructed (figure 1.14), it probably rests between the walls of the Mill Dam (Lambrechts, 2003). These structures undoubtedly impede groundwater flow across their widths (roughly a north-south direction). Despite its suspected location within the dam, the interceptor, like many other sewers, is a potential sink. The Storrow Drive underpass, described earlier, is a known groundwater sink. The Boston Marginal Conduit (figure 1.13) could also be both a groundwater sink and a barrier to flow, as described earlier. Finally, the sea wall bordering the north side of Back Street (figure 1.12) is also a likely barrier to flow.

Figure 3.1a – Resistivity Survey Site

Looking east along the eastbound land of Storrow Drive

Looking west along the eastbound lane of Storrow Drive

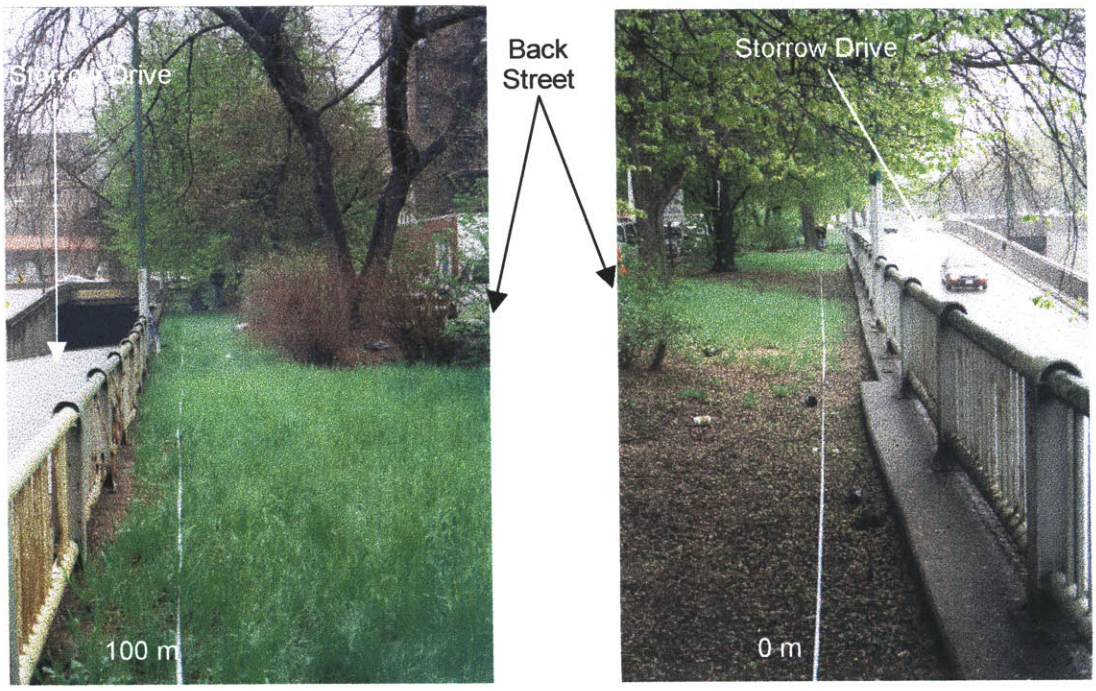


Figure 3.1b – Resistivity Survey Site

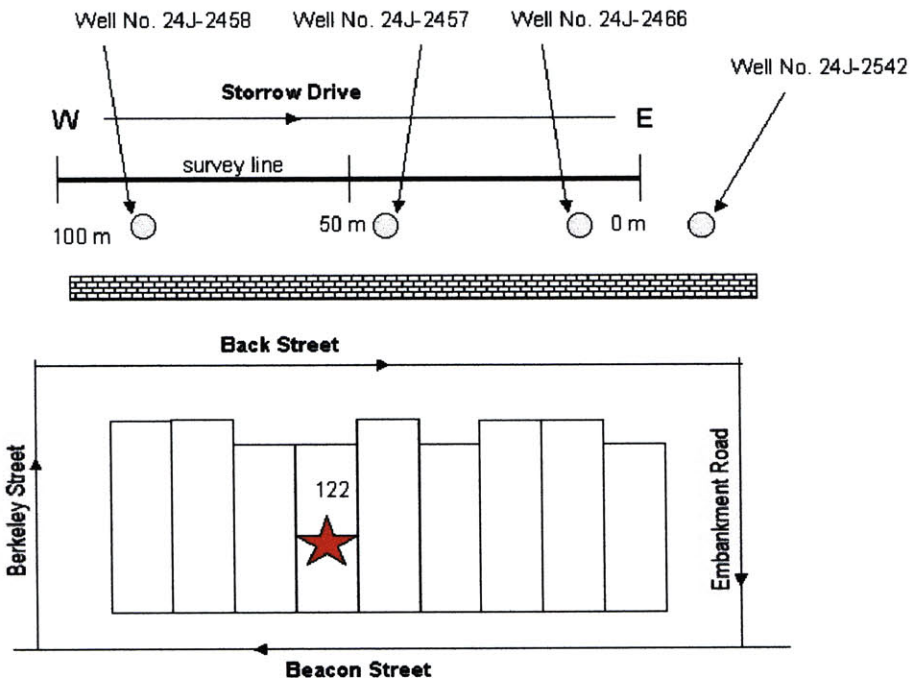


Figure 3.2a – Resistivity Survey Results (Logarithmic Plot)

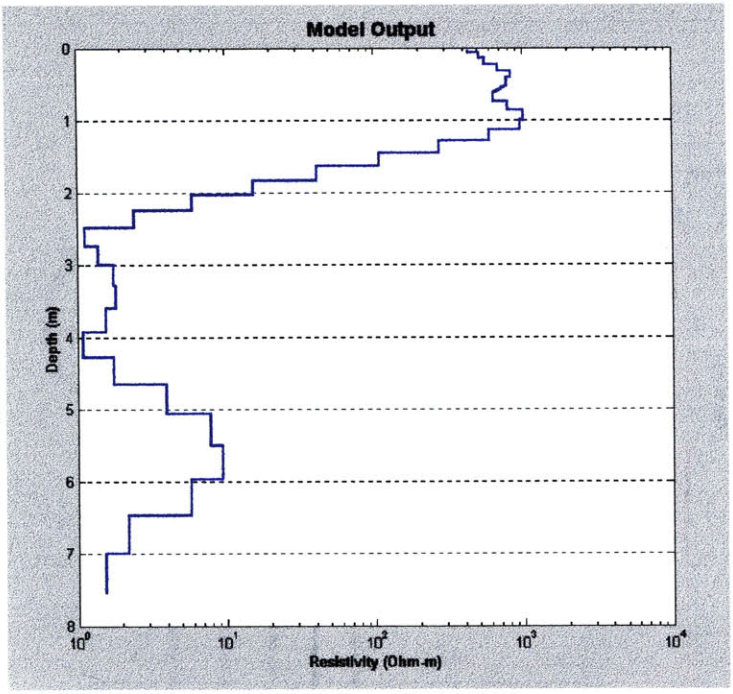


Figure 3.2b – Resistivity Survey Results (Linear Plot)

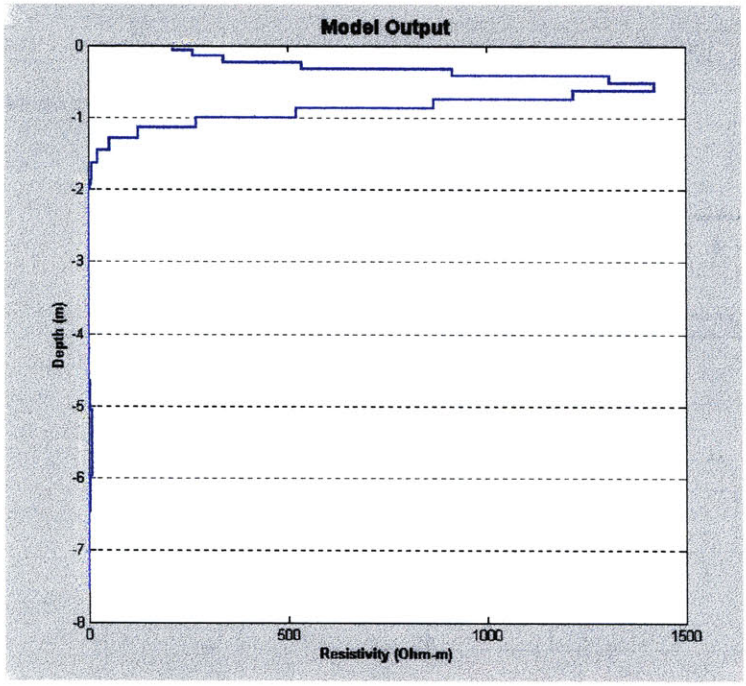


Figure 3.2c – Analysis of Resistivity Survey

Integrating the true resistivity inversion yields a curve (green) whose inflection point(s) reveals the depth to the boundary between resistive layers, or in this case, the water table. As indicated on the plot, the water table is at about a depth of 1 meter.

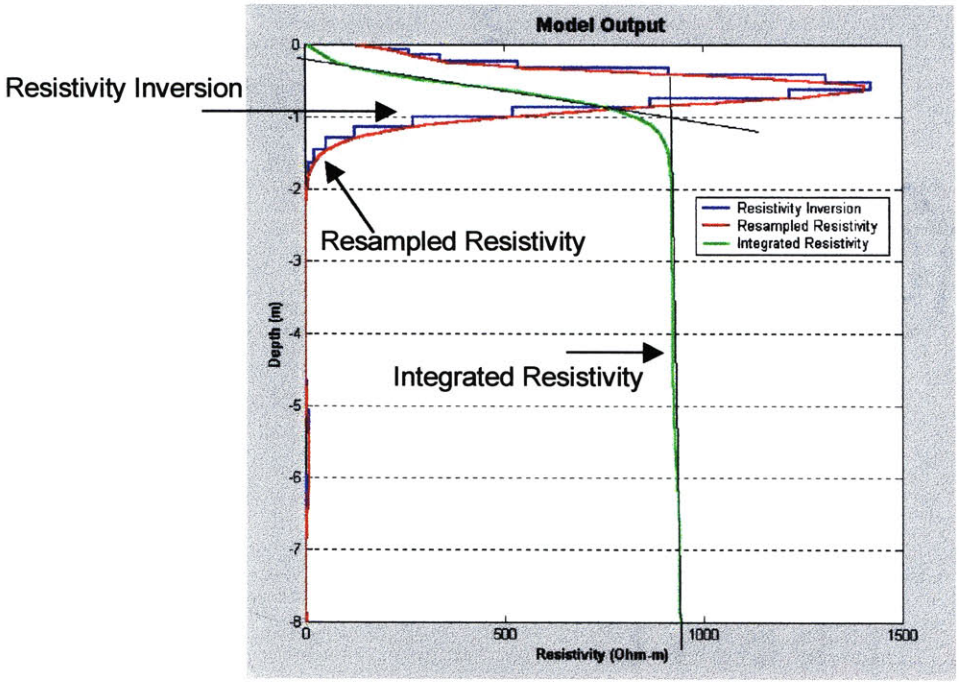
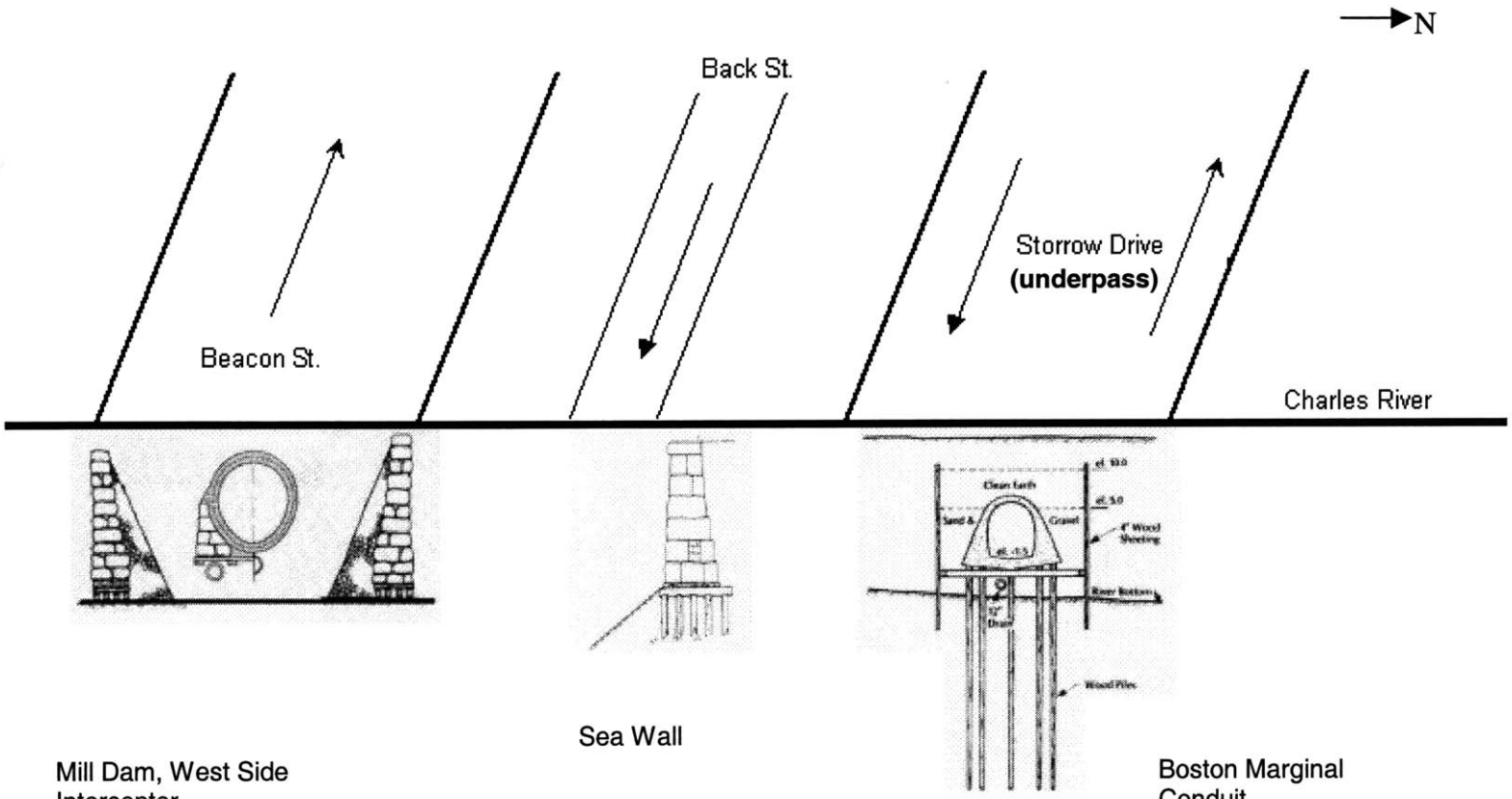


Figure 3.3 – The Subterranean Infrastructure



CHAPTER 4: DATA PROCESSING

GPR is only as good as its ability to resolve a given target, and interpreting radar data is a skill guided by intuition and experience. To that end, data processing is highly biased. We should note that in many cases a radargram need not be processed. However, there are a few basic processing techniques that are frequently applied. These fundamental manipulations can transform raw data into something more suitable for interpretation and evaluation. While most basic data processing is done in real time so the data can be monitored as it is collected, post-survey processing allows the user to apply basic processing techniques systematically and to employ more advanced methods of enhancing the target features.

4.1 Real Time Processing

Two real time data processing schemes were used for this research. Both involve temporal filtering. The first corrects for the very low frequency component of the data, known as "wow." Although the magnitude of this low frequency component and how it appears in the data varies with ground conditions and antenna separation, it is present in all GPR data. The "wow" happens because the low frequency component of the radar signal diffuses into the ground rather than propagates. The lower end of the spectrum thus sees an inductive response rather than a propagating response, and as a result a large transmit pulse may be followed by a slowly decaying transient pulse (Sensors & Software, 1996). To correct for this, a high pass filter was applied to all datasets during data collection and post survey processing. This is sometimes referred to as "dewowing" the data and is often described as a signal saturation correction.

The second real time filtering technique involves selecting a time gain. Because the strength of the radar signal decreases with time, it is necessary to apply a gain to amplify or boost the weaker signals. Generally, the best way to choose an appropriate type and amount of gain is to examine the amplitude fall off time for a given trace or set of traces. This kind of accuracy is not possible to achieve in real time, nor is it necessary. Applying real time gain simply allows one to ensure that the radar is properly functioning and that the signal is penetrating to sufficient depth. There are several types of gain that can be applied. For real time data monitoring in this study, automatic gain control (AGC) was used.

4.2 Post Survey Processing

The pulseEKKO system stores all the data in raw format, regardless of how the data was gained or filtered during collection. As in real time processing, all data sets were dewowed and some type of gain scheme was applied. Post survey processing provides the opportunity to bring out weaker signals, to enhance target features, and to derive quantitative information such as velocity or attenuation. Depending on the type of survey and the amount of spurious activity or noise present in the traces, different processing schemes were applied. The order in which one filters the data is also important since many filters are nonlinear. A brief description of each follows.

4.2.1 Gain

The pulseEKKO processing software provides five different options for applying gain: automatic gain control (AGC), spreading and exponential compensation (SEC), constant gain, no gain, and user gain. The type of gain used should be based on *a priori* data rather than by

simple trial and error. AGC was most frequently used in this study. AGC applies a gain inversely proportional to the signal strength[1]. It is an attempt to equalize all signals. While this type of gain is useful for defining the continuity of reflecting events, it does not preserve relative amplitude information (Sensors & Software, 1996). AGC is useful for this study because the water table, and perhaps also the granite caps, should approximate a relatively continuous reflector.

Spreading and exponential compensation (SEC) was used to isolate reflectors in the CMP survey. SEC gain may be classified as a physical phenomenon based systematic gain, since it attempts to emulate the variation of signal amplitude as it propagates in the ground (Annan, 1993). SEC is a combination of linear and exponential time gain[2]. The goal of SEC is to compensate for energy losses that result in decreased signal strength. For a CMP survey, this is especially relevant for enhancing the signals of deep reflectors, since the offset between transmitting and receiving antennas increases.

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[1] For each point in the trace, AGC is calculated as follows (Systems & Software, 1996):

1. Compute the average signal strength using 0.5 window width points before and 0.5 window width points after the current point,
2. Compute a gain which is inversely proportional to this signal strength,
3. Limit this gain in some fashion,
4. Repeat for every point in the trace,

Apply the gain to the data set.

[2] The SEC gain function is of the form (Sensors & Software, 1996):

$$g(t) = C + (1 + \tau/\tau_w)e^{\beta\tau}$$

where: C = start value (constant)
$\tau = (t - (\tau_w + t_0))$, and τ_w = pulse width
t_0 = timezero
$\beta = \alpha * v / 8.69$
v = radar wave velocity in m/ns
α = radar wave attenuation in dB/m

4.2.2 Enhancing Targets: Spatial and Temporal Filtering

Both spatial and temporal filtering provide a way to enhance targets in a radargram. Temporal filtering is filtering along the time axis of the data set. By contrast, spatial filtering is applied in the distance, or offset, domain. One might think of the spatial domain as trace-to-trace filtering between each trace in a data set, as opposed to filtering temporally within a single trace over each trace of the data set. As previously described, dewowing is one example of a temporal filter used to analyze our data sets. A spatial low pass filter was also sometimes applied to enhance any flat lying reflectors, such as the water table, in the data set.

4.2.3 Damping Noise: Median Filtering and Trace Editing

Median filtering (aka alpha mean trim filtering), and trace editing provide effective means of damping the noise is an data set. Applying a median filter is similar to applying a gain, except the purpose is to mute a noisy signal rather than to amplify a weak one. Median filters can be applied both spatially and temporally. In this case, spatial median filters were more effective at silencing noise than temporal median filters.

4.2.4 Normal Moveout Velocity Estimation

In order to obtain accurate depth estimates, which is the ultimate purpose of this thesis, it is necessary to obtain a measurement of electromagnetic wave velocity in the subsurface material. This is most often achieved by a common midpoint (CMP) velocity sounding or wide angle reflection and refraction (WARR) survey. Both are analogous to a seismic refraction survey. Three CMP surveys, using antenna frequencies of 450, 225, and

225 MHz respectively, were performed to obtain a velocity estimate for the fill that covers most of the Back Bay.

For a CMP sounding, both antennas are moved outward from the central point on the profile line at discrete intervals. (For a WARR sounding, one antenna is held fixed and the other is moved away.) Data are acquired at each interval. The initial antenna spacing is usually n_x, the Nyquist spacing interval for reflection profiling[3]. The move out interval is then $\frac{n_x}{2}$. Maximum separation, if not otherwise constrained by the survey environment, is generally 1-2 times the reflector depth. However, high attenuation in the ground can cause signals to die out before this maximum separation is reached (Annan and Cosway, 1992).

Velocity calculation is based on reflection arrival times having hyperbolic dependence on antenna separation. Time-distance analysis of this 'move-out hyperbola' allows the calculation of wave velocity and therefore target depth. Consider, in figure 4.1, a given midpoint location, M. The travel time of an electromagnetic wave transmitted at point Tx to point D and reflected back to point Rx may be expressed as a function of offset by:

$$t^2(x) = t^2(0) + \frac{x^2}{v^2} \qquad (4.1)$$

where x is the offset distance between transmitter and receiver, v is the velocity of the medium above the reflector, and $t(0)$ is twice the two-way travel time along the vertical path MD (Yilmaz, 1987). We can rearrange this equation to solve for velocity, or we can observe that

---

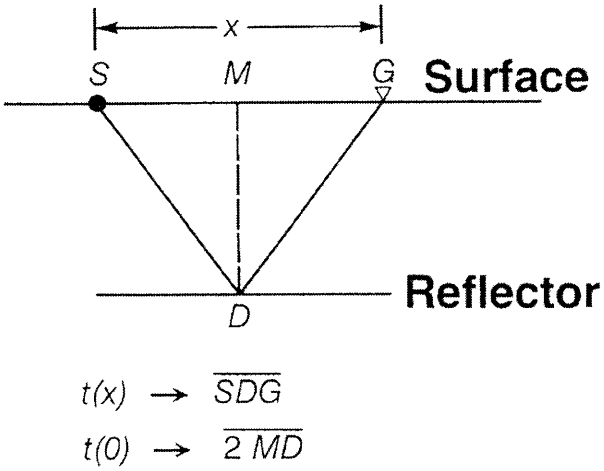
[3] Nyquist spacing is given as one quarter the wavelength of the host material, or:

$$n_x = \frac{c}{4f\sqrt{K}} = \frac{75}{f\sqrt{K}} \text{ (in meters)}$$

where f is the antenna frequency in MHz and K is the dielectric constant (relative permittivity) of the host material (Annan and Cosway, 1992).

this is simply the equation of a line with slope $\frac{1}{v^2}$. A plot of $t^2(x)$ versus x^2 yields a straight line, and the square root of the inverse slope is the velocity. Once a velocity is known, depth to a target may be determined given one way travel time.

Figure 4.1 – Normal Moveout Geometry
(Yilmaz, 1987)



CHAPTER 5: RESULTS

Figure 5.1 is a diagram of the survey site, 122 Beacon Street, Boston, MA. Over a period of one month, nine GPR surveys were conducted along two profiles in the rear of the building. The profiles are marked survey line 1 and survey line 2 in figure 5.1. (See also figures 2.6a-c for photographs.) Six surveys were fixed offset reflection profiles and three were common-midpoint velocity soundings. Relative success was achieved with mixed results. Only two of the nine surveys provide interpretable data. The table below provides basic details for each survey.

Table 5.1 – Survey Information

Data File	Survey Type	Antenna Frequency (MHz)	Antenna Separation (m)	Step Size (m)	Survey Length (m) / Survey Line
BCN1_1	reflection	450	0.25	0.05	6.30 / line 1
BCN1_2	reflection	450	0.25	0.05	6.25 / line 1 flush to outer wall
BCN1_3	CMP	450	-	0.05	3.20 / line 1
BCN2_1	reflection	450	0.25	0.05	6.65 / line 1
BCN2_2	reflection	450	0.25	0.05	5.75 / line 2
BCN2_3	reflection	225	0.50	0.05	5.40 / line 2
BCN3_1	reflection	225	0.50	0.05	5.40 / line 2
BCN3_2	CMP	225	-	0.05	2.60 / line 2
BCN3_3 *	CMP	225	-	0.05	2.30 / line 1

* Battery died midway through survey. Data is incomplete.

BCN1_1 (figure 5.2) was processed with a median spatial filter to remove large amounts of spurious noise. Automatic gain control with a maximum gain of 250 was also applied. Large vertical shifts appear in nearly two-thirds of the traces, rendering meaningful interpretation difficult. BCN1_2 (figure 5.3) exhibits the same data shifts as BCN1_1 to an even greater extent. The source of these bad traces is unknown. Instrument error is

suspected. A median temporal filter and low pass filter with a 500 MHz cutoff were applied to dampen the noise. The data were gained using AGC with a maximum gain of 250 to enhance possible reflectors. BCN1_3 (figures 5.4a, 5.4b) has the same data shifts as BCN1_1 and BCN1_2, except applying median and low pass filters along with spreading and exponential compensation enhances the target reflector. BCN1_3 was also processed using Seismic Unix, and the results were similar.

A possible source of data error in the first three surveys is the laptop computer used to collect the data. The second series of surveys used a different computer to collect data, and better results were achieved. BCN2_1 (figure 5.5) shows significantly improved resolution after processing with a median temporal filter and correction for true velocity based on a velocity analysis of BCN1_3. BCN2_2 shows similar improved resolution even though a median spatial filter was not as effective at removing noise compared to BCN2_1.

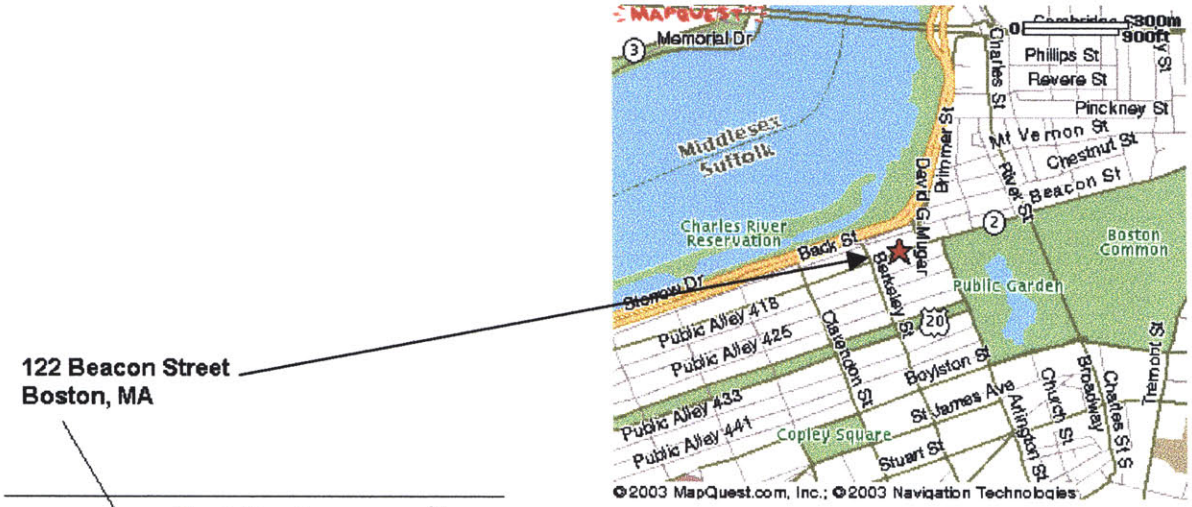
BCN2_3 was the first survey for which the 225 MHz antennas were used. Penetrating to greater depths, even without filtering BCN2_3 (figure 5.6) reveals more about the subsurface than any of the previous reflection profiles. Reflections appear at two-way travel times of 80-90 ns rather than 50-60 ns in the previous figures. Passing BCN2_3 through a median spatial filter eliminates most of the noise and the reflectors retain their original characteristics (figure 5.7).

The aim of collecting the third and final series of data was to gather more data with the 225 MHz antennas to better resolve more of the subsurface. BCN3_1 is the same survey as BCN2_3, except the data is slightly more noisy and more difficult to interpret. BCN3_2 (figure 5.8) is a CMP survey along line 2, a second attempt at gathering velocity data. This profile has not been filtered. BCN3_3 is a CMP survey along the same profile as the first

CMP survey, BCN1_3. However, the battery ran out of power in the middle of BCN3_1, rendering the data relatively incomplete. Figure 5.9 shows the incomplete results of BCN3_3, but even this partial rendering reveals reflectors not seen in BCN1_3.

Figure 5.1 – Diagram of the survey site and street map (http://www.mapquest.com)

122 Beacon Street, Boston MA



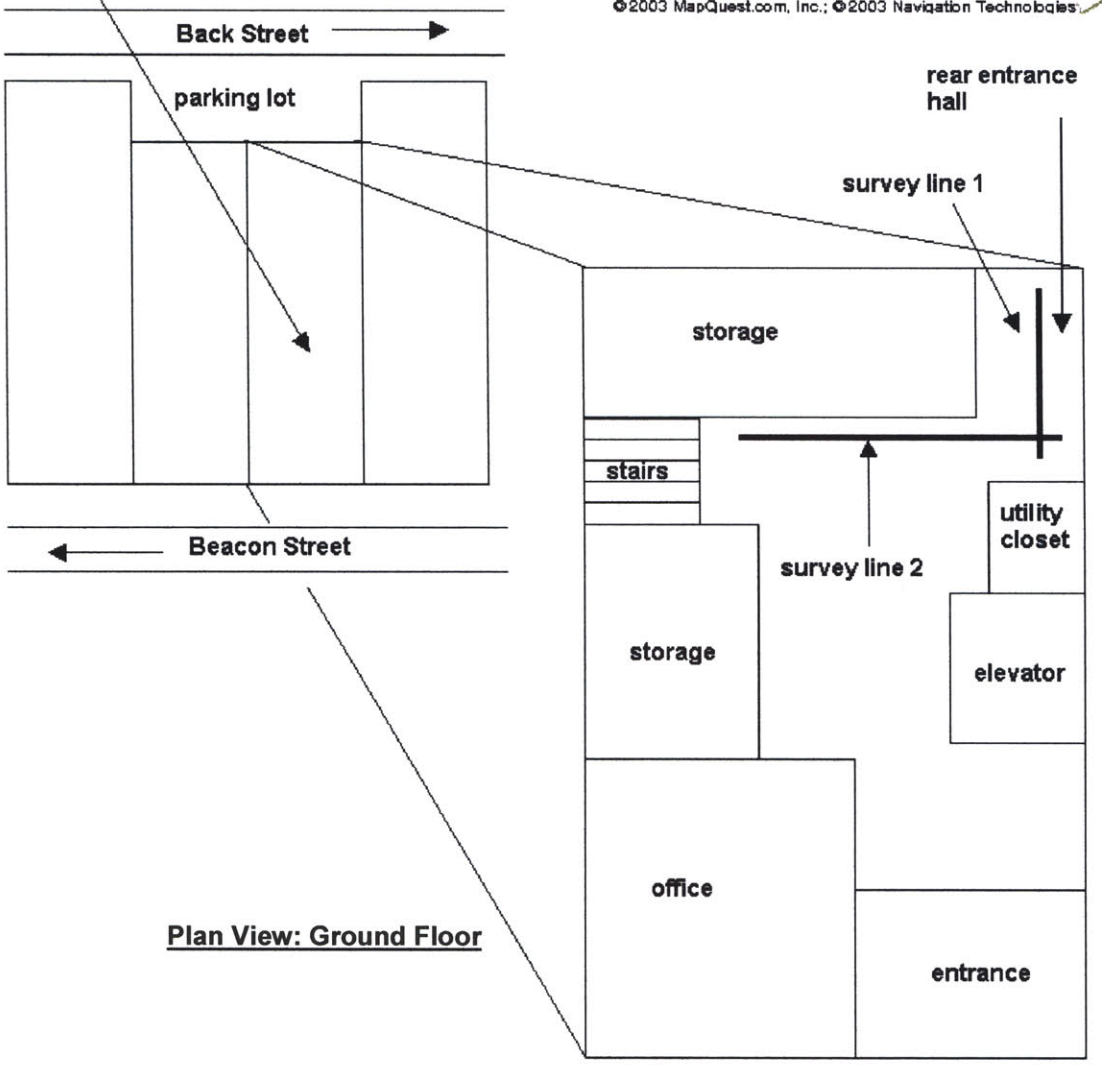


Figure 5.2 – BCN1_1

This data set was processed with a median spatial filter and gained with automatic gain control, gain maximum = 250. Large data shifts (red arrows) prohibit meaningful interpretation.

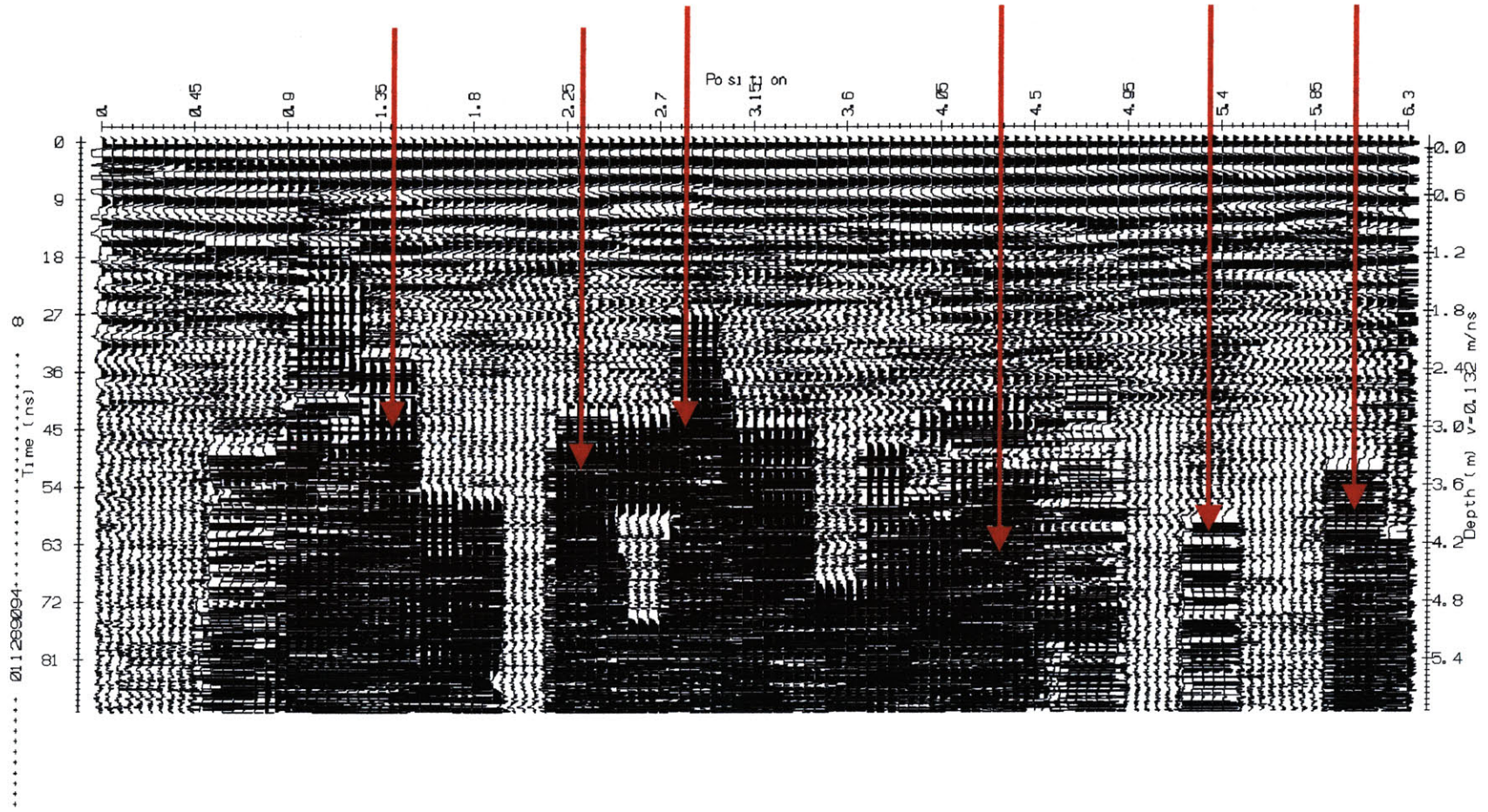


Figure 5.3 – BCN1_2

The same data shifts appear as in BCN1_1 except this data set was processed with a median temporal filter and a low pass filter, neither of which removed much of the noise. Again, it is difficult to derive a clear interpretation from this record. Red arrows mark a few bad traces.

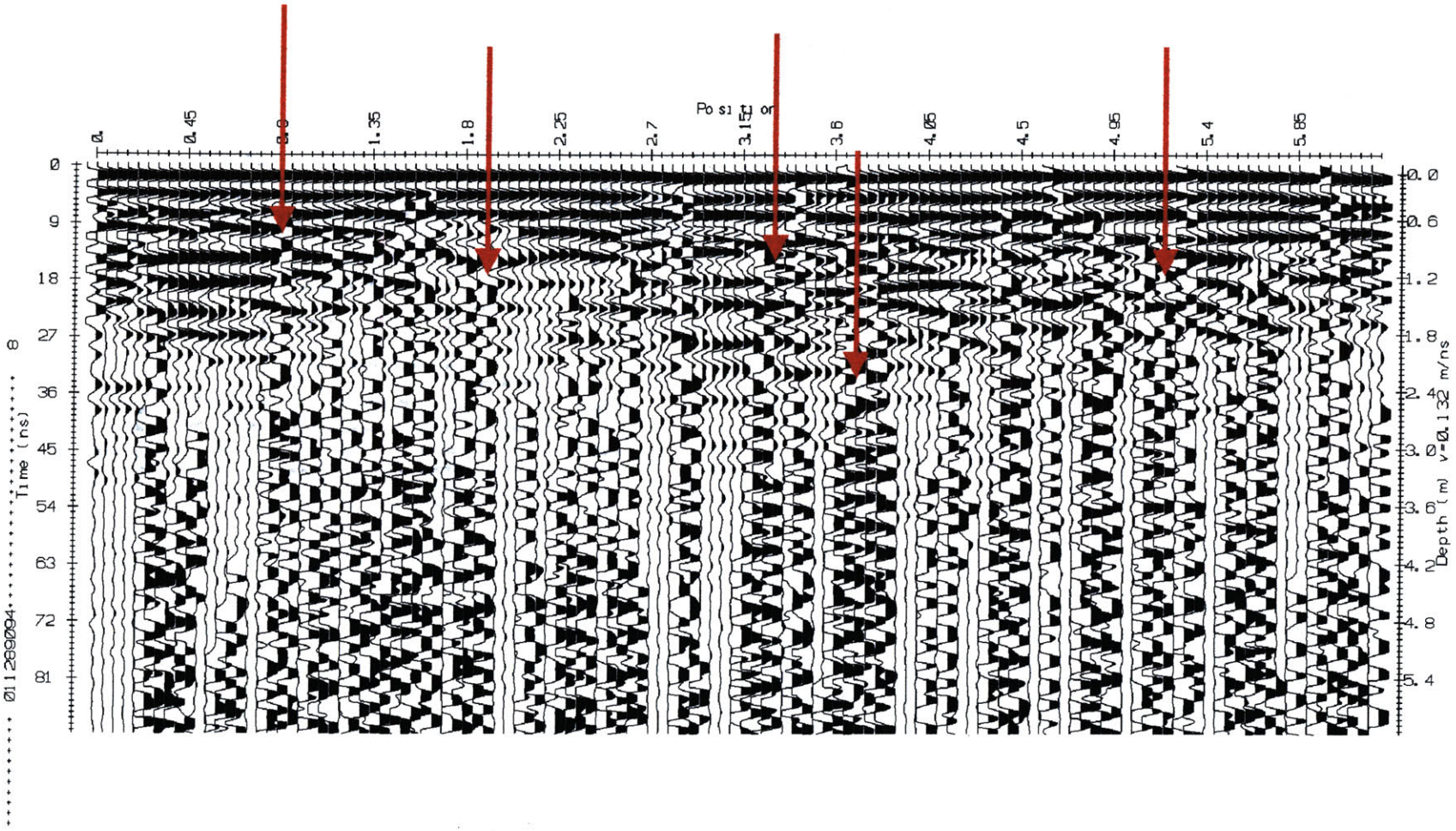
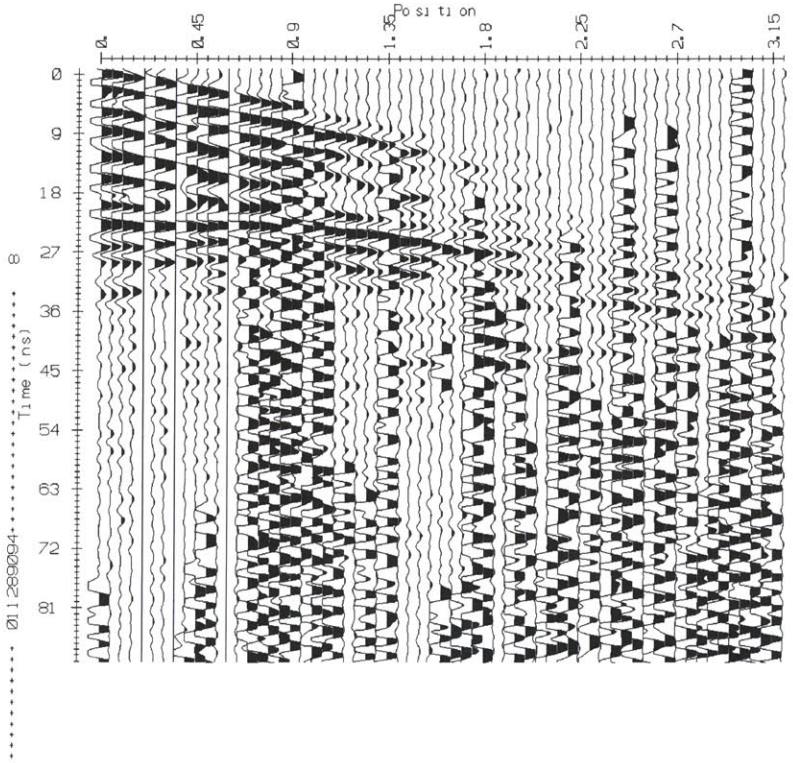


Figure 5.4 – BCN1_3

5.4a – This plot was used to calculate NMO velocity based on the reflector indicated by the arrow.



5.4b – A color rendering of the plot in 5.4a.

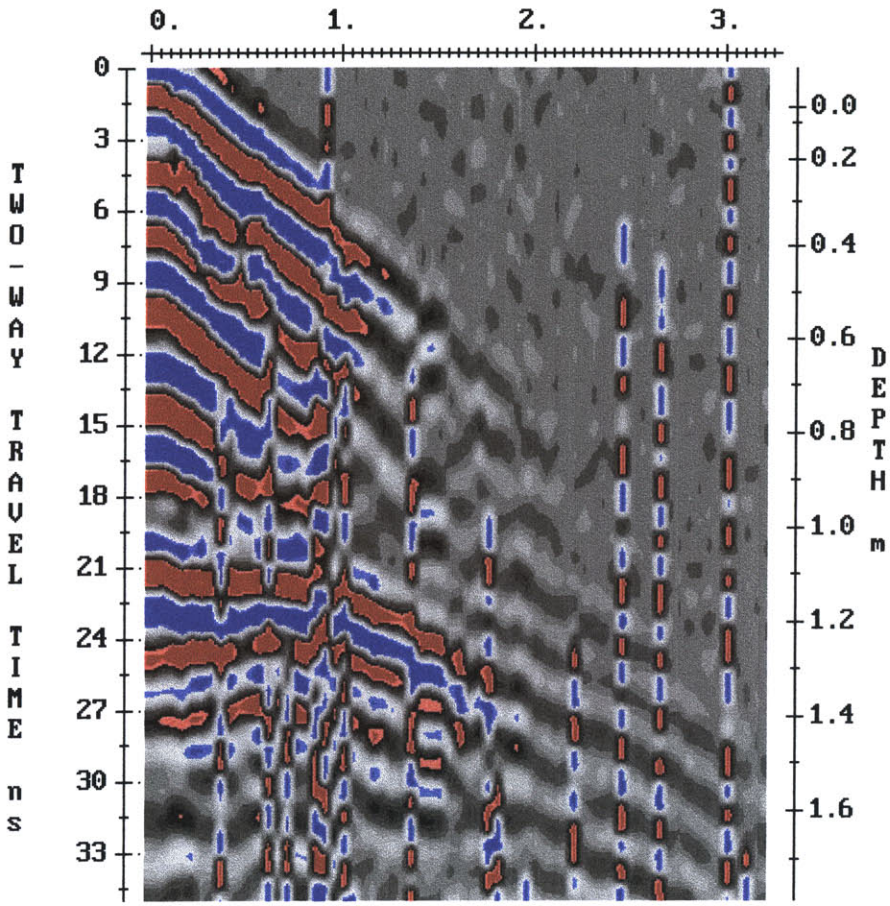


Figure 5.5 – BCN2_1

Resolution is significantly improved by processing the data with a median filter and correcting for the NMO velocity.

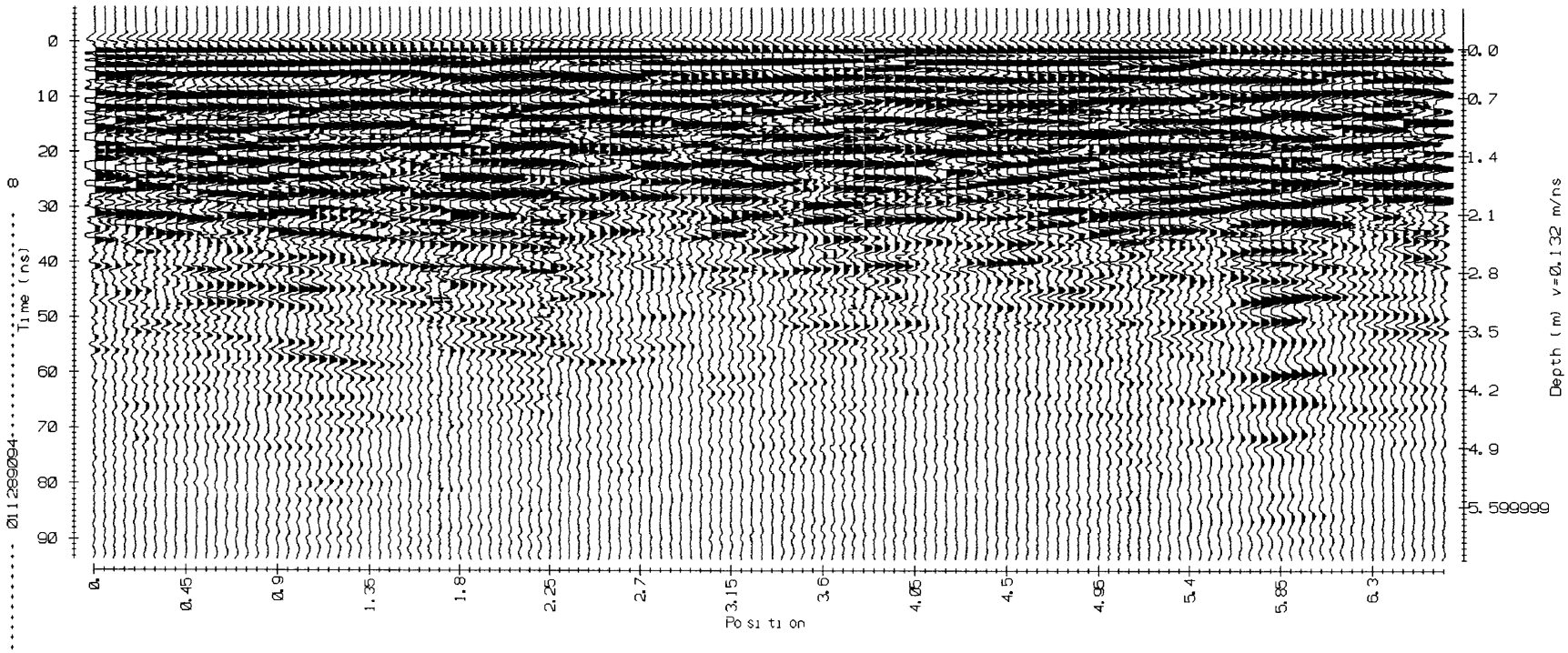


Figure 5.6 – BCN2_3

BCN2_3 reveals more subsurface details that previous profiles. Deeper reflectors are visible.

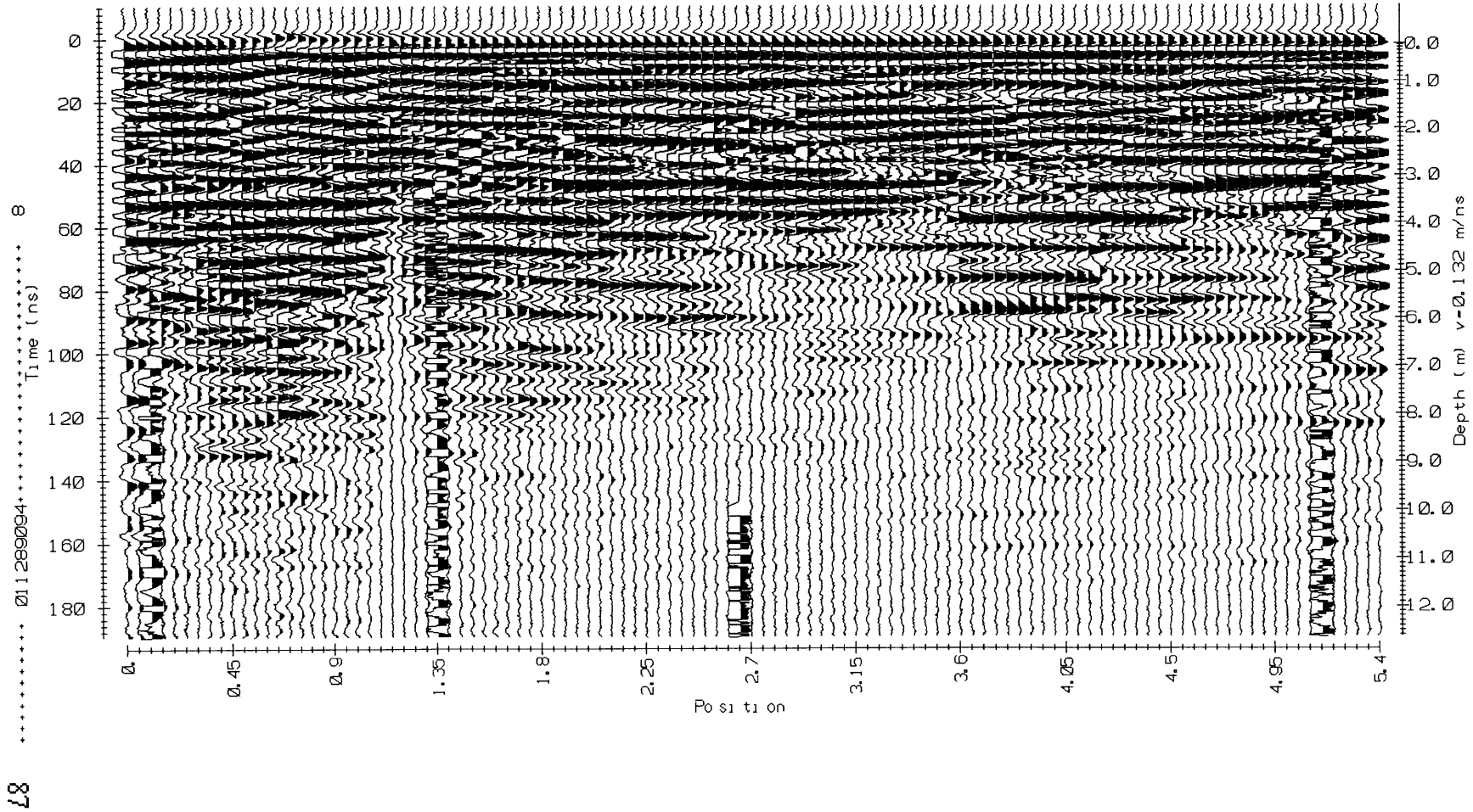


Figure 5.7 – BCN2_3

The same data in figure 5.6 processed with a median filter to remove noise. The reflectors retain their original characteristics.

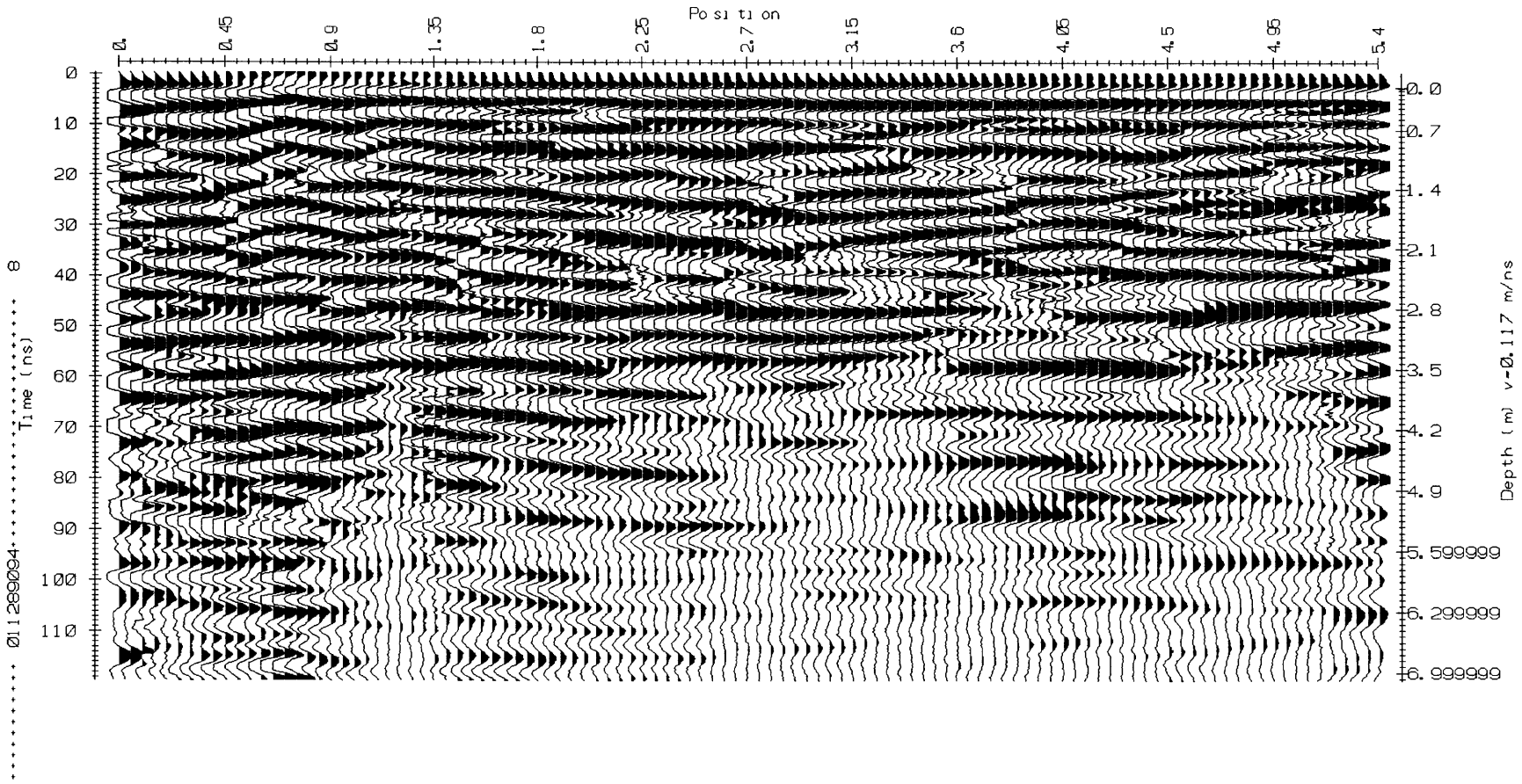


Figure 5.8 – BCN3_2

BCN3_2 is a CMP survey along line 2 (not processed).

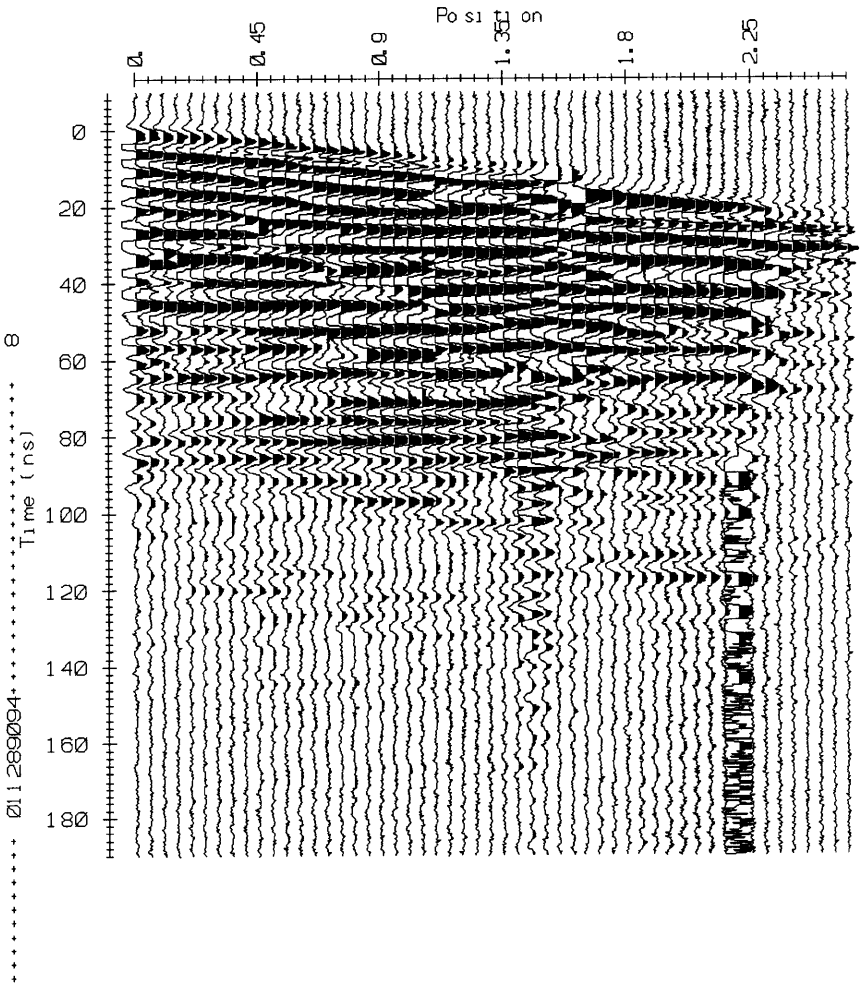
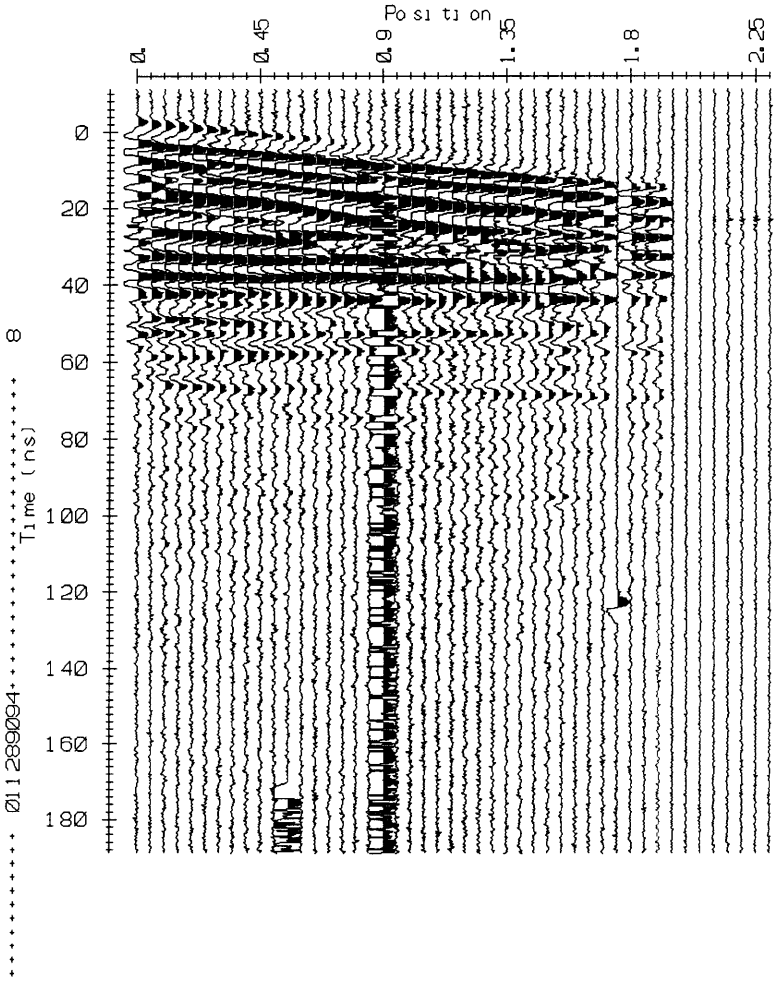


Figure 5.9 – BCN3_3

This is a CMP survey along line 1. (Data are relatively error free but incomplete due to failure of the power source.)



CHAPTER 6: ANALYSIS AND CONCLUSIONS

The interpretation of geophysical data often relies heavily on *a priori* information. In this study, the radar data is not convincingly definitive, so corroborative and deductive evidence is especially important. There are three sources of information with which we can establish a context for interpreting the radar data: 1) observations from recent excavations, 2) groundwater elevations measured in monitoring wells, and 3) the results of the resistivity survey. Information about the soil, stratigraphy, and underground infrastructure must also be considered. Table 6.1 provides a summary of groundwater data and possible locations of the water table.

Table 6.1 – A Summary of Available Groundwater Data

Source (all data collected in Spring 2003)		Water Table Elevation (referenced to the BCB)	Elevation of Ground Surface (referenced to the BCB)	Depth to the Water Table from the Ground Surface	
				meters	feet
Excavation Observations		6.5 (at 146 Beacon St.)	12	1.65	5.5
Monitoring Wells		Average value = 3.15	12-13	2.8	9.4
Resistivity Survey		9-10	12-13	1	3
GPR	interpretation 1	6.5	12	1.65	5.5
	interpretation 2	2	12	3	10

Data from borehole logs accumulated by the Committee on Subsoils of Boston suggests that the depth of the fill may only be between 3-6 feet for this block of Beacon Street (figure 2.5a). The data further suggest that a silty sand mixture, perhaps interbedded with thin layers of peat or fine silt and also containing shells, is the dominant soil type to a depth of approximately 30 feet. A layer of sand and gravel between 15-20 feet thick underlies the silty sand. For the preservation of wood piles, this is good: silty organic layers of soil retain water better than sandy soils or fill and thus the effect of water table fluctuations might be

somewhat mitigated or at least delayed. For GPR, however, the drier the soil the better the data. The higher the water content, the more signal loss and the shallower the signal penetration.

With a 450 MHz antenna, it is reasonable to expect 1-1½ meters of resolvable data, while a 225 MHz antenna might be expected to penetrate a full 2-3 meters. If the borehole data is accurate, it is possible that the soil conditions somewhat obscure the radar signal. The water table should still be a strong reflector, but both the granite pile caps and the piles themselves leave a much smaller electrical footprint. The prevailing soil conditions could make them much harder to detect.

The well data and the observations from recent excavations present an interesting contradiction. Water levels in the wells near 122 Beacon Street have been consistently low since the winter of 2002 (figures 2.4a, 2.4b). Most, on average, have water levels no greater than El. 3.0-4.0 (between 8-9 feet below ground surface). However, excavations, which occurred within a month of this study and within 1-2 months of well readings taken in the spring of 2003, suggest that water levels in the close vicinity of 122 Beacon Street are at El. 6.5, only 5.5 feet below ground level.

There are four wells on Back Street between 100 and 134 Beacon Street, spanning an area which roughly corresponds to the block of Beacon Street bounded by Embankment Road to the east and Berkeley Street to the west. The distance from the wells to the residences on the water side of Beacon Street is not more than 20-30 meters along a straight line. The most recent readings from these wells, taken in the spring of this 2003, measure groundwater levels at El. 2.46, 2.60, 2.62, and 3.84 respectively moving westerly along Back Street. The next two wells along an east-west profile starting at the next block of Beacon Street show

groundwater elevation decreasing to El 2.16 and El. 1.15. It is important to note that these wells are on the Storrow Drive (north) side of the sea wall (figure 3.1b) which runs parallel to Back Street (figure 1.12). It is therefore possible that there are two distinct hydrologic regimes on either side of the wall. The seawall might act as a barrier which maintains higher water levels to the south while wells to the north are drawn down by the pumping which is known to occur at relatively consistent, high rates in the vicinity of the Storrow Drive underpass.

The sea wall is an indicator that buried infrastructure plays an unknown role. The block of Beacon Street under consideration in this study is bounded on the south by the remnants of the Mill Dam, within which sits the West Side Interceptor. To the north beneath Storrow Drive is the Boston Marginal Conduit, and to the east the Storrow Drive underpass is a known groundwater sink. Certainly, there also are other sewers and drains whose possible hydrological impact we have neglected. Finally, the residences on the water side of Beacon Street are, like the rest of the Back Bay, surrounded almost entirely by impervious surfaces. Groundwater in this area is not being recharged by infiltration.

The one area near this block of Beacon Street which might appreciably benefit from limited infiltration and recharge is the grassy strip between Back Street and Storrow Drive where the resistivity survey was performed. The results of the resistivity survey indicate that the water table is approximately 1 meter (3 feet) below the ground surface, which is approximately El. 9 - El. 10. The resistivity data agree more with water level estimates from the recent excavation at 146 Beacon Street and support the notion that this grassy area might be an infiltration zone, however limited by its proximity to the sea wall.

There are at least two possible interpretations of the radar data. From the three CMP velocity soundings (BCN1_3, BCN3_2, BCN3_3), it is possible to calculate a normal moveout velocity for the strongest reflector in BCN1_3 (indicated by the arrow in figure 6.1) based on the method described in chapter 4. Solving the NMO equation yields a velocity of 0.132 m/ns, which is a reasonable value for relatively dry sand where ε_r = 3-6 (Reynolds, 1997). Using this velocity and considering the evidence from recent excavations, which puts the water table at El. 6.5, or 5.5 feet below ground surface, it is possible that this reflector represents the water table. We can also determine where in time such a reflector should appear in the radargram.

By a very simple calculation, since:

distance (d) = velocity (v) * time (t) (6.1),

then:

$$t = \frac{d}{v} = \frac{1.68m}{0.132m/ns} = 12.7ns \quad \text{one-way travel time} \qquad (6.2)$$

$$= 25.4\ ns \quad \text{two-way travel time.}$$

Examining the best radargram, BCN2_3, we can try to match a reflector to this travel time. Figure 6.2 shows the first 60 ns (two-way travel time) of BCN2_3, with two discontinuous reflectors around 25 ns highlighted. We might expect the water table to be more laterally continuous than the reflectors in this time interval. Although the water table may vary with surface relief, our survey lines were flat and short compared to the scale on which any topographical relief may occur. The water table should also be a flat reflector, contrary to those that appear around 25 ns.

Furthermore, if either of these reflectors represented the depth to the water table, we would expect much less signal penetration beneath them due to the signal attenuation caused

by water. However, the data from BCN2_3, BCN3_2, and even BCN3_3 indicate that there are relatively strong reflectors at depth. It is perhaps equally reasonable to conclude that the reflector at 25 ns in BCN1_3 is the fill/sandy silt interface we might expect to encounter at depths of 3-6 feet. However, in light of the evidence from recent excavations, we can make a strong argument that this reflector represents the water table.

Looking again at a plot of the data from BCN2_3 (figure 6.3), the strongest, flattest reflector is at a depth of approximately 42 ns. If we assume that the reflector identified in BCN1_3 is the fill/sandy silt interface, then the velocity scale in the plot is inaccurate for two-way travel times greater than approximately 25 ns since the velocity in sandy/silt will be slightly different from the velocity in the fill. A velocity of .055-.095 m/ns is a reasonable estimate for partially wet sandy silt (Reynolds, 1997). Depth to this reflector is then approximately 10 feet below ground level, or between El. 2. This interpretation agrees more with the well data than the resistivity survey or the excavation evidence.

There are two possible locations for the water table based on two modes of interpretation. Based on the CMP survey the water table may be estimated at depths of 5.5 feet below ground surface (El. 6.5). Based on a straight interpretation of the radargram, the water table may be as deep as approximately 10 feet below the ground surface (El. 2). In either case, the remaining objective is to locate the wood piles. GPR works because contrasting electrical properties exist at boundaries between subsurface media. If the contrast is too weak (i.e. dielectric constants are too similar), then a target goes unresolved. The relative dielectric constant for a mixture of moist sand or silt may range from 10-30, depending on the relative amounts of sand and silt, and it is the same for wet wood[1]. Even if

[1] Reynolds, 1997 and Dielectric Constants of Common Industrial Materials: http://www.ab.com/catalogs/c114/4capprox/40058.pdf

the wood has already begun to decompose into a peat moss or other organic intermediate, the contrast is not likely to be sufficient. The relative dielectric constant for granite is between 5-8. At the lower end of the sand/silt range there is still very little contrast.

However, if the sand/silt layer is dominated by a relatively moist sandy silt, there might be sufficient contrast to produce an event or anomaly in the radargram. The piles are most likely cut off at depths varying between El. 5.0 - El. 7.0 (5-7 feet or 1.5-2 meters) below ground level). Conservatively, granite caps 1-2 feet thick could vary in depth between El. 6.0 - El.8.0 (4-6 feet or 1-2 meters below ground level). If the caps are at nearly the same elevation (depending upon how the structure has settled, this is not necessarily a true assumption), they might be expected to approximate a planar reflector. Some refractive behavior might also be expected depending on how tightly spaced the caps are.

A plot of BCN2_3 (figure 6.4) highlights several events in the radargram which may represent the granite caps. If we take the water level as determined by the CMP survey (BCN1_3), this line of reasoning suggests that the piles are just submerged at or slightly exposed below the water table. If the water table is as low as the well readings suggest, despite contradictory evidence from recent excavations and the resistivity survey, then it is likely that the tops of the piles are exposed. However, based on evidence from recent excavations and considering the resistivity data, the former scenario is the more likely conclusion. The piles at 122 Beacon Street may be slightly exposed at or above the water table, depending on the elevation at which they were cut off. Even if the piles at 122 Beacon Street are slightly exposed above the water table, soil conditions are favorable for moisture retention, limiting the effects of exposure and slowing decay.

Figure 6.1 – The strongest reflector in BCN1_3.

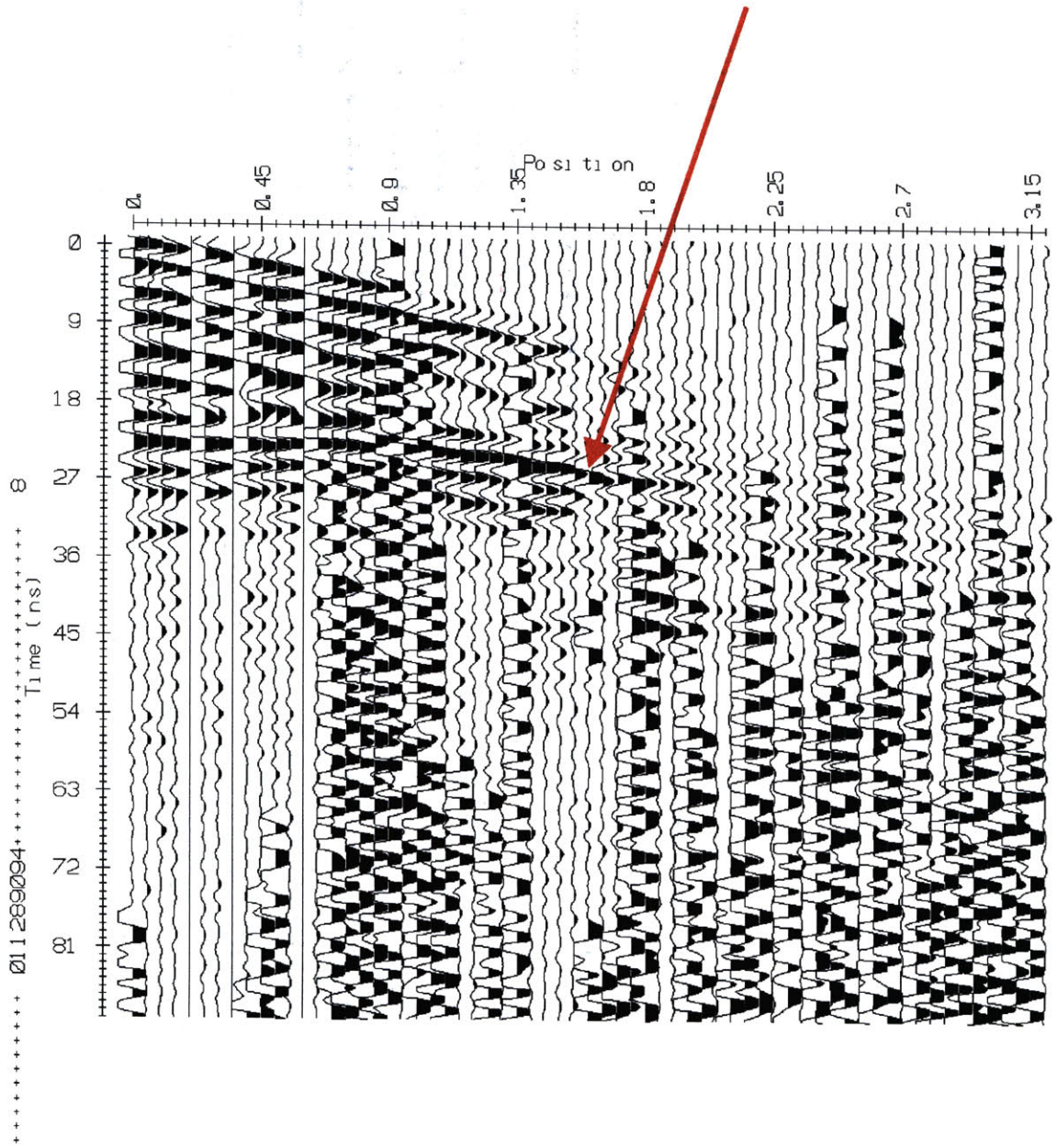


Figure 6.2 – BCN2_3

The first 60 ns of BCN2_3 with reflectors near 25 ns highlighted.

Figure 6.3 – BCN2_3

The strongest, flattest reflector in BCN2_3 in a time window showing approximately 120 ns.

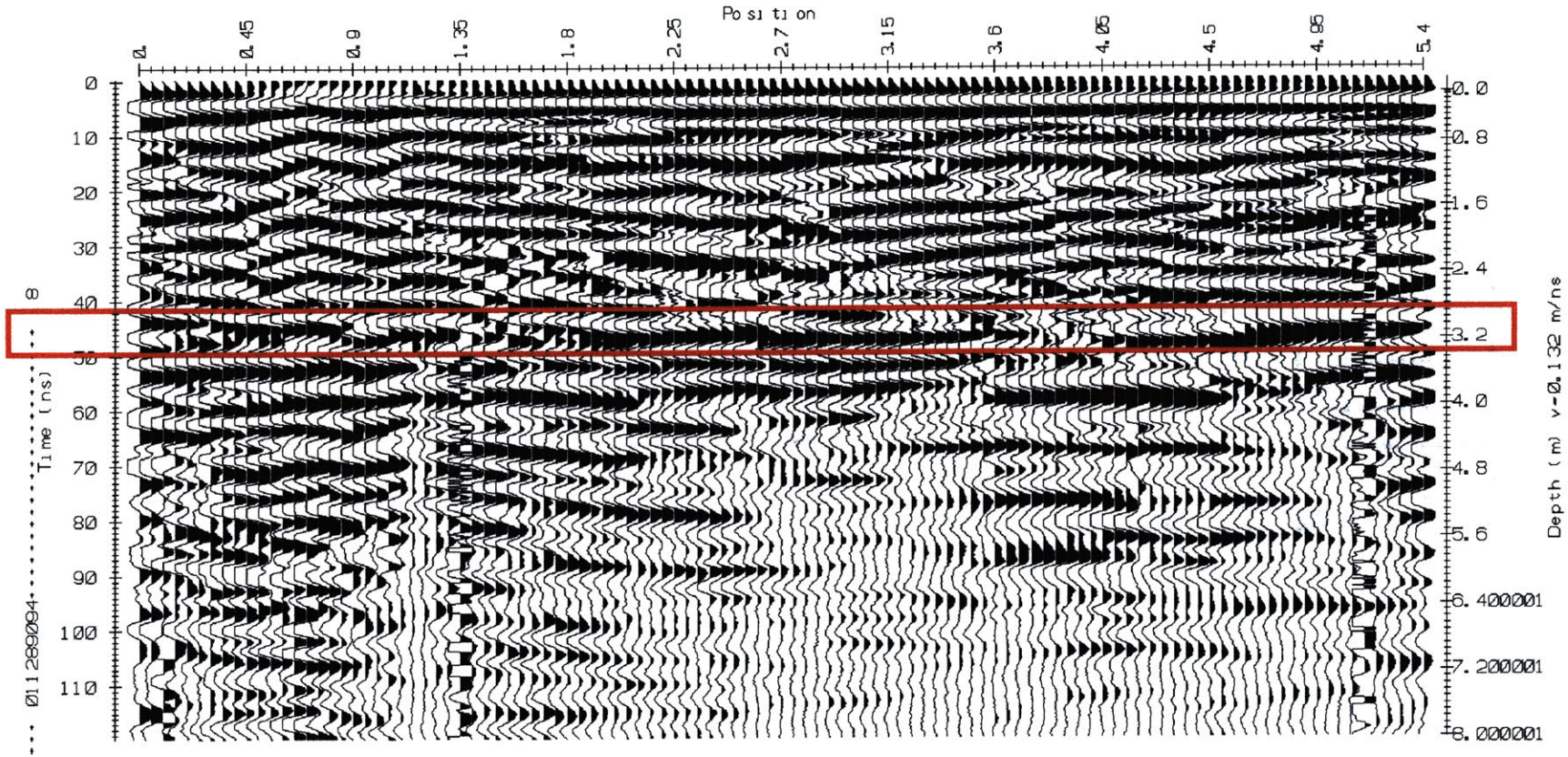


Figure 6.4 – BCN2_3

Events, or 'anomalies,' in the radargram.

CHAPTER 7 – RECOMMENDATIONS FOR FUTURE RESEARCH

The problem of wood pile deterioration has important social, economical, and political consequences. There are several basic ways to characterize this type of problem. One approach involves building hydrological models to better understand the groundwater system. Another approach is to use geographic information systems to quantify the effects of land use or to monitor seasonal changes. Certainly, residents of the Back Bay could continue, lot by lot, to dig test pits. However, there is little doubt that a refinement of sensing technologies, such as ground penetrating radar, is a worthwhile endeavor.

Two potential areas of further study include: 1) the use of GPR to monitor groundwater levels in areas where there is little or no well coverage, and similarly the use of GPR to increase the frequency and range of current well monitoring activities; and 2) the development of a tomographic approach to GPR. In this study, GPR was used to generate simple two-dimensional reflection profiles, like an X-ray exam. However, survey techniques could be modified to image the subsurface in the same way CAT (computed axial tomography) scans are used to create an image in three dimensions. This type of radar tomography would involve generating a series of velocity 'slices' and then arranging these slices to provide a three-dimensional profile of velocity with depth. This method might enhance our ability to visualize subsurface velocity contrasts and pick out anomalies that occur at fixed intervals, such as wood piles or the water table.

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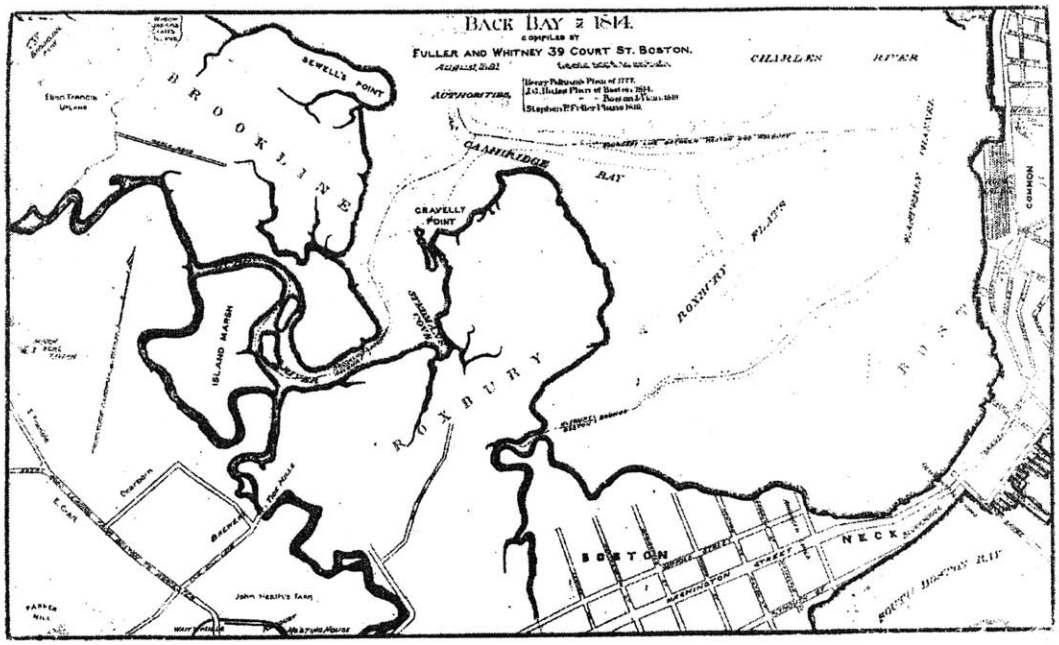
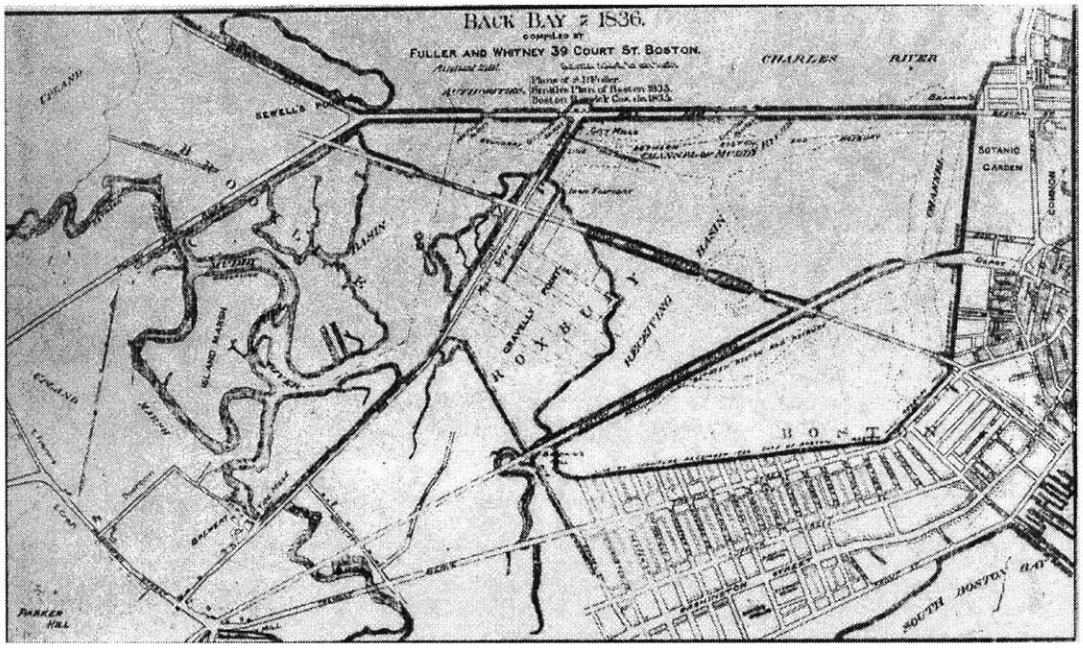
Appendix I – An Illustrative Chronology of Filling in the Back Bay

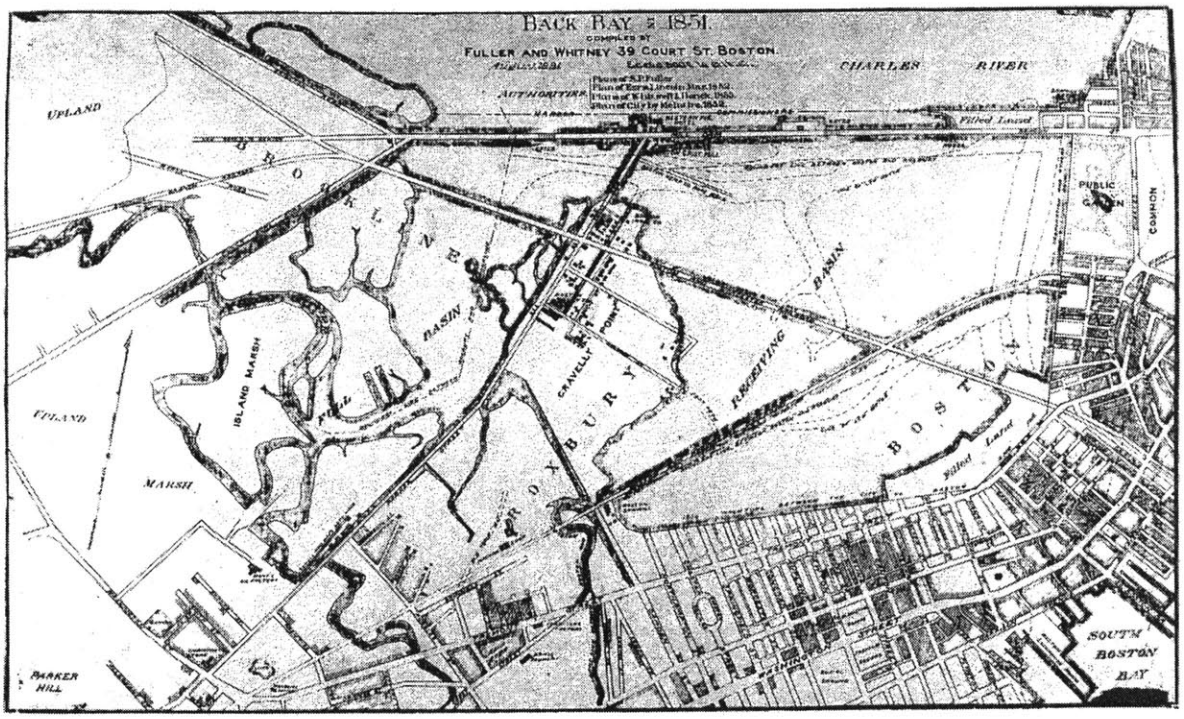
I.1 – Back Bay, 1814
(Aldrich, 1970)



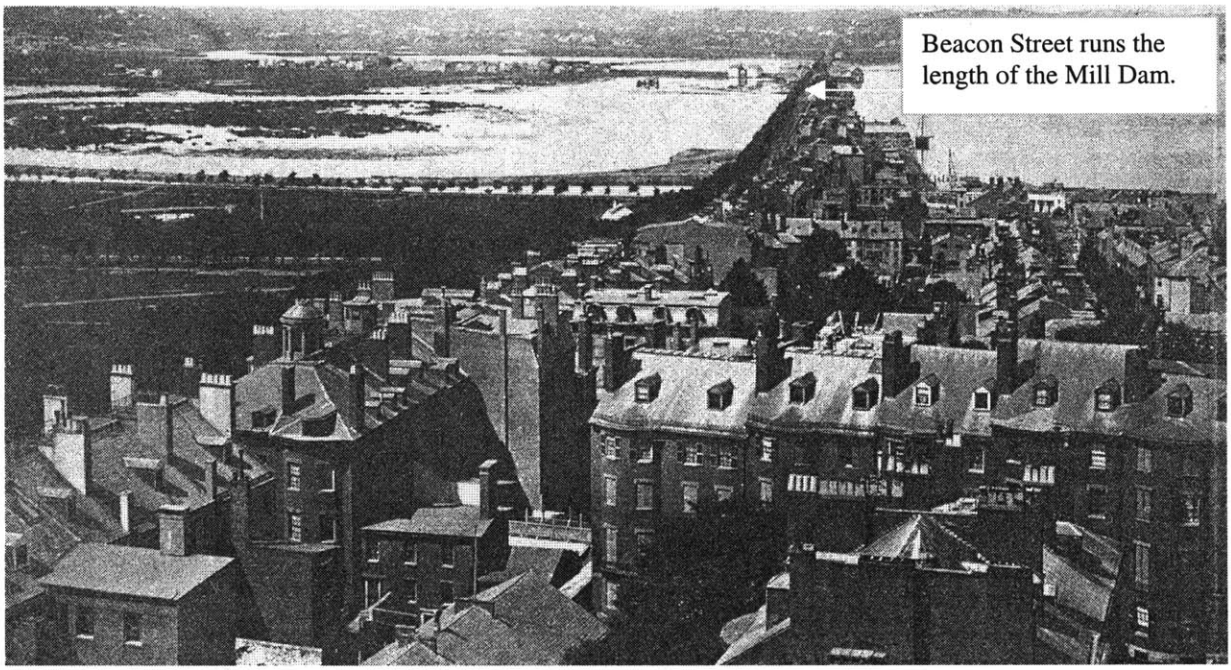
I.2 – Back Bay, 1836
(Aldrich, 1970)



I.3 – Back Bay, 1851
(Aldrich, 1970)

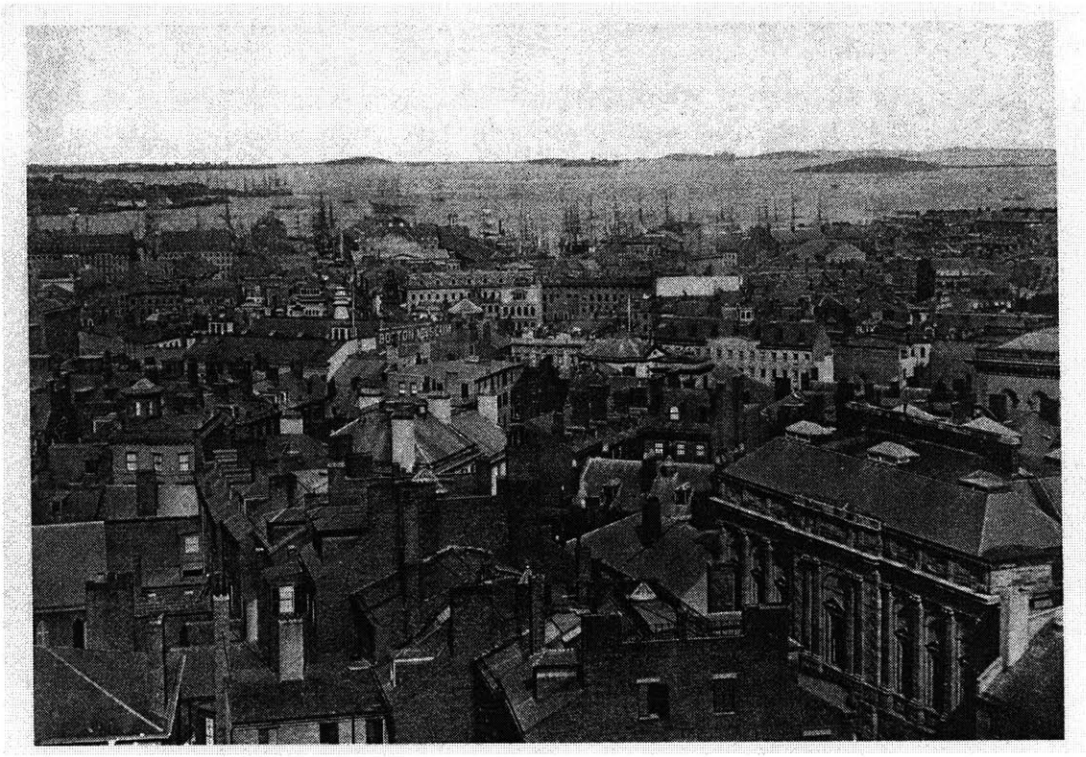


I.4 – A view of the Back Bay looking west from atop the State House in the late 1850s.
(Whitehall, 1963)

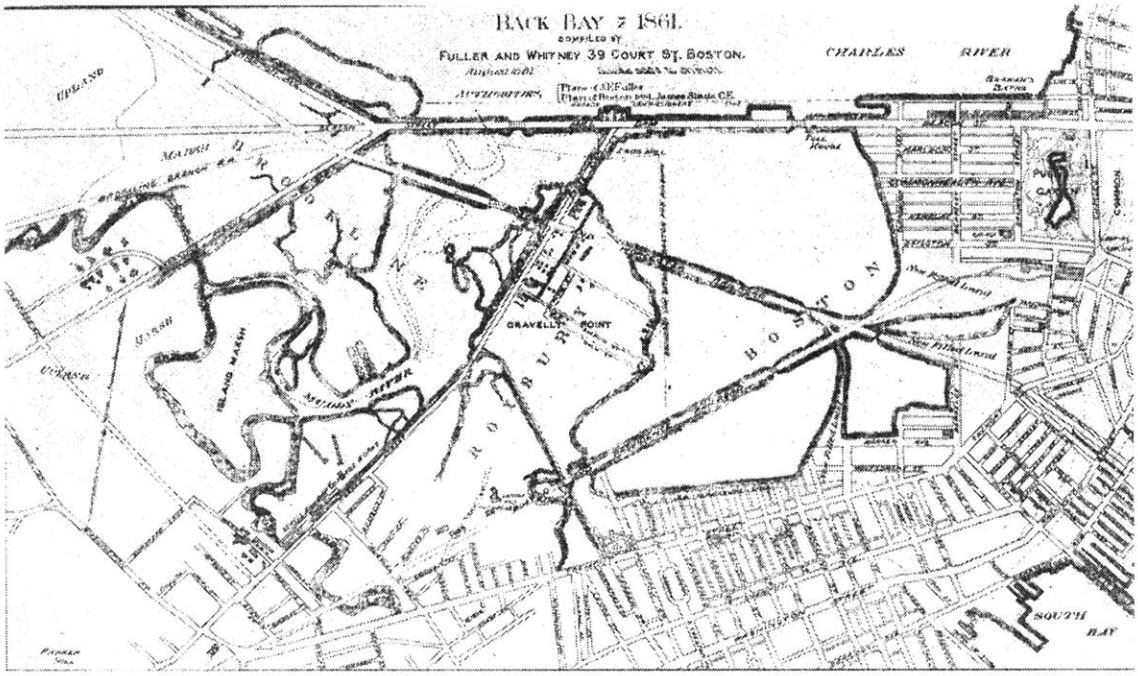


I.5 – A view of the Back Bay looking east from atop the State House in the late 1850s. (Whitehill, 1963)

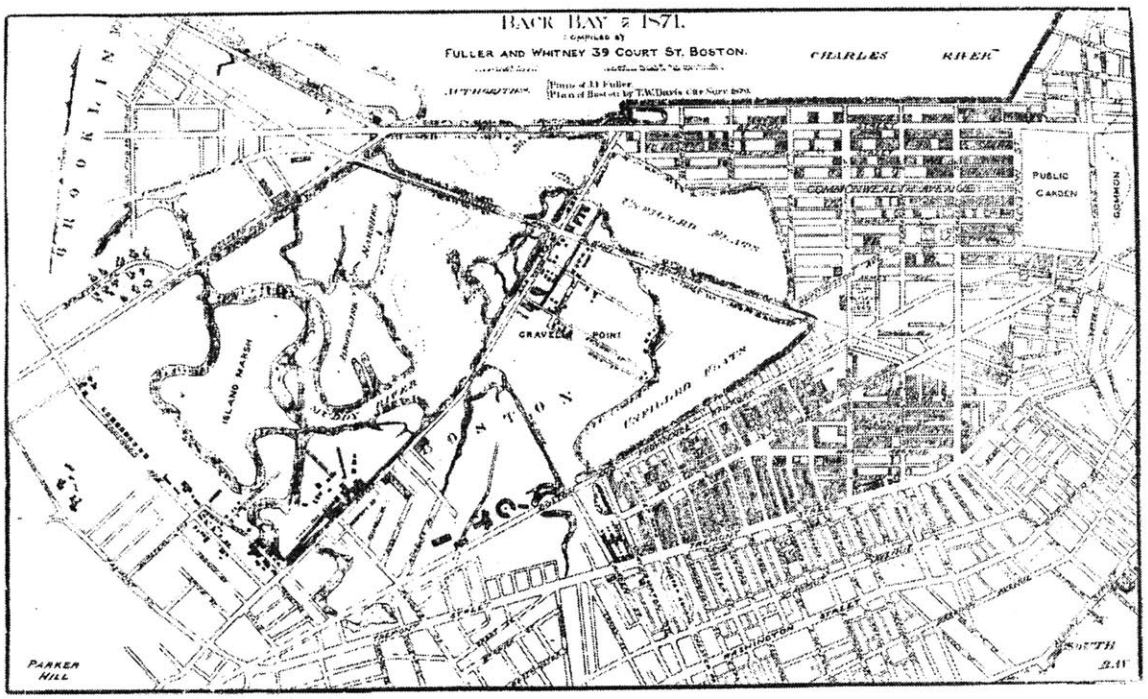
I.6 – A view of the Back Bay looking south from atop the State House in the late 1850s. (Whitehill, 1963)



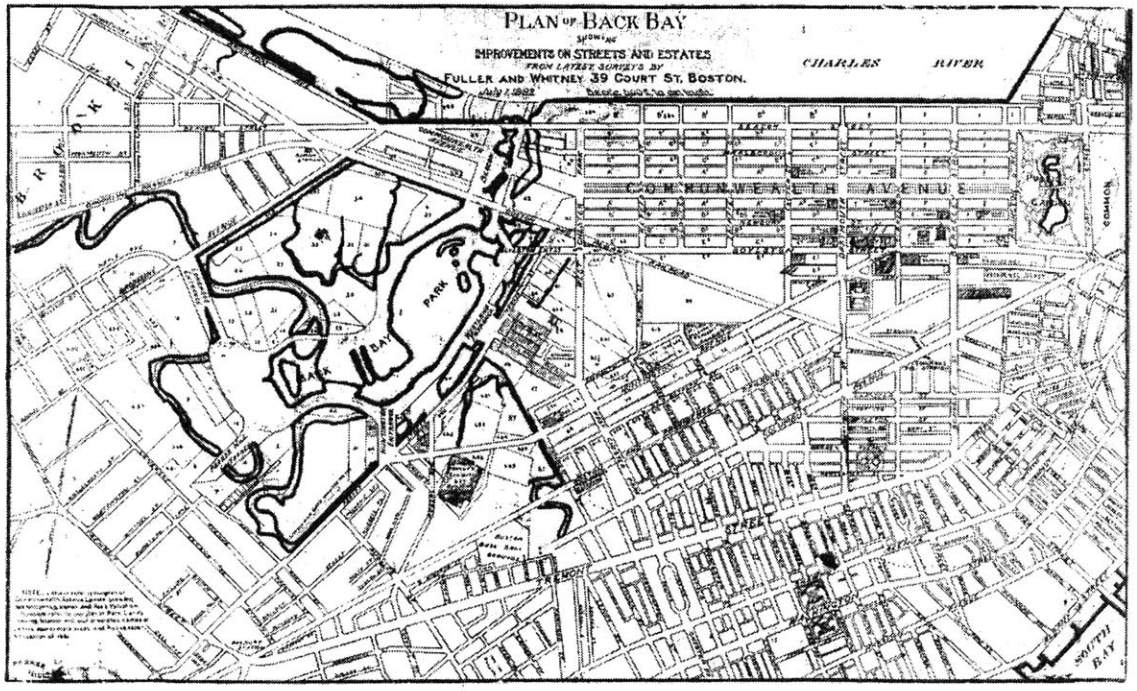
I.7 – Back Bay, 1861
(Aldrich, 1970)



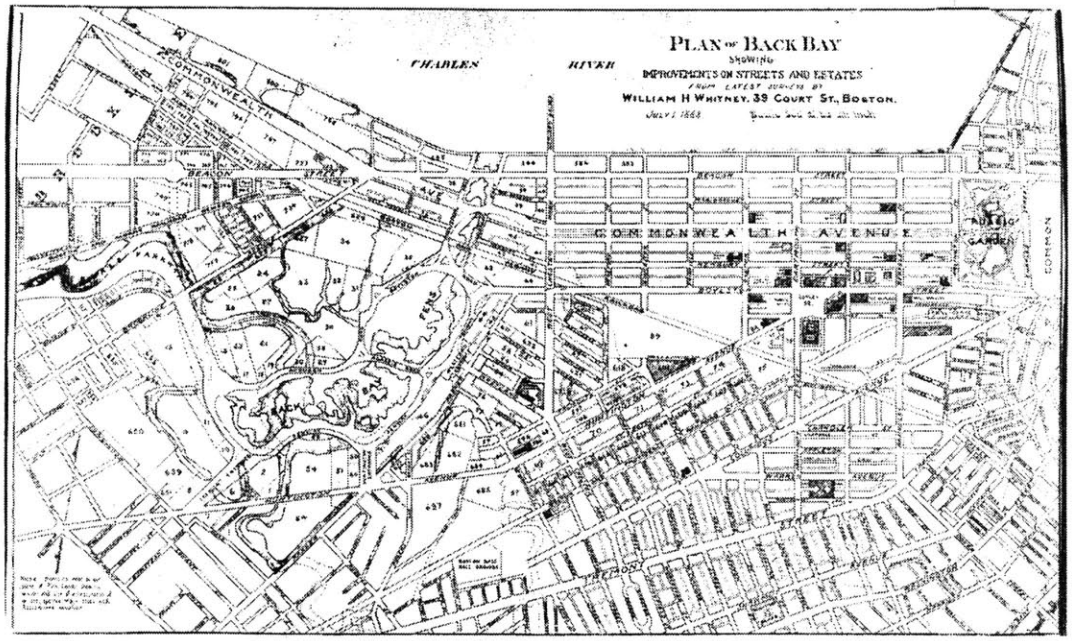
I.8 – Back Bay, 1871
(Aldrich, 1970)



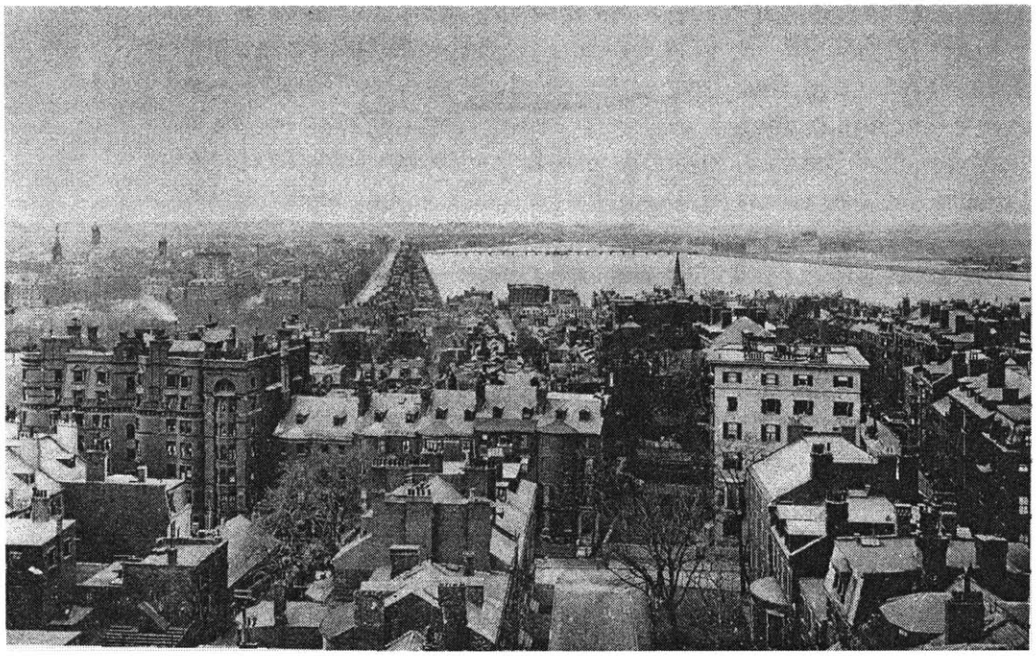
I.9 – Back Bay, 1882
(Aldrich, 1970)



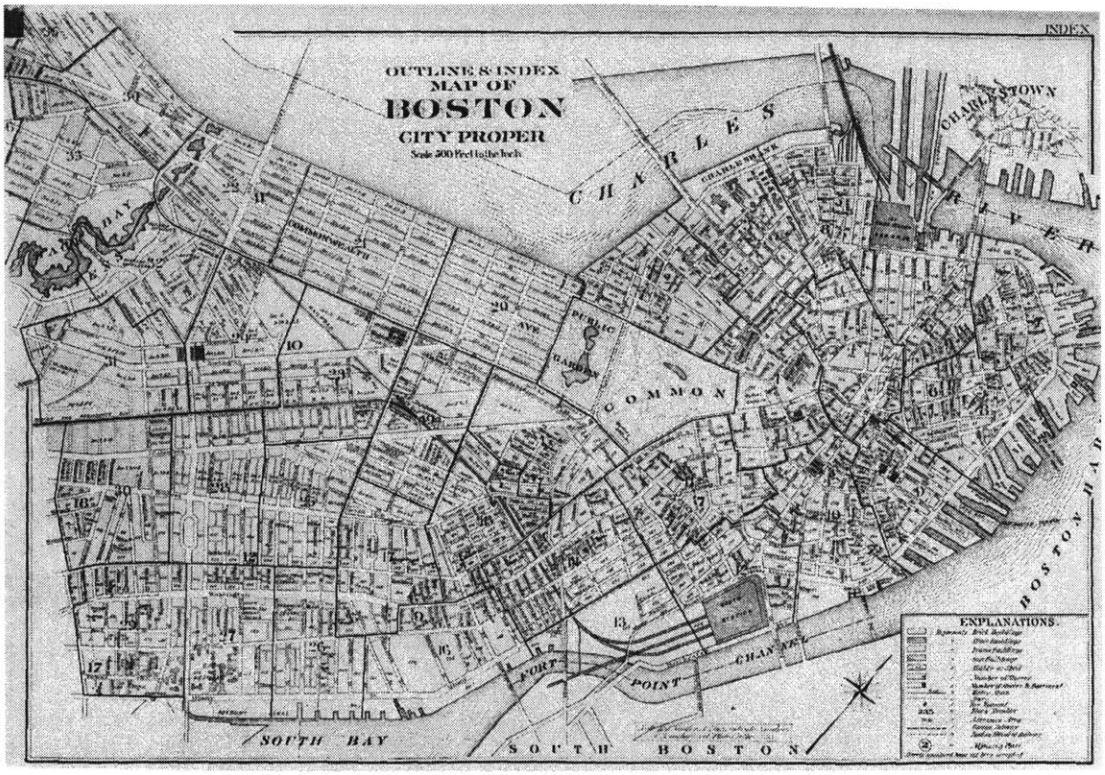
I.10 – Back Bay, 1888
(Aldrich, 1970)



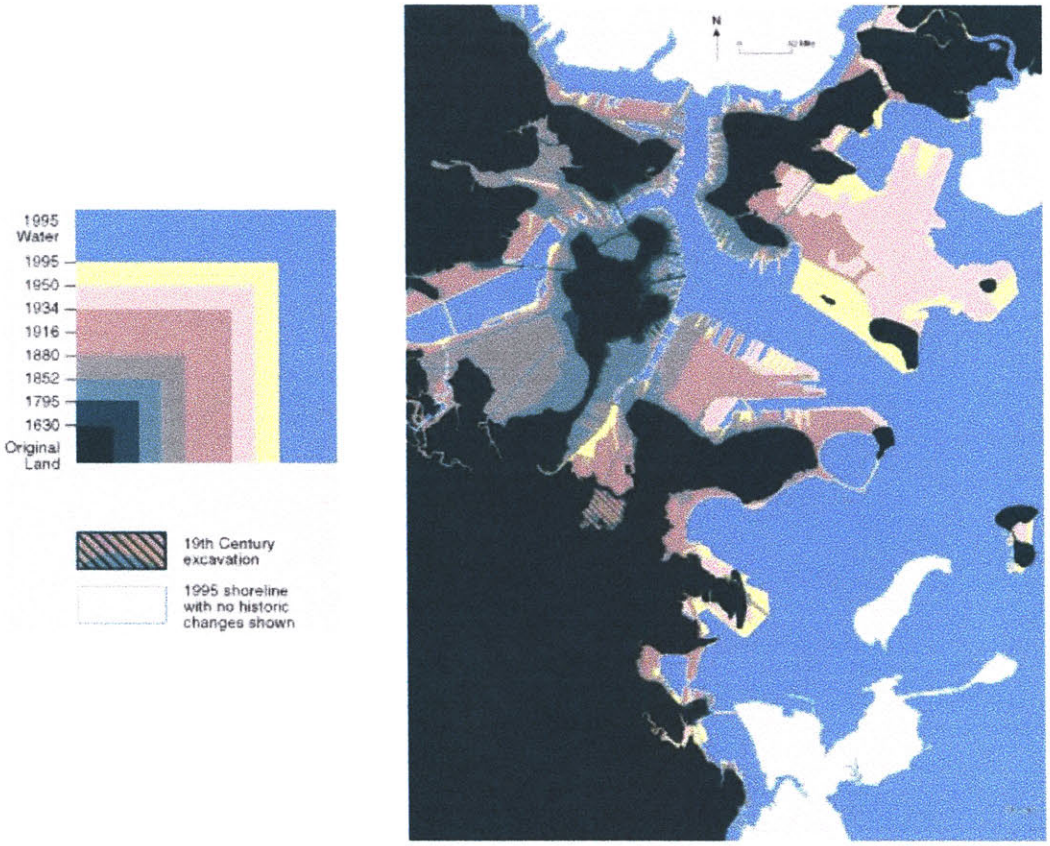
I.11 – A view of the Back Bay looking west from atop the State House in 1900. (Whitehill, 1963)



I.12 – The Back Bay is almost completely filled: A map of the city of Boston in 1902. (Bergen, 1990)



I.13 – The progression of filling:
(http://www.mappingboston.org)



I.14 – A contemporary view of the Back Bay, looking east-northeast.
(http://www.skypic.com/boston/5-6090.jpg)

APPENDIX II – STRATIGRAPHY OF THE BACK BAY

Bedrock in the Boston Basin belongs to the Boston Group, which includes two formations. The lower formation is the Roxbury Conglomerate, and the upper formation is the Cambridge Slate, also frequently referred to as the Cambridge Argillite. The Cambridge Slate, estimated to be 2000-4000 feet thick, underlies most of the Back Bay and the Boston Peninsula. Strata overlying this formation in the Back Bay include, from oldest to youngest: glacial till, marine clay (the Boston blue clay), sand and gravel outwash, organic silts and peats, and man-made fill (figure II.1) (Aldrich, 1970).

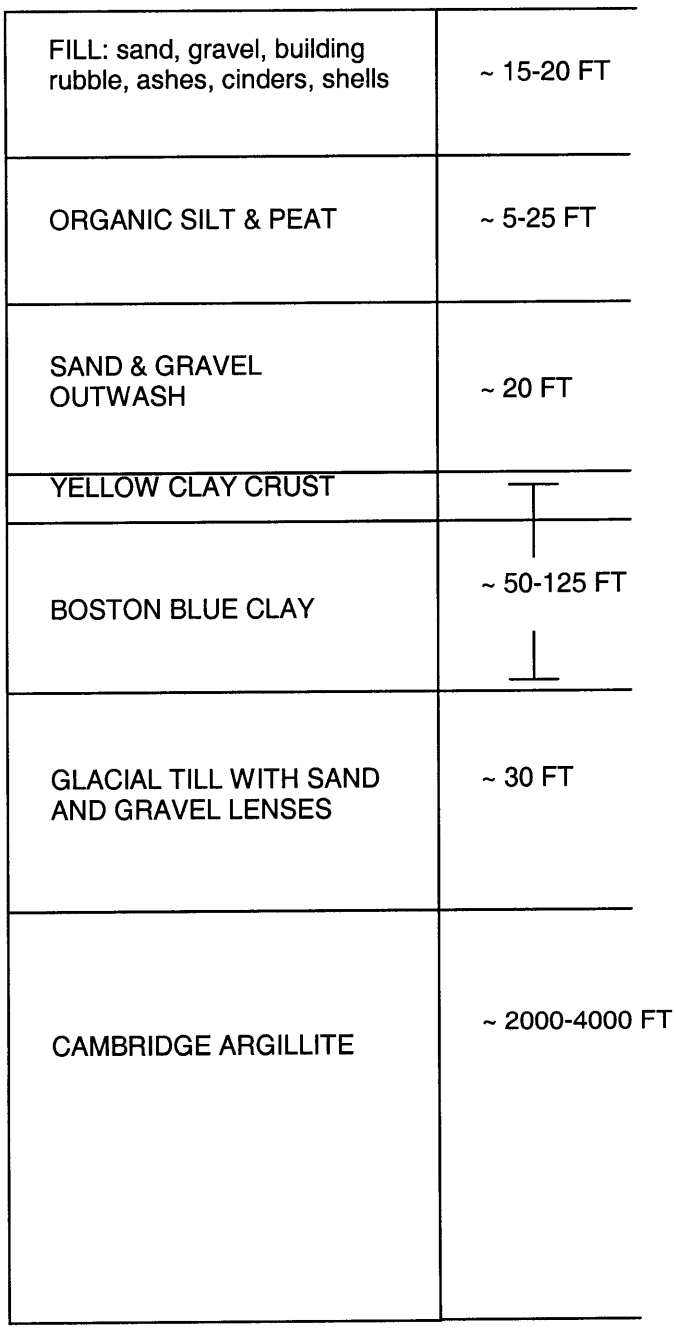
The glacial till, deposited by a Pleistocene glacier, covers the Cambridge Argillite beneath the Back Bay with thickness varying from a few feet to as thick as 30 feet in some areas. The till is an unsorted, non-stratified mix of rock fragments of all sizes compactly arranged, making excavation difficult. A relatively pervious stratum of sand and gravel, most likely an outwash deposit, occurs within the till in many locations throughout the Back Bay (Aldrich, 1970).

Overlying the glacial till is the Boston blue clay, generally occupying topographic lows between the predominant glacial till highs. Throughout the Back Bay, the clay is typically 50-125 feet thick, though in certain locations it has been found to reach depths of 180 feet or greater. This layer has been observed to contain lenses of fine sand, pockets of granular soils, and occasional boulders. The fall of sea level relative to the land subsequent to deposition of the clay layer exposed the surface of the clay to weathering and erosion. Surface layers became desiccated, forming a hard, stiff weathered crust often referred to as yellow clay (Aldrich, 1970).

A layer of sand and gravel outwash was deposited over the weathered crust of the Boston blue clay following a glacial readvance estimated to have occurred 12,000-14,000 years ago. This formation is well-developed and generally continuous throughout the Back Bay. In contrast to the glacial till, it is very pervious and is easily excavated (Aldrich, 1970).

Finally, there are three distinct types of organic soils which overlie the sand and gravel outwash: freshwater peat, organic silt with shells, and salt marsh peat. The freshwater peat accumulated from an ancient freshwater swamp and is generally thin, less than 5 feet in thickness. As sea level rose some 8,000-10,000 years ago, the freshwater peat bogs were flooded and marine silts and peats were eventually deposited over them. Sequences of outwash overlain by freshwater peat overlain by marine silt and peat are especially well-developed near the fringes of the Back Bay. In some areas the freshwater peat is absent and the marine organics are deposited directly on the outwash stratum. As a single unit, the organic soils cover the Back Bay continuously with thickness varying from 5-25 feet. This layer was considerably compressed by the uppermost layer of sand and gravel fill, which in some localized areas may also contain ashes, cinders, shells, or building rubble (Aldrich, 1970).

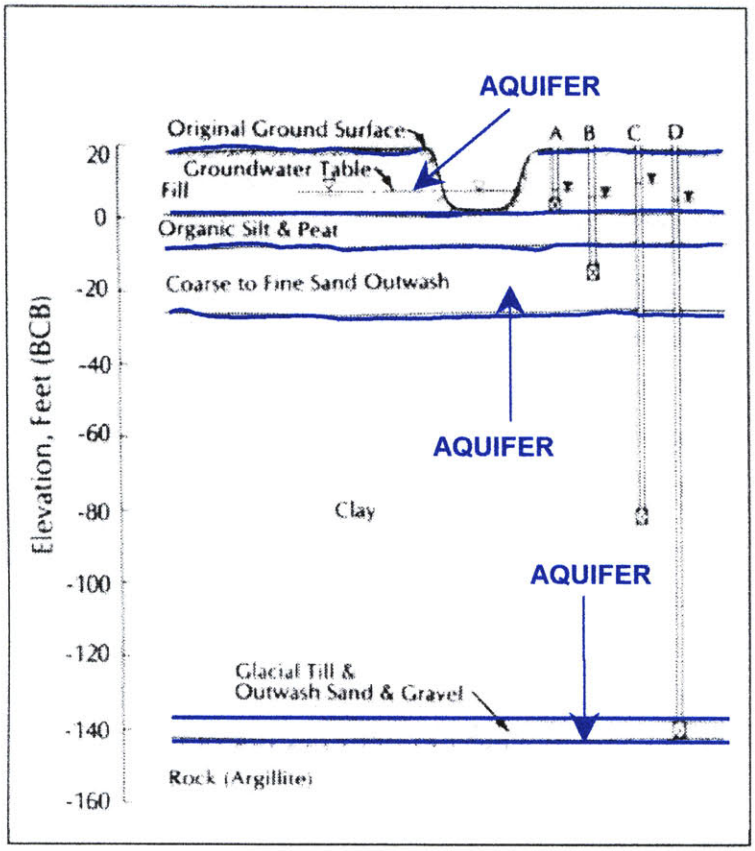
Figure II.1 – Stratigraphic Column



APPENDIX III - WATER TABLES IN THE BACK BAY

There is more than one water table beneath the Back Bay. Three main aquifers occur between layers of impervious strata (figure III.1). The lowest is a relatively thin though nearly continuous pervious layer of outwash sand and gravel and till beneath the blue clay. The second aquifer, confined by the blue clay below and the organic soils above, is pervious outwash material primarily concentrated in the western and northern sections of the Back Bay. The aquifer of principal importance in this study is the uppermost aquifer, which is confined to the fill (Aldrich and Lambrechts, 1986).

Figure III.1 – Aquifers in the Back Bay
(Aldrich and Lambrechts, 1986)



APPENDIX IV – MAXWELL'S EQUATIONS

Trying to see through dirt with radar is a tricky endeavor. The good news is that physics has nailed the mathematical theory concerning electromagnetic phenomena. Our understanding of the electrical properties of rocks and soils is not as thorough, but at least we have a solid mathematical foundation to build on. James Clerk Maxwell owes a great debt to Gauss, Ampère, and Faraday. From their observations he deduced a set of four equations which are true in any material at any point in space and as a function of time. (In this sense they are more fundamental than Newton's laws of mechanics, which fail the test of relativity). Maxwell's equations provide an elegant description of dynamic, time-varying phenomena in which the coupling of electric and magnetic fields produces electromagnetic waves capable of propagating through space and matter.

Maxwell's equations are (Ulaby, 1997):

$$\nabla \cdot \mathbf{D} = \rho_v \qquad (1)$$

$$\nabla \cdot \mathbf{B} = 0 \qquad (2)$$

$$\nabla \times \mathbf{E} = -\frac{\partial \mathbf{B}}{\partial t} \qquad (3)$$

$$\nabla \times \mathbf{H} = \mathbf{J} + \frac{\partial \mathbf{D}}{\partial t} \qquad (4)$$

where:

D is the electric displacement,

B is the magnetic flux,

E is the electric flux,

H is the magnetic field strength,

J is the electric current density,

ρ_v is the electric charge density.

Equation 1, Gauss's law, relates an electric field to its sources, electric charge. Equation 2, often referred to as Gauss's law for magnetism, essentially does the same thing for magnetic fields. It dictates that magnetic fields must be closed loops since monopoles do not appear to exist in nature the way positive and negative point charges do. Equation 3 is Faraday's law: a changing magnetic field produces an electric field. Equation 4 is derived in part from Ampère's law: a magnetic field is produced by an electric current. To complete the theory, Maxwell, on the basis of symmetry in nature, hypothesized that a magnetic field will also be produced by a changing electric field.

To better understand how electromagnetic waves behave in nature, we must integrate information about the material properties of the media through which the fields move. Avoiding any further mathematical digressions, we may simply say that three relations are important: permittivity describes the dielectric properties of a material; permeability describes a material's behavior in a magnetic field; conductivity (or, inversely, resistivity) describes the charge mobility of a material (Cist, 1999).

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