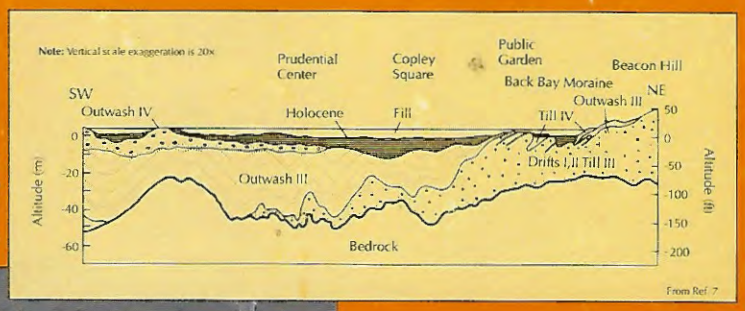
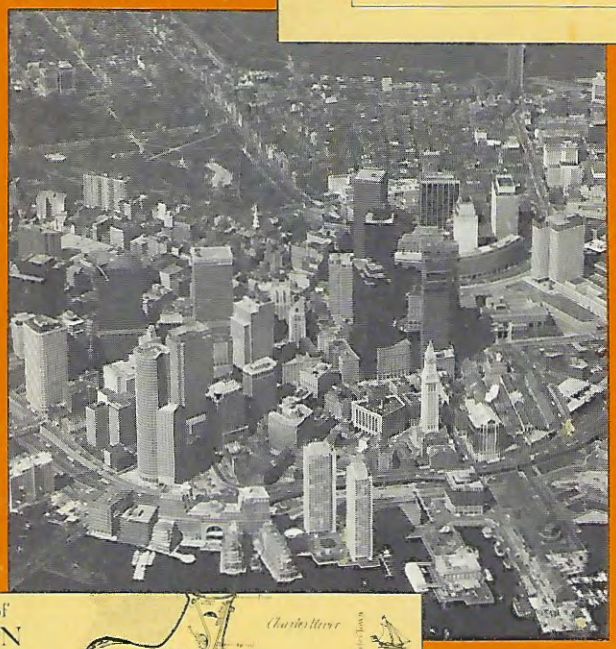
CIVIL ENGINEERING PRACTICE

JOURNAL OF THE BOSTON SOCIETY OF CIVIL ENGINEERS SECTION/ASCE

SPRING 1989 VOLUME 4, NUMBER 1 ISSN: 0886-9685

Boston Geology: A Survey of Engineering Impacts





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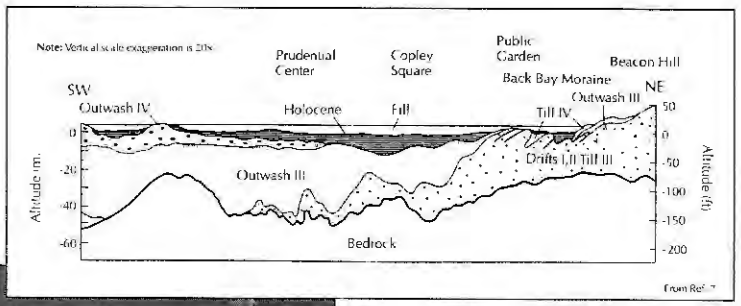

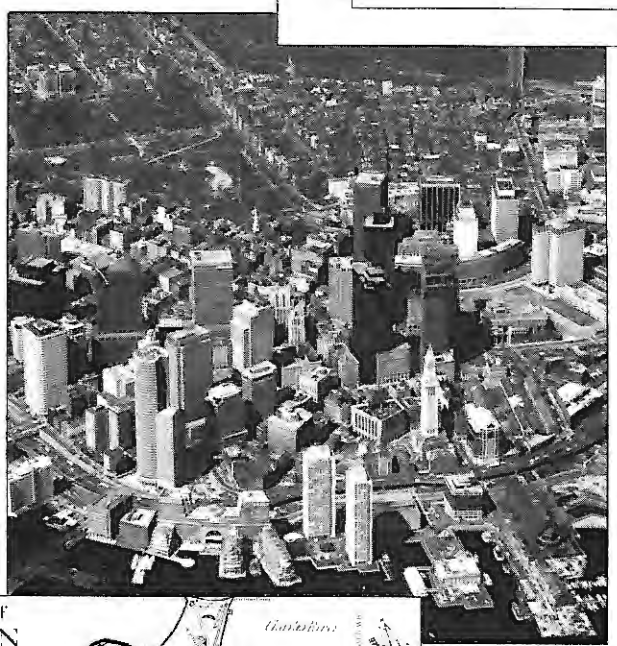

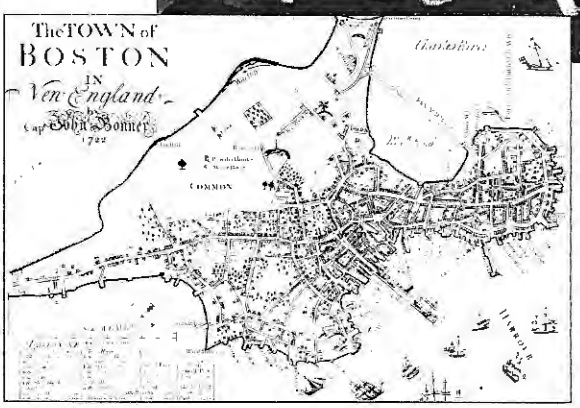

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CONTENTS

CIVIL ENGINEERING PRACTICE: JOURNAL OF THE BOSTON SOCIETY OF CIVIL ENGINEERS SECTION/ASCE (ISSN: 0886-9685) is published twice yearly in the Spring and Fall by the Boston Society of Civil Engineers Section/ASCE (founded in 1848). Editorial, circulation and advertising activities are located at: Boston Society of Civil Engineers Section/ASCE, The Engineering Center, Two Center Plaza, Boston, MA 02108; (617) 227-5551. Known as *The Journal of the Boston Society of Civil Engineers Section/ASCE* until 1985, Vol. 71, Nos. 1 & 2. Third-class non-profit bulk postage paid at Richmond, Virginia.

Subscription rates are: *U.S. & Canada* — Individual, $20.00/year; Library/Corporate, $25.00/year. *Foreign* — Individual, $25.00/year; Library/Corporate, $30.00/year.

Back issue rates for *Civil Engineering Practice* and *The Journal of the BSCE Section/ASCE* are available at $8.00 per copy. Please make all payments in U.S. dollars.

Members of the Society receive *Civil Engineering Practice* as part of their membership fees.

Civil Engineering Practice seeks to capture the spirit and substance of contemporary civil engineering through articles that emphasize techniques now being applied successfully in the analysis, justification, design, construction, operation and maintenance of civil engineering works. Views and opinions expressed in *Civil Engineering Practice* do not necessarily represent those of the Society.

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Looking Toward the 21st Century

This issue marks the beginning of the fourth year of publication of *Civil Engineering Practice: The Journal of the Boston Society of Civil Engineers Section/ASCE*. This anniversary reflects the success of the Journal in capturing the spirit and substance of civil engineering practice and it represents the Society's continuing commitment of energy and resources to the Journal. As well, the Journal will continue to present articles that are comprehensive in scope while remaining understandable to the non-specialist. This issue is of particular significance, as we head toward the close of this century in one more short decade, since eight of the ten articles contained herein are focused on the geology of the Boston area and the effects this widely contrasting geology has on engineering works.

Currently, Boston is launching one of the most massive series of developments in the city's history. Key facets of this development are the depression of the Central Artery, construction of a third harbor tunnel and the revamping of the area's wastewater management system in order to clean up Boston Harbor. These projects represent a vast undertaking that will cost in excess of $10 billion and employ more than 15,000 people over the next 10 years. It would be a mere understatement to say that all three of these projects will have far-reaching effects and benefits that will extend well into the 21st century. Due to their nature, all of these projects will require a high degree of geotechnical work.

The eight articles on Boston's geology were selected and adapted from a much larger unpublished work that is entitled, "Geology of the City of Boston, Massachusetts," edited by David Woodhouse and Patrick J. Barosh. The chapters included here were chosen on the basis of special interest to engineers and are presented as separate articles in the Journal. These articles begin with David Woodhouse's historical sketch, "The History of Boston: The Impact of Geology," on page 33. Other articles follow and include such topics as the fundamental geological characteristics of the area, geotechnical factors that bear on engineering projects throughout the city, discussions of the historical earthquake hazard and seismic-related regulations, issues affecting environmental projects and presentations of important information on major engineered structures that includes special emphasis on tunneling projects.

The overhauling of the area's wastewater management system is addressed by Donald R.F. Harleman in his article, "Boston Harbor Cleanup: Use or Abuse of Regulatory Authority?" Originally presented as a part of the Society's John R. Freeman Fund Annual Lecture, Dr.

Harleman presents a critique of the current plan to make the waters of Boston Harbor clean. As in any project with a price-tag in the billions of dollars, any disagreement as to how these monies are to spent will engender controversy. Professor Lewis Edgers, President of the Society, has noted that "the issue of advanced primary treatment and combined sewer overflow work on Boston Harbor versus secondary treatment as presently proposed is a complicated one." The Society, he points out, "has recently voted to support a study of these issues by the Water Science and Technology Board of the National Research Council." In addition, two congressmen from Massachusetts have taken an interest in reexamining the current plan. We hope that the Journal can become a forum of debate on this timely, important and sensitive issue.

In addition, in keeping with new developments in practice, an article by Fadi Karaa and James Hughes addresses how microcomputers, or personal computers, are affecting project management activities. Their discussion includes thoughts on determining whether such a personal-computer-based system would be advantageous for your firm and how to go about evaluating such a system.

We hope you will find that this issue of *Civil Engineering Practice* contains information that will help you understand and appreciate the manifold aspects of a wide range of civil engineering projects, as well as sustain your interest and provoke your thoughts. The Journal welcomes discussions on the articles published herein and invites readers to comment on this issue. Articles reflect the views of their authors and publication of any article does not represent endorsement by the Society. Let us know how we are doing and what you think.

Richard J. Scranton

Richard J. Scranton
Chairman, BSCES Journal Editorial Board

Microcomputer Configurations for Project Management

Productivity gains in construction project management can be realized via the use of estimating, scheduling and graphical software applications designed for microcomputers.

FADI A. KARAA & JAMES K. HUGHES

Computers have long been used in the construction industry. However, up until the "microcomputer revolution" — brought about by both new hardware and software systems — only large mainframe computers were available for construction jobs. These computers were located most often at the company headquarters or major field offices, and required specialized staff for their operation and maintenance as well as special facilities for their housing.

The mainframe computer had usually been acquired for accounting and financial management purposes. A construction project manager who wanted to use the system for scheduling was very frequently forced to share time with the primary users of the computer, a situation that often diluted the main value of the computer (*i.e.*, its ability to have data quickly input, to compute the results and provide updated reports that the manager could use for analysis). As a result, computer use for project management was restricted to constructing a detailed project schedule during the early project planning stage and then to periodically update the schedule for analysis, usually on a quarterly basis. However, often by the time the computer-generated scheduling reports were finished and back in the project manager's hands, the data had become outdated. Turnaround time was so long that the hand-drawn barchart still remained the most viable tool for the management of the project. Computer-generated reports and charts were retained to impress the client's representatives, showing how modern and up-to-date the construction firm was, or simply to abide by the scheduling requirements of the contract documents.

Estimating, however, remained a task whose productivity could be greatly improved through the use of such mainframes, since it requires manipulating vast amounts of basic data from historical and current records on unit costs and productivities. Interactive and computerized estimating systems are made operational by furnishing adequate computing

power that can deliver reasonable response times per transaction such as the extraction of data records or the computation of a bid item price.[1] However, since these mainframe systems required a sizeable staff base of trained operators and a large amount of capital investment for hardware and software, their use was limited to companies with sufficient resources to support the additional expenses.

In addition to the accessibility problem, mainframe systems did not allow for direct on-line inputs and tailored outputs. The system was not directly accessible by the project manager. All interaction with the system was with the staff of trained operators. Therefore, the system was of little use to the project manager, and even less to the project engineer and field staff.

The advent of the microcomputer changed much of that. The construction industry has realized the potential gain from microcomputer technology, and is applying it to such applications as estimating, scheduling and reporting.

Suckarieh cites three main microcomputer software applications systems that can benefit construction management: spreadsheets, database management systems and time management systems.[2] Word processing systems could be added to this list, since they are extensively used for the purpose of daily and weekly reports, memos and letters to suppliers, subcontractors, clients and management.

The use of spreadsheet systems for estimating is discussed by Lansford and McCullough, Suckarieh, and Herbsman.[3,2,4] Although sophisticated interactive estimating systems such as those described by Herbsman have the advantages of both larger data storage and smaller response time, particularly for standalone systems, the simplicity and low cost of spreadsheet systems have made them a convenient way to enhance the productivity of estimators by automating the time-consuming calculations involved in estimating.[4]

Database management systems that furnish the means for storing, retrieving and sorting of data enhance the estimating function by providing convenient and systematic access to input data on historical unit prices and productivities. While spreadsheet systems can also be used for data management (and many commercial spreadsheet applications do offer data management functions), they are put to better use in manipulating the data that can be supplied to the spreadsheet from data management systems. Since most commercial database management systems allow the transfer of information to popular spreadsheet systems, their combined use is a definite plus. Integrated database management and spreadsheet systems are also available today, as a response to the multiple needs for their combined use.

Microcomputer Scheduling

The availability of microcomputer scheduling systems has made it possible to "decentralize" the scheduling function, removing it from the exclusive domain of a trained staff of mainframe operators. The end user — *i.e.*, the project management team — can have a fully operational system at the project site or wherever else it is deemed suitable. The ability to input data, compute a solution, analyze reports on the screen or printed, and to modify input and test thoroughly different options represents an invaluable analysis tool for scheduling. The project manager no longer has to wait for data to be coded, entered into the mainframe system, computed and output. There is no longer any need to sort through reams of paper to find the required data, or to manually compare different outputs.

While the microcomputer does have its limitations, poses its own unique set of problems and cannot solve all of the problems of the past, it has to a large extent changed the way many construction managers and companies operate.

Project Management Software

The full value of the microcomputer for the project manager can be realized in software currently available for the management of time and resources involved in most projects. In selecting the appropriate software system, the user must carefully consider what the software is expected to do and what it actually can do. Extensive research suggests that a systematic approach to evaluating microcomputer project management software encompasses a range of factors that are discussed below.[5]

Network Input. There are basically two formats for the input of data into a project management system: activity-on-node (AON) precedence relationships and activity-on-arrow (AOA), *i, j,* node notation.

In the AON notation, once an activity has been entered into the system, a prompt appears asking for the activities that precede the entered activity. For example, for a concrete placing activity, the preceding activity would be the reinforcement placement activity. The chain of these activity relationships throughout the project defines the project length, critical path and floats.

The AOA notation relies on the start and end nodes for the activities to define the relationships and calculate the project duration. For example, if the reinforcement activity ended on node 20, then the concrete placing activity would begin at that node.

Although both formats produce the same final results, the precedence input has gained in popularity recently since it eliminates the need to have dummy or logic-only activities in the network. Ideally, a system would allow either of the two formats to be used, but that is not the case at the moment. While the decision between the two is largely a matter of individual preference, it should be recognized that some of the available software systems are better than others depending on the format selected. Current operating rules at a firm should be considered, since adjustments to a new system can result in inefficiency and low usage of the software.

Variable Calendar. The software chosen should have a means of setting the calendar for the project duration. It should be able to automatically forecast activity start and finish dates based on the overall project start date, durations of the activities and the non-work days that occur within the periods. It should also be able to calculate backwards from important milestone dates. The calendar should be able to accept holidays and variable work weeks, and to use them in the data computations. An advanced system should be able to adjust the work weeks for each activity. This ability to adjust periods is important since activities have different work-week periods. For example, adjustments need to be made for activities that work on a five-day work week such as concrete placement and activities that occur on a seven-day work week such as concrete curing. If these concrete curing activities were on the critical path, it is easy to see how this factor could be an important consideration in the overall project duration, and one that could easily be overlooked in the planning stage.

Level of Detail. The project manager is interested in the details of the activities as well as in the overall duration and cost of the project. However, it is often impractical for the project manager to look at these details if the project is large and includes many activities. For this reason, many software packages avoid this problem by supplying the capability to break the project into sub-networks. The project manager interested in the details of a particular set of activities or a particular time period is thus able to recall that portion of the supernetwork for examination while also retaining the capability of watching the big picture of the overall project.

This feature can also be used to aid in the design of the network that encompasses the framework of the project — its activities and their relationships to one another. The design step discussed above lays out the network logic used for large projects. By using subprojects for a specific series of activities such as formwork, reinforcement and concrete placement, and then inserting the subproject into the framework of the superproject, the microcomputer can facilitate the network design process.

Updating. The project management software should not only be useful for project planning, but should also permit subsequent inputs for actual costs, resource usage and task durations. The project manager is concerned with the actual performance of the project and how it compares to the planned progress. It is also desirable to have a projection capability to update the planned progress using the cost and schedule variances to date. The capability to update will give the manager the earliest possible notification of indicated changes in project cost, activity completion dates and project completion date on a continuous basis.

Updated data is essential for periodic progress and cash flow reports. The earned value of the work to date forms the necessary

basis for requesting progress payments.

Evaluation of Schedule Changes. The software selected should enable the manager to evaluate the impact on the schedule of such factors as duration, unit costs resources. As an aid to decision making it is vital that the manager be able to ask "what if?" questions while maintaining the original schedule in the file. The software system should permit quick and easy updating of the project schedule and highlight the changes that had been made and the activities that have been impacted.

Report Generation. This factor is probably the most important consideration in evaluating a project management software system. The reports that the software can generate should be tailored to the needs of the manager. A rigid system that provides set reports with excessive amounts of information are of little use to the manager. The information that the manager needs should be readily available in a format that is easily readable. A well-constructed software system would have a set of easily usable report formats, plus a utility that permits the manager to create *ad hoc* or special reports. These report formats should be able to be permanently saved (usually on disk) so the manager will not have to recreate the format of a recurring report each time it is used.

The software should permit selecting what data, or data ranges, should be included in the report. It should also have the capability to let the manager choose how the data will be sorted. A manager concerned with cutting the project length should be able to obtain a list of the critical activities. A manager concerned with resource usage should be able to obtain lists of the activities that use those resources. Auxiliary inputs are quite useful for special sorting purposes, making it possible to design reports from the subnets as well as the project as a whole.

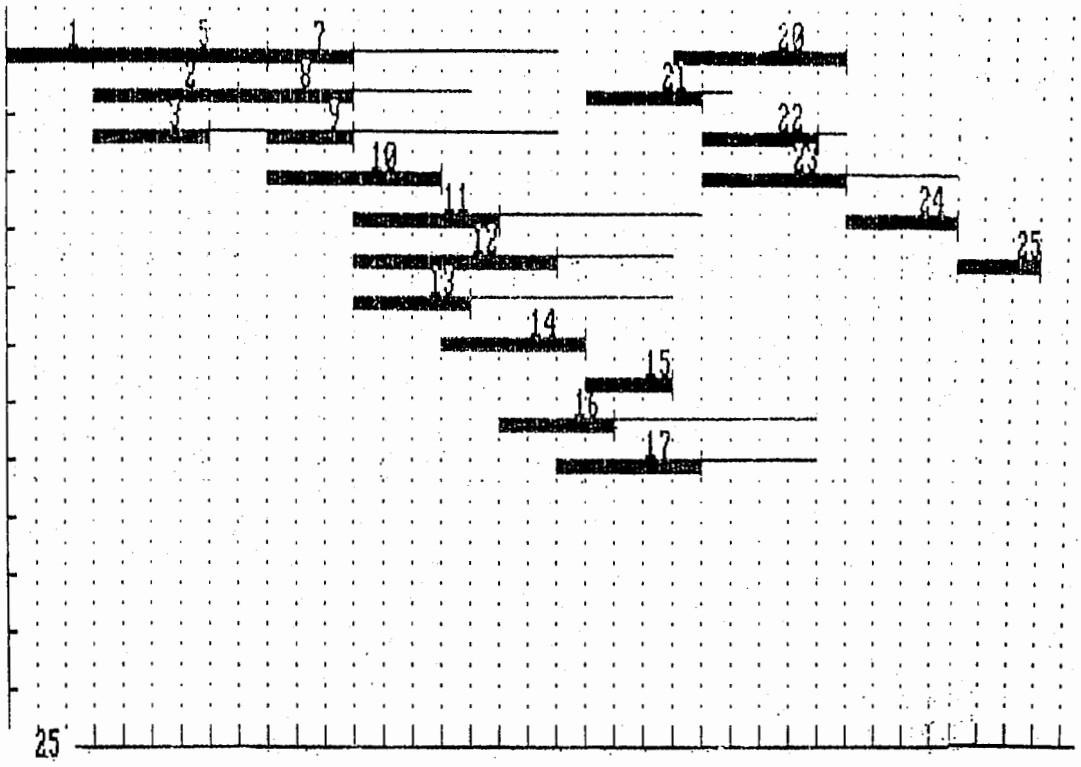
Graphical Output. Text or tabular reports are critical to detailed analysis of project, but a picture is worth a thousand words. Reports are enhanced by barcharts and network diagrams. The selected software should have facilities to tailor these outputs in the same fashion as the reports.

It is important to have the capability to produce barcharts that can depict resource usage over time. Graphs displaying variances between projected and actual expenses, cash flow and task duration are beneficial.

In evaluating which software system to purchase, consideration must be given to the hardware requirements of the system. Will the software run on the microcomputers already installed in the office? Will the computers require more memory or different graphics subsystems? Will hard disk space need to be expanded? Will the software work with the printers available in the office or must special graphics printers or plotters be purchased? Can the costs of such purchases be justified?

Execution Speed. The execution speed of the software depends primarily on the programming techniques used to write the software, the number of activities in the network and the amount of information included in the program on each of the activities. The computer runtime to execute the necessary calculations for a project can vary from five minutes up to one hour for a 500-activity network. The importance of the execution speed will depend on the role that the manager plays. If the manager is located on the site, and is concerned primarily with that one project, the manager may be willing to accept a slower execution speed since the microcomputer is not needed immediately for calculations on other projects, or for other tasks. On the other hand, if the manager is handling multiple projects, or the microcomputer is needed for other functions such as word processing, the office may not be able to afford to tie up the computer for long periods of time.

Some of the currently available programs seem to update the program as each activity is input. This type of approach slows the input process, and requires more user time, but when the input process is completed, so are the calculations. Other programs provide rapid input speeds, but are followed by lengthy calculation periods. The update-as-entered-type software, while demanding more user time, seems to allow easier error detection. It is extremely inefficient to input a great amount of detail for a project, start the computations, and only to have them break off because a logic error has been detected by the computer. The amount of time spent searching back through the data to locate the error(s) may offset the savings gained



Note: Relationships between the activities are not displayed (for example, Activity 7 does not follow Activity 5). However, Activity 9, which does follow Activity 5, does not start until that activity is complete. The float times are indicated by the narrow lines. However, they indicate only the floats for the last connected activity on a particular line.

FIGURE 1. BASIC barchart diagram.

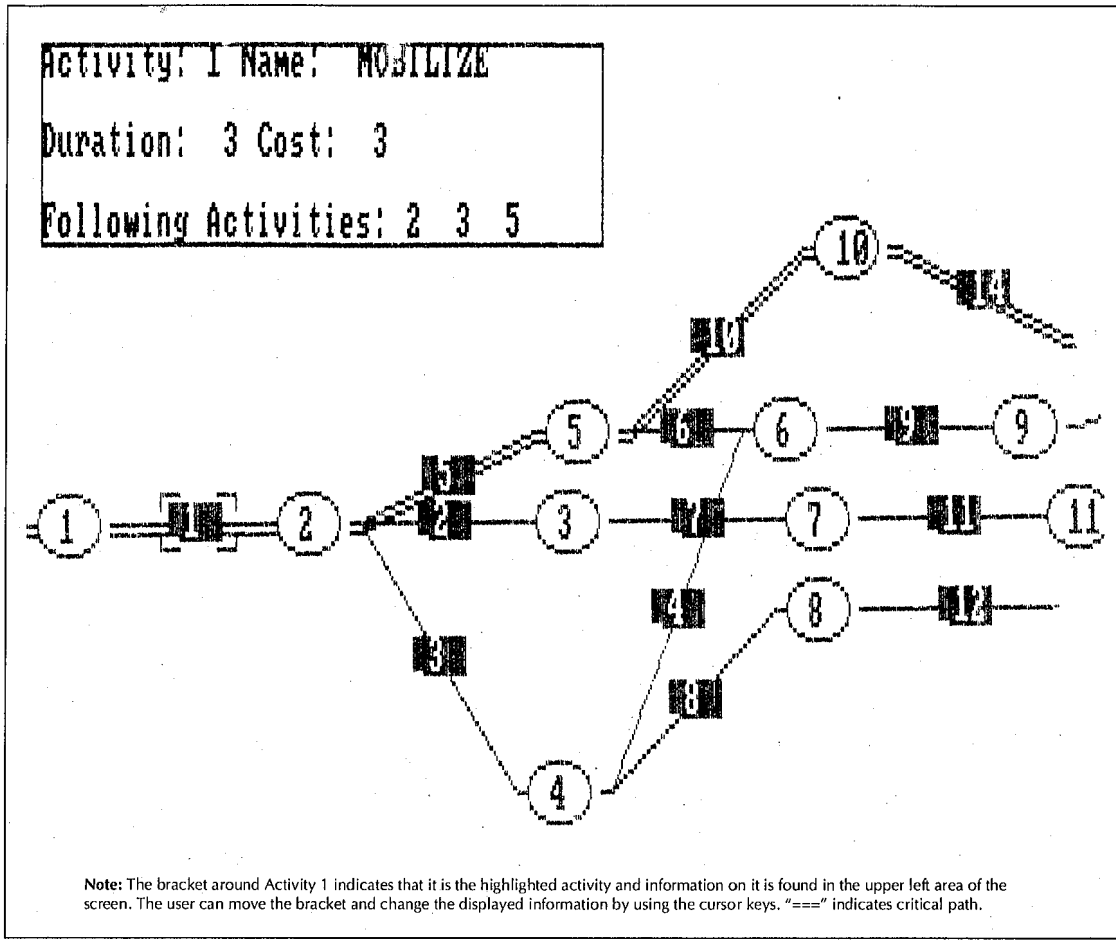
initially during the much shorter input sequence.

The purchase decision in this area should focus on the capabilities of the available hardware and the personnel available for inputting data. Execution speed is generally a function of price — the more expensive the system, the greater the speed.

Flexibility. Input sequences, and report and graph generation, must be adaptable to the needs of the office. No single software system will completely satisfy every requirement that will be placed on it. However, the manager must decide early on in the evaluation process whether the computer will drive the office or will the office dictate how the computer is to be used. In terms of the hardware the project management software might run on, consideration should be given to the other sorts of application software that might be run on the microcomputers and how all of the pieces fit. In terms of the project management software itself, a wide range of functions might make it suitable for all sorts of projects, or even for other tasks (*e.g.*, its database functions might be used to maintain small databases that might not be worth acquiring a large database software program for; or its graphic output facilities might be used to provide graphs occasionally of non-project data).

User Friendliness. There are certain traits to look for in software (and to a lesser extent in the system hardware for microcomputers since the user works with and interfaces primarily with the software) that will make it more adaptable and easy to use:

- Documentation: Is the reference material provided with the software complete and easy to understand? Are help screens



Note: The bracket around Activity 1 indicates that it is the highlighted activity and information on it is found in the upper left area of the screen. The user can move the bracket and change the displayed information by using the cursor keys. "===" indicates critical path.

FIGURE 2. Partial BASIC AOA network diagram.

provided and are they easily retrieved? Is there an on-disk tutorial with accompanying documentation.

- Command Structure/Interface: A wide variety of command input techniques are available, from typing in commands or selected abbreviations, to moving a cursor or highlighted bar. Some programs require selecting a letter followed by pressing the return key. As the software is updated, these techniques tend to become more and more sophisticated, which translates to simpler commands for the user. The important considerations here are that the commands be consistent through the program, and that the command system be easy to use and intuitive (*e.g.*, the command for saving a project file is, for example, typing the letter "s" and not typing "$IOD").
- Compatibility: Programs obviously must be compatible with the hardware already present in the office. File transfer capability between database, spreadsheet, time management and word-processing software is highly desirable. Another consideration would be compatibility with the office's peripheral equipment for microcomputers such as printers, video monitors, plotters, networking systems, *etc.* A major consideration in some offices will be compatibility with a mainframe computer. Access to the database and to data previously input to the mainframe is obviously an advantage, but one that is sadly not available as of yet for the majority of project management systems.

TABLE 1
Time Scheduling Macro:
Menu & Data Entry

```
/XMMenu~
Add    Quit    Sort    CompuErase    Barchart
Add an ActivitSort CompuErase BCreate a Barchart
(GOTO)A/xq    /xgso/xgco/xmMenu{barchart}~{loop1}~
/xnActivity Number ?~{riBarcharTable
/XnStart Node ? ~~{rightErase BErase Table
/XnEnd Node ? ~~{right}~{goto}z{goto}a10~/rea10.50~
/XnDuration ? ~~{right}~{for counter,1,50,1,loop50}
/XnCost ? ~~{end}{left}~{right 2}~re.{pgdn}{pgdn}~
```

TABLE 2
Time Scheduling Macro:
Network Computations

```
/rnddatb~/rnctest1~~/rncfin~~/rnclf~~/rncst{goto}c10~
{goto}a10~/rncdatb~{end}{down}{end}{right}~/c~b26~
/rnccount~a1~
/rnccounter~b1~/rnccounter2~c1~
{right}~
{for counter,1,count,1,strch}
{goto}datb~/rndfin~/rndlf~{right 2}~
{for counter,1,count,1,loop4}
{goto}datb~{right 6}~/rncmaxlf~~
{for counter,1,count,1,loop8}
{goto}maxlf~{left 4}~/rndfnode~/rncfnode~~
{goto}datb~{right 2}~
{for counter,1,count,fick}
{goto}datb~{end}{down}~{right}~/rndstr~/rnd
{for counter,1,count,1,loop10}
{barchart}
```

- Training/Ease of Use/Support: Is it difficult to learn how to use the software? A powerful and fast program might be one that is more difficult to learn. Does the software publisher, or do third-party vendors, provide training seminars? Does the software publisher provide technical support? Is this support free or paid? What is the level of service provided should there be problems with using the software down the line? Is the software publisher a "solvent" company that will be around to support and upgrade its product, or is it a company that might disappear, leaving users without a solid basis of technical support to solve any problems that might occur?

Cost/Performance Trade-off. Costs of available basic project management software systems for microcomputers vary widely, from under $500 to over $3,000. Other features, including software for utilities and additional functions, as well as special peripheral equipment such as graphics printers, may add several hundred to thousands of dollars to the cost of the system. The best approach here is to determine as much as possible beforehand what uses the system will be put to when it is used. Complicated resource usage tables, reports and graphs, while potentially a great aid to managing the project, are worthless if no one uses them. Their use requires that someone spend time designing the inputs to the system, inputting the initial plans and updating the data as the project goes on. Whether the potential benefits of a larger type of system are promising enough to justify the higher costs of equipment, operation and maintenance is a challenging question that the company management should face intelligently.

Special Features. Aside from the management features described above, the marginal benefits of certain "add-on" functions can be immense and help in deciding which software system to purchase. Two of these features that would aid in project management are optimization and calculation routines. Optimization routines are available that can supply the manager with what combination of durations, costs and resource usage would provide the optimum results in terms of money and time. However, this type of function often takes a lengthy period of time to execute and requires considerable time for the calculations involved in determining the optimum situation. Although the results often turn out to be impractical, they can be of valuable assistance in "brainstorming" and "what-if" analyses.

Calculation capability exists for figuring costs and durations in a free format. Conducting calculations separate from the set input format found in most software, and transferring the data to the required input field, allows the manager to correct or analyze minor changes in the data without leaving the microcomputer or the basic program that is running.

The evaluation of project management software systems can best be undertaken by running a sample project on different systems, and noting their performance according to

Activ Number	Start sNode	End fNode	Duration rncduRes		Early Start	Early Fin	Late lStart	Late lfin	SlackFol1 slackFol1		fol2 fol2	fol3 fol3	#fol #fol
1	1	2	3	1	0	3	0	3	0	5	3	2	3
2	2	3	5	1	3	8	10	15	7	7			1
3	2	4	4	2	3	7	7	11	4	8	4		2
4	4	6	0		7	7	16	16	9	9			1
5	2	5	6	2	3	9	3	9	0	10	6		2
6	5	6	0		9	9	16	16	7	9			1
7	3	7	4	4	8	12	15	19	7	11			1
8	4	8	5	4	7	12	11	16	4	12			1
9	6	9	3	4	9	12	16	19	7	13			1
10	5	10	6	5	9	15	9	15	0	14			1
11	7	11	5	6	12	17	19	24	7	16			1
12	8	12	7	7	12	19	16	23	4	17			1
13	9	14	4	8	12	16	19	23	7	20			1
14	10	13	5	3	15	20	15	20	0	21	15		2
15	13	14	3	3	20	23	20	23	0	20			1
16	11	18	4	3	17	21	24	28	7	23			1
17	12	15	5	3	19	24	23	28	4	19	18		2
18	15	17	0		24	24	29	29	5	24			1
19	15	18	0		24	24	28	28	4	23			1
20	14	17	6	7	23	29	23	29	0	24			1
21	13	16	4	7	20	24	21	25	1	22			1
22	16	17	4	1	24	28	25	29	1	24			1
23	18	19	5	1	24	29	28	33	4	25			1
24	17	19	4		29	33	29	33	0	25			1
25	19	20	3		33	36	33	36	0				0

Note: This data table is based on a 25-activity project. The start and end nodes, resources and activity numbers were entered using a macro. Another macro was used to compute the early and late start and finish dates, and other activities.

FIGURE 3. Spreadsheet output of network processing.

selected criteria. The project run would be representative of the projects normally encountered. In this evaluation process, a good way to quantify the evaluation would be to give a "weight" in numeric terms to the factors mentioned above (based on the company's or manager's estimation of the importance and value of the factor) and rate the software's performance in each factor. In an earlier report, several systems were evaluated according to the aforementioned factors.[5]

Simple Development Options

Two development options were considered:[5] a program was written in BASIC using AOA or AON notations, and a program was constructed using the macro programming techniques within the spreadsheet software that provided time analysis and reporting functions.

The BASIC program had the following capabilities:

- output report on activities, their early start and finish times, cost and duration, and floats.
- modification and updating of activities and their sequence.
- the ability to produce a bar chart (see Figure 1)
- the ability to produce a network diagram in both AOA and AON notations. A sec-

```
                                   DAILY PROGRESS

    0    5    10   15   20   25   30   35   40   45   50
-------------------------------------------------------------------------------
¦
¦
¦
¦ 1 XXX    5   3   2
¦   2 =====-------     7
¦   3 ====----     8   4
¦       4 ---------     9
¦   5 XXXXXX     10  6
¦         6 -------     9
¦        7 ====-------     11
¦       8 =====----     12
¦         9 ===-------     13
¦         10XXXXXX     14
¦            11=====-------     16
¦            12==========     17
¦            13====-------     20
¦               14XXXXX     21  15
¦                   15XXX     20
¦                 16====-------     23
¦                   17=========     19  18
¦                       18-----     24
¦                       19----     23
¦                      20XXXXXX     24
¦                   21=====-     22
¦                       22=====-     24
¦                       23=========     25
¦                           24XXXX     25
¦                               25XXX     0
¦
```

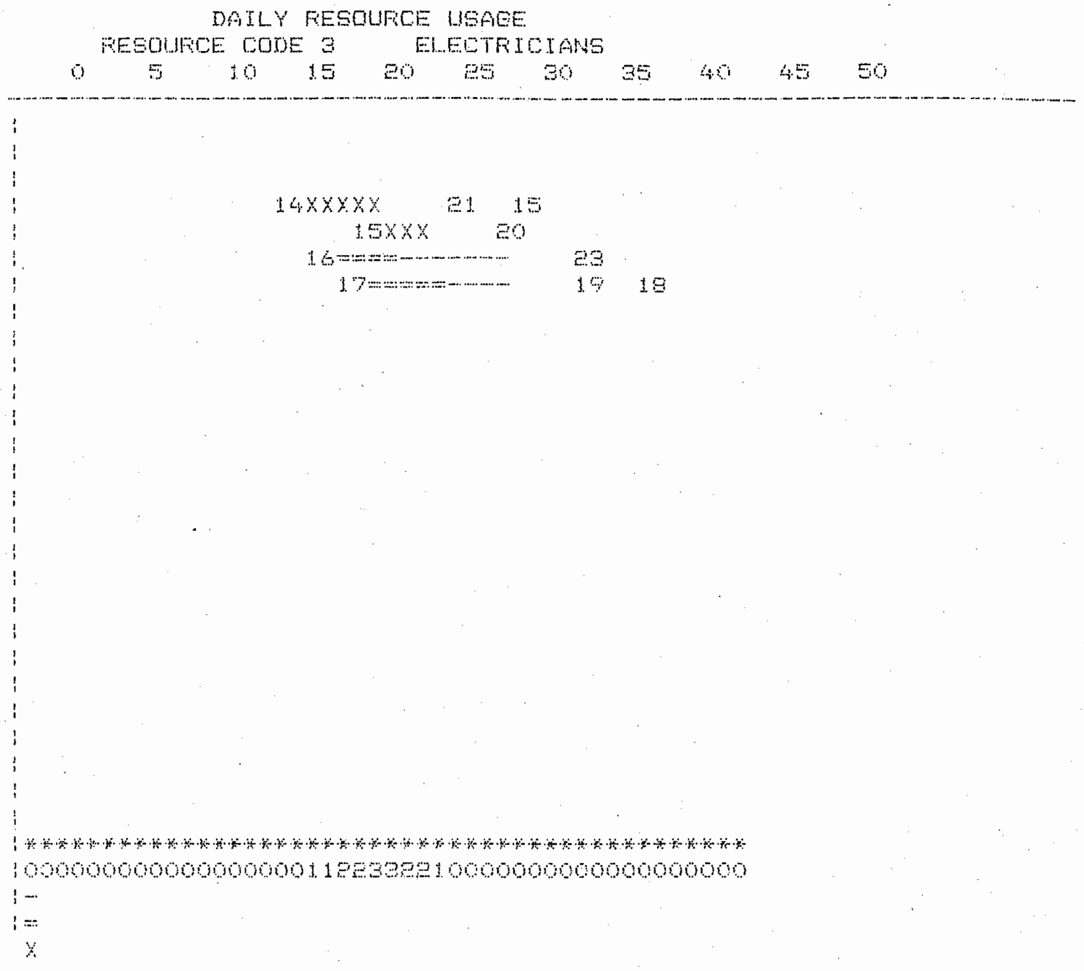
Note: This fenced barchart represents a 25-activity project covering 36 days. "XXX" indicates critical activity durations; "===" indicates non-critical activity durations; and "---" indicates float durations. The numbers on the top line show the number of working days into the project. Each cell represents one day.

FIGURE 4. Spreadsheet version of the barchart.

tion of a sample AOA diagram is shown in Figure 2.

A major advantage of using spreadsheet software to develop a project management application program is the ability to tailor the spreadsheet to the format that the user desires. Another advantage is that there are a large number of special-purpose spreadsheet software systems that are commercially available and that are adaptable to this end.[6]

The use of macros in spreadsheet software allows the development of a user-friendly, menu-driven project management system. Reporting and graphing are very well performed by spreadsheets. The disadvantages to using spreadsheet software is the relative complexity of macro programming, and a lower speed of execution than some of the sophisticated commercial project management sys-



```
            DAILY RESOURCE USAGE
      RESOURCE CODE 3      ELECTRICIANS
   0    5    10   15   20   25   30   35   40   45   50
-------------------------------------------------------------------
 :
 :
 :
 :                 14XXXXX     21   15
 :                     15XXX       20
 :                   16===--------       23
 :                     17=====-----      19   18
 :
 :
 :
 :
 :
 :
 :
 :
 :
 :
 :
 :
 :
 :
 :
 :
 :
 :
 :
 :
 :**********************************************
 :0000000000000000001122332210000000000000000000
 :-
 :=
  X
```

Note: This fenced barchart is based on activities that use a single resource, namely electricians, or Resource No. 3. "XXX" indicates critical activity durations; "===" indicates non-critical activity durations; and "---" indicates float durations. The activity table was first sorted to select those activities using the resource. This table was then used as the basis for the barchart. The rows of asterisks and numbers indicate the total of that resource used that day.

FIGURE 5. Fenced barchart selected on a single resource.

tems. In particular, calculations for time management and network processing are slower on spreadsheets.

Time Scheduling Macro

The time scheduling macro consisted of:

- Entering the activity data via instructions, such as shown in Table 1.
- Sorting on start node then finish node.
- Computing such values as early start, early finish, late start, late finish, slacks and the following activities as shown in Table 2.

These sequences above were performed on a 25-activity project.

Reporting & Graphing

The output of the time scheduling run is reproduced in a spreadsheet represented in Figure 3. This figure displays the data table obtained by "reading" the outputs of the time management system. The data displayed can be easily changed by adding and deleting the columns and changing the steps in the macro.

Based on the time scheduling outputs, macros can be used to generate such customized reports as:

Date: 1/ 1/80
Project Id: HOSPITA
Project Start: 1/ 1/86
Project Currency: 1 US$
Base: newclass

****
* PALM WEST HOSPITAL *
* P R O M I S *
****

Page 1
Last Acc: 1/ 1/80
Last Upd:

Report #: 541-1

ACTIVITY INPUT LISTING

Selection: All

ACT. ID	SUBPROJ. ID	ACTIVITY DESCRIPTION	EST DUR	PLANNED S/F DATE	SCHED CONS.	ACTIVITY CODES WWC	RESP	AUX1	AUX2	AUX3
01		START	0	1/ 1/86	SNLT	5d				
02		MOBILIZATION	2	1/ 1/86	SNLT	5d	GC			
03		MACHINE EXCAVATION	5			5d	GC			
04		HAND EXCAVATION	4			5d	GC			
05		BACKFILL/COMPACTION	16			5d	GC			
06		FORMWORK/SPREAD FOOTING	12			5d	GC			
07		FORMWORK/CONTINUOUS FOOTINGS	1			5d	GC			
08		GRADE BEAM FORMS	7			5d	GC			
09		FORMWORK/FOUNDATION WALL	3			5d	GC			
10		FORMWORK/SLAB ON GRADE	2			5d	GC			
11		FORMWORK/RETAINING WALLS	5			5d	GC			
12		REINFORCEMENT/SPREAD FOOTING	5			5d	GC			
13		REINFORCEMENT/CONT. FOOTING	1			5d	GC			
14		REINFORCEMENT/GRADE BEAM	4			5d	GC			
15		REINFORCEMENT/FOUNDATION WALL	1			5d	GC			
16		WELDING WIRE FABRIC	44			5d	GC			
17		REINFORCEMENT/RETAINING WALL	1			5d	GC			
18		CAST/SPREAD FOOTINGS	4			5d	GC			
19		CAST/CONTINUOUS FOOTINGS	1			5d	GC			
20		CAST/GRADE BEAM	1			5d	GC			
21		CAST/FOUNDATION WALL	1			5d	GC			
22		CAST/SLAB ON GRADE	8			5d	GC			
23		CAST/RETAINING WALL	1			5d	GC			
24		TROWEL FINISH SLAB	12			5d	GC			
25		CURE AND HARDEN	14			5d	GC			
26		CURE VERTICAL SURFACE	13			5d	GC			
27		FINISH	0			5d				

FIGURE 6. Activity input listing for a time and resource scheduling system.

- a fenced barchart (see Figure 4)
- a daily resource usage barchart for a given resource code using the spreadsheet's sorting capability
- different graphics outputs (for example, a graph of a resource versus time as depicted in Figure 5)

Figure 4 presents a barchart printed out from the information contained in Figure 3. The barchart is fenced by the activities that follow. The number of the activity is listed in front of the symbol for the activity. The duration of the project is indicated by "===" or "XXX" for critical activities. The floats for the activities are indicated by "---". While the printouts are not particularly fancy, they do provide the manager with a visual display of the project progress.

Figure 5 reveals a barchart for activities utilizing a particular resource required for a sample project, electricians in this case. The data table was reselected to display only activities using the specified resource. The barchart was then reprinted.

The real value of using spreadsheet software for this purpose is the flexibility that the user has in designing the system and designating the inputs, the data to be used and the selection and sorting criteria, as well as the ability to

```
Date:  1/ 1/80        ************************************                Page    1
Project Id: HOSPITA   *          PALM WEST HOSPITAL          * Last Acc:  1/ 1/80
Proj Start:  1/ 1/86  *              P R O M I S             * Last Upd:
Proj Currency: 1 US$  ************************************
'  abase: newclass                                                  Report #: 543-2
                                PROJECT RESOURCE TABLE


Selection: All
```

RESOURCE ID	DESCRIPTION	UNITS OF MEASURE	STANDARD RATE	OVERTIME RATE	DEF. OH. PROFIT	MAX DAILY AVAILABLE
1	REINFORCING STEEL	TON	480.00	0.00	10.00	50.00
2	WELDING WIRE FABRIC	SQ. FT.	7.65	0.00	10.00	7500.00
3	CONCRETE	CU. YD.	46.00	0.00	10.00	0.00
B-10A	COMPACTION	CREW-DAY	289.85	434.78	10.00	0.00
B-12N	EXCAVATION	CREW-DAY	788.00	1182.00	10.00	0.00
BLAB	BUILDING LABORER	MAN-DAY	122.00	183.00	10.00	0.00
C-1	FORMWORK	CREW-DAY	605.00	907.00	10.00	0.00
C-2	FORMWORK	CREW-DAY	936.00	1404.00	10.00	0.00
C-6	CONCRETE FINISHING	CREW-DAY	824.30	1236.45	10.00	0.00
C-9	CONCRETE FINISHING	CREW-DAY	175.90	263.85	10.00	0.00
CFW	CONT. FOOTING FORM	SQ. FT.	0.48	0.00	10.00	300.00
FWW	FOUNDATION WALL FORM	SQ. FT.	0.77	0.00	10.00	1050.00
GBW	GRADE BEAM FORMS	SQ.FT.	0.73	0.00	10.00	3000.00
RDMAN	RODMAN	MAN-DAY	166.40	249.60	10.00	0.00
RWW	RETAINING WALL FORMS	SQ. FT.	0.86	0.00	10.00	1700.00
SFW	SPREAD FOOTING FORMS	SQ. FT.	0.55	0.00	10.00	4500.00
[illegible]	SLAB ON GRADE FORMS	LIN. FT.	0.17	0.00	10.00	850.00

FIGURE 7. Project resource table generated by a time and resource scheduling system.

perform "what-if" analyses. However, considerable time might be spent by the user in writing and revising the programs to suit his needs. In fact, the macros might need to be revised for each different project. Another disadvantage is the amount of time that spreadsheets use to complete calculations. For example, the macro utility from the spreadsheet software used here was intended for short operations. Calculations for construction project management often require long macro scripts and repeated calculations, posing distinct problems in debugging. Still, writing efficient macro programs is possible and it does have certain advantages in its customization capability.

Other capabilities that are available on most commercially available spreadsheet software programs can also be used in construction management. The graphical outputs are an obvious addition to any reports that the program generates. There are also various statistical packages that could be adapted and used in conjunction with spreadsheet programs for aiding in the computation of overtime, and in developing production estimates.

Standard Sophisticated Systems

Another project management software option is to use one of the commercially available sophisticated standard time and resource scheduling software systems. This type of software can handle very large projects, and provides a wide variety of features such as subnetworking, reporting of progress-to-date performance as well as projecting time and cost performance.

A sample project consisting of a hospital substructure was run on one of these dedicated application programs. The activity listing is shown in Figure 6. The software maintains resources and costs, which can be used to create a project resource table as shown in Figure 7. After the network has been processed, a wide variety of standard reports and graphs are available, such as the resource requirements listing and costing reports (see Figures 8 and 9), time analysis reports, and graphs such as a barchart (see Figure 10), network diagram, cash

Date: 1/ 1/80 | Page 1
Project Id: HOSPITA | Last Acc: 1/ 1/80
Project Start: 1/ 1/86 | Last Upd:
Project Currency: 1 US$
Base: newclass | Report #: 541-3

****
* PALM WEST HOSPITAL *
* P R O M I S *
****

RESOURCE REQUIREMENTS LISTING

Selection: All

ACTIVITY ID	DESCRIPTION	ESTIMATED DURATION	WORK WEEK CODE	RESOURCE ID	RESOURCE DESCRIPTION	QUANTITY NEEDED	UNITS OF MEASURE	DAY OF ACT	NO. OF DAYS	S / O	JOB CODE	OH & PROFIT (%)
03	MACHINE EXCAVATION	5	5d	B-12N	EXCAVATION	5.00	CREW-DAY	1	5	S		10.00
04	HAND EXCAVATION	4	5d	BLAB	BUILDING LABORER	40.00	MAN-DAY	1	4	S		10.00
05	BACKFILL/COMPACTION	16	5d	B-10A	COMPACTION	16.00	CREW-DAY	1	16	S		10.00
06	FORMWORK/SPREAD FOOTING	12	5d	C-1	FORMWORK	12.00	CREW-DAY	1	12	S		10.00
				SFW	SPREAD FOOTING FORMS	4424.00	SQ. FT.	1	12	S		10.00
07	FORMWORK/CONTINUOUS FOOTINGS	1	5d	C-1	FORMWORK	1.00	CREW-DAY	1	1	S		10.00
				CFW	CONT. FOOTING FORM	288.00	SQ. FT.	1	1	S		10.00
08	GRADE BEAM FORMS	7	5d	C-2	FORMWORK	7.00	CREW-DAY	1	7	S		10.00
				GBW	GRADE BEAM FORMS	2820.00	SQ.FT.	1	3	S		10.00
09	FORMWORK/FOUNDATION WALL	3	5d	C-2	FORMWORK	3.00	CREW-DAY	1	3	S		10.00
				FWW	FOUNDATION WALL FORM	997.00	SQ. FT.	1	2	S		10.00
10	FORMWORK/SLAB ON GRADE	2	5d	C-2	FORMWORK	2.00	CREW-DAY	1	2	S		10.00
				SGW	SLAB ON GRADE FORMS	750.00	LIN. FT.	1	1	S		10.00
11	FORMWORK/RETAINING WALLS	5	5d	C-2	FORMWORK	5.00	CREW-DAY	1	5	S		10.00
				RWW	RETAINING WALL FORMS	1646.00	SQ. FT.	1	3	S		10.00
12	REINFORCEMENT/SPREAD FOOTING	5	5d	1	REINFORCING STEEL	29.90	TON	1	5	S		10.00
				RDMAN	RODMAN	40.00	MAN-DAY	1	5	S		10.00
13	REINFORCEMENT/CONT. FOOTING	1	5d	1	REINFORCING STEEL	0.30	TON	1	1	S		10.00
				RDMAN	RODMAN	2.00	MAN-DAY	1	1	S		10.00
14	REINFORCEMENT/GRADE BEAM	4	5d	1	REINFORCING STEEL	10.20	TON	1	4	S		10.00
				RDMAN	RODMAN	16.00	MAN-DAY	1	4	S		10.00
15	REINFORCEMENT/FOUNDATION WALL	1	5d	1	REINFORCING STEEL	0.60	TON	1	1	S		10.00
				RDMAN	RODMAN	2.00	MAN-DAY	1	1	S		10.00
16	WELDING WIRE FABRIC	44	5d	2	WELDING WIRE FABRIC	7260.00	SQ. FT.	1	44	S		10.00
				RDMAN	RODMAN	440.00	MAN-DAY	1	44	S		10.00
17	REINFORCEMENT/RETAINING WALL	1	5d	1	REINFORCING STEEL	2.00	TON	1	1	S		10.00
				RDMAN	RODMAN	2.00	MAN-DAY	1	1	S		10.00
18	CAST/SPREAD FOOTINGS	4	5d	3	CONCRETE	392.00	CU. YD.	1	2	S		10.00
				C-6	CONCRETE FINISHING	4.00	CREW-DAY	1	4	S		10.00
19	CAST/CONTINUOUS FOOTINGS	1	5d	3	CONCRETE	4.00	CU. YD.	1	1	S		10.00
				C-6	CONCRETE FINISHING	1.00	CREW-DAY	1	1	S		10.00
20	CAST/GRADE BEAM	1	5d	3	CONCRETE	134.00	CU. YD.	1	1	S		10.00
				C-6	CONCRETE FINISHING	1.00	CREW-DAY	1	1	S		10.00
21	CAST/FOUNDATION WALL	1	5d	3	CONCRETE	8.00	CU. YD.	1	1	S		10.00
				C-6	CONCRETE FINISHING	1.00	CREW-DAY	1	1	S		10.00
22	CAST/SLAB ON GRADE	8	5d	3	CONCRETE	1178.00	CU. YD.	1	4	S		10.00
				C-6	CONCRETE FINISHING	8.00	CREW-DAY	1	8	S		10.00
23	CAST/RETAINING WALL	1	5d	3	CONCRETE	27.00	CU. YD.	1	1	S		10.00
				C-6	CONCRETE FINISHING	1.00	CREW-DAY	1	1	S		10.00
24	TROWEL FINISH SLAB	12	5d	C-9	CONCRETE FINISHING	12.00	CREW-DAY	1	12	S		10.00
25	CURE AND HARDEN	14	5d	BLAB	BUILDING LABORER	140.00	MAN-DAY	1	14	S		10.00
	CURE VERTICAL SURFACE	13	5d	BLAB	BUILDING LABORER	52.00	MAN-DAY	1	13	S		10.00

FIGURE 8. Resource requirements listing output by a time and resource scheduling system.

flow and resource profiles, and earned value analysis. Resource-constrained scheduling is also possible from some of these dedicated programs which provide an integrated project management environment.

Make or Buy Decisions

From the preceding discussion, it is quite clear

Date: 1/ 1/80
Project Id: BEACH
Project Start: 1/ 1/86
Project Currency: 1 US$
Database: GROUP2

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* PALM WEST HOSPITAL *
* P R O M I S *
****

Page 1
Last Acc: 1/ 1/80
Last Upd:

Report #: 513-3

RESOURCE COSTING REPORT - TARGET SCHEDULE

Selection: All

RESOURCE ID	DESCRIPTION	UNITS OF MEASURE	REQUIREMENTS FROM	REQUIREMENTS TO	COST/DAY	NO. OF DAYS	COST	CUMULATIVE COST
B10A	MECH EXC	CREW DAYS	1/16/86	1/17/86	289.85	2	579.70	579.70
			1/20/86	1/24/86	289.85	5	1449.25	2028.95
			1/27/86	1/31/86	289.85	5	1449.25	3478.20
			2/ 3/86	2/ 6/86	289.85	4	1159.40	4637.60
					TOTAL:	16	4637.60	
B12N	MACH EXC 2	CREW DAYS	1/ 3/86	1/ 3/86	788.00	1	788.00	788.00
			1/ 6/86	1/ 9/86	788.00	4	3152.00	3940.00
					TOTAL:	5	3940.00	
BL	BUILDING LABORER	MAN DAYS	1/10/86	1/10/86	1220.00	1	1220.00	1220.00
			1/13/86	1/15/86	1220.00	3	3660.00	4880.00
			3/17/86	3/21/86	488.00	5	2440.00	7320.00
			3/24/86	3/28/86	488.00	5	2440.00	9760.00
			3/31/86	4/ 2/86	488.00	3	1464.00	11224.00
			6/13/86	6/13/86	1220.00	1	1220.00	12444.00
			6/16/86	6/20/86	1220.00	5	6100.00	18544.00
			6/23/86	6/27/86	1220.00	5	6100.00	24644.00
			6/30/86	7/ 2/86	1220.00	3	3660.00	28304.00
					TOTAL:	31	28304.00	
C1	CARPENTERS1	CREW DAYS	2/ 7/86	2/ 7/86	1210.00	1	1210.00	1210.00
			2/10/86	2/14/86	605.00	5	3025.00	4235.00
			2/17/86	2/21/86	605.00	5	3025.00	7260.00
			2/24/86	2/24/86	605.00	1	605.00	7865.00
					TOTAL:	12	7865.00	
C2	CARPENTERS2	CREW DAYS	2/10/86	2/12/86	1872.00	3	5616.00	5616.00
			2/13/86	2/14/86	936.00	2	1872.00	7488.00
			3/13/86	3/14/86	1872.00	2	3744.00	11232.00
			3/17/86	3/21/86	936.00	5	4680.00	15912.00
					TOTAL:	12	15912.00	

FIGURE 9. A resource costing report that has been generated by a time and resource scheduling system.

that the implementation of a simple project management software system is feasible. In particular, the combination of database management software and spreadsheet software is particularly advantageous, and provides the benefits of customized reporting. However, the computational power of dedicated time and resource management software can be fully exploited only with a significant in-house investment in supplementary software development that far exceeds the purchase price of the basic system.

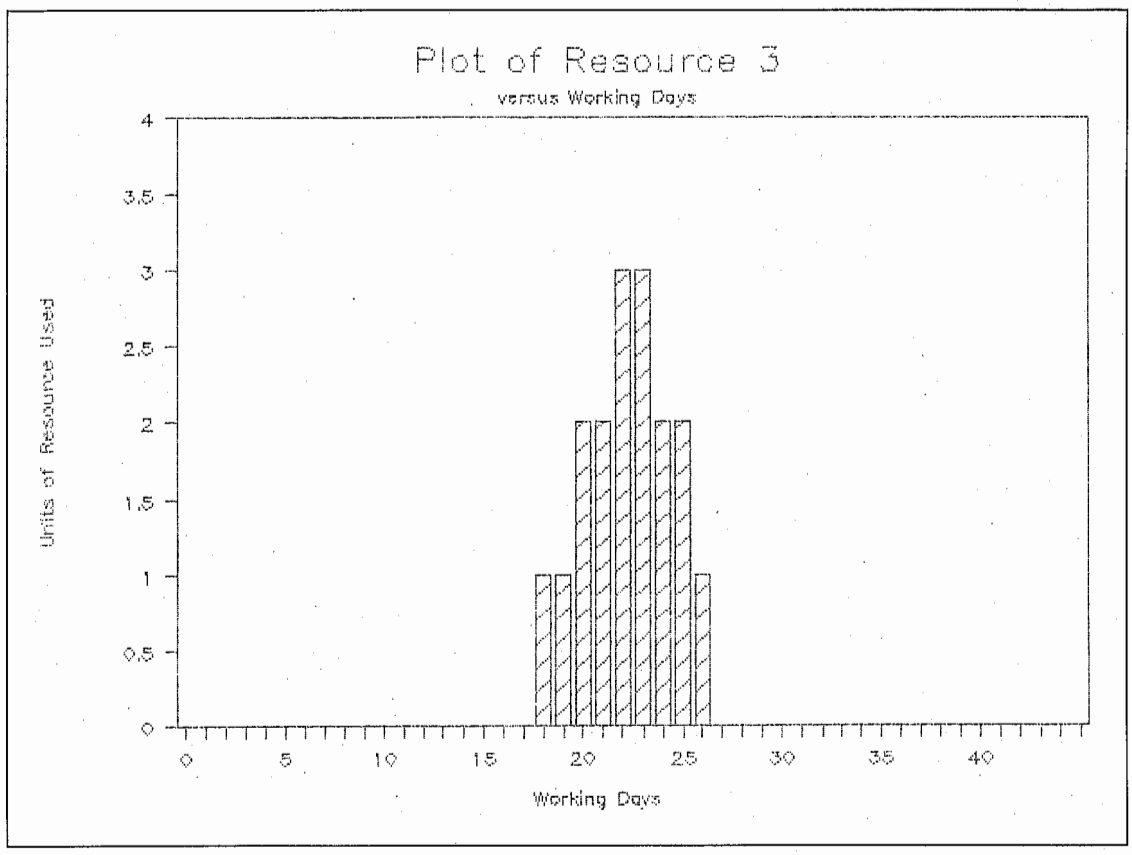

FIGURE 10. Resource profile produced using the graphics capabilities of spreadsheet software.

In Table 3, database management, spreadsheets, integrated time and resource management systems, and word processing software are evaluated for different functions such as estimating, time management, standard/customized reporting, *etc.* The only areas of relative weakness of integrated project management software are in estimating (often performed by specialized software), and customized report and graph generation. Spreadsheets, and possibly database management software, can be used in conjunction with a sophisticated time and resource management system. Some of these dedicated programs provide a file transfer feature that simplifies interchanging data with other spreadsheet and database management programs. Using macros such as those described above, customized reporting or inputting capabilities can be created by reading from, or writing to, the time and resource management system.

Conclusions

The company that is about to select a microcomputer project management system is faced with a wide variety of choices of software with varying prices and capabilities. The quality and features of this software are undergoing continuous improvement and change.

Developing programs in-house can be considered for simple applications. A combination of commercially-available sophisticated project management software, spreadsheet and database management software can enhance overall system performance significantly, particularly when supplemented by customized reporting and input utilities developed in-house. This latter approach is generally the most cost-effective development option, unless the advantages of compatibility with other functions such as job costing and cost control dictate the development of a costlier, more fully

TABLE 3
Usefulness of Different Software Systems for Construction Management Functions

	Software System			
Function	**Database Management**	**Spreadsheet**	**Integrated Time & Resource Management**	**Word Processing**
Estimating	UC	U	L	S
Network Design	N/A	L	U	N/A
Time Management	N/A	L	U	N/A
Resource Management	N/A	L	U	N/A
Standard Reports	N/A	N/A	U	N/A
Standard Graphics	N/A	N/A	U	N/A
Customized Reports	U/UC	U	L	S
Customized Graphics	U/UC	U	L	S

Key: U = Useful UC = Useful when combined with other systems L = Limited use S = Support N/A = Not applicable

customized system, with the associated need for a much more expensive and time-consuming in-house programming and debugging effort.

NOTES — *The spreadsheet software used for this article was Lotus 1-2-3, by Lotus Development. The dedicated project management system was PROMIS, by Strategic Software Planning Corp. The term, "network," as used here refers to the interrelated elements encompassed within a project and not to the linking together of computers, peripherals, instruments and other devices.*

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JAMES K. HUGHES *is a Major in the United States Army, and is a member of the Corps of Engineers. He is a graduate of the Construction Management Program at Northeastern University.*

REFERENCES

1. Herbsman, Z., and Mitrani, J.D., "INES — An Interactive Estimating System," *Journal of Construction Engineering and Management*, ASCE, Vol. 110, No. 1, March 1984, pp. 1933.

2. Suckarieh, G., "Project Management Control with Microcomputers," *Journal of Construction Engineering and Management*, V. 110, N. 1, March 1984, pp. 72-78.

3. Lansford, C.B., and McCullough, B.G., "Microcomputer Spreadsheet Applications," *Journal of Construction Engineering and Management*, ASCE, Vol. 109, No.2, June 1983, pp. 214-223.

4. Herbsman, Z., "Project Management Training Using Microcomputers," *Journal of Management in Engineering*, ASCE, Vol. 2, No. 3, July 1986, pp. 165-176.

5. Hughes, J.K., "Microcomputer Use in Construction Management," Master's Report submitted to the Department of Civil Engineering, Northeastern University, June 1986.

6. Waddill, J.A., and Mayes, K.A., "Teaching Construction Scheduling through Advanced Spreadsheet Application," Proceedings of the Twenty-Second Annual Conference of the Associated Schools of Construction, April, 1986, pp. 32-38.

Boston Harbor Cleanup: Use or Abuse of Regulatory Authority?

Does the current regulatory climate ensure that the plan proposed to clean the harbor represents the best solution environmental technology can offer?

DONALD R.F. HARLEMAN

Boston Harbor is frequently described as the "dirtiest" in the country. During the presidential campaign the harbor was the subject of heated debate on whether its condition and the delay in cleaning it up was the fault of federal or state and local government. Regardless of who is at fault, a set of plans and priorities for the cleanup of Boston Harbor is finally in place. The present court-ordered schedule calls for the construction of a new primary treatment plant, interim sludge disposal facilities and a nine-mile ocean outfall by 1995. Additional treatment in a secondary stage, as well as facilities for the land disposal of all sludge, is required by 1999. The estimated cost of all of these new facilities is $6.1 billion. The bulk of these costs will be footed by the users of the system. In this way, Boston area residents will pay the highest water and sewer rates in the nation largely because the federal Environmental Protection Agency's (EPA) construction grant program, which previously would have paid 75 percent or more of the cost, has now been phased out of the federal budget.

Do the present court-ordered facilities and the schedule for the cleanup represent an environmentally sound solution for Boston Harbor? The answer is no. To understand the reasons why the current plan does not deliver its intended consequences, it is necessary to go back to the passage of the federal Clean Water Act of 1972, and beyond that to the evolution of the harbor pollution problem and plans by other governmental agencies to remedy it.

Clean Water Act of 1972

The Clean Water Act of 1972 required that all publicly-owned treatment plants, without regard to the nature or location of the water into which they discharge, achieve secondary treat-

ment by 1977. The EPA defined such treatment in terms of the amount by which two quantities — biochemical oxygen demand (BOD) and suspended solids — are reduced in the treatment plant effluent. BOD is of concern because it tends to reduce dissolved oxygen levels in the receiving water. The severity of the depletion depends on a replenishment process, known as reaeration, in which oxygen is transferred from the air through the water surface. If the treatment plant effluent is discharged to a fresh water stream, the dissolved oxygen may drop to a level that is harmful to aquatic life because reaeration is limited by small water surface areas. In such cases, high levels of BOD removal are desirable. In the ocean, water surface areas are large and oxygen is replenished readily: hence BOD removal is less critical. Suspended solids are of concern because their removal in the treatment process improves the clarity and reduces the formation of bottom deposits in the receiving water. In addition, hazardous and toxic substances tend to be adsorbed onto suspended solids.

A primary plant, which is the first stage of any treatment system, removes about 40 percent of the incoming BOD and 60 percent of the suspended solids in a series of sedimentation tanks. The secondary treatment stage is a biological process that provides additional treatment to the effluent of the primary plant. The result is that the overall removals of both BOD and the suspended solids are increased to about 85 percent. The additional removal of BOD in the secondary stage is accomplished by adding oxygen to the treatment tanks in order to speed the growth of bacteria that feed on, and thereby oxidize, dissolved organic oxygen-demanding material in the wastewater. The bacteria, which continually grow and die, and other suspended solids settle out in the secondary sedimentation tanks.

The sediment residue from the primary and secondary treatment stages is called sludge. The sludge contains everything that has been removed from the raw wastewater, as well as new biomass generated in secondary treatment. Consequently, the amount of sludge produced by the combined primary-secondary stages is about twice as much as is produced by the primary stage alone (p. 2-2).[1] Current regulations prohibit the ocean disposal of sludge.

Soon after the 1972 act was passed, many municipalities argued to Congress and the EPA that secondary treatment was not universally necessary for the protection of the coastal marine environment. They contended that large reductions in BOD, while important for inland freshwater streams and lakes, were of little benefit to the coastal ocean where treatment plant effluents are mixed and dispersed by tidal currents and aerated by large water surface areas. They also pointed out that long outfall pipes could terminate in coastal areas at a significant depth in the water and that these areas exhibited substantial tidal flushing action. Furthermore, multi-port diffusers, thousands of meters in length, could be attached to the outfalls in order to reduce the concentration of treated effluents by more than a hundredfold through the process of jet mixing. A number of communities in the Los Angeles area that have been discharging primary effluent through ocean outfalls had accumulated evidence that demonstrated the scientific merit of their claims for exemption from the secondary treatment requirement. Congress was persuaded and in the Clean Water Act of 1977 (in particular, in Section 301(h)) directed the EPA to allow municipal marine dischargers to test their case in the administrative process.

Boston Harbor Pollution Background

Boston began discharging untreated waste into the harbor more than a hundred years ago. It was not until 1968 that all dry weather sewage began receiving primary treatment at Nut and Deer Islands. However, that treatment has been essentially negated by the fact that the primary effluent and the sludge is discharged, after some digestion and chlorination, through short pipes near the entrance to the harbor. Although the ill effects of this practice were sought to be ameliorated by discharging the sludge mainly during the outgoing tide in order to disperse it more adequately, much of Boston's present problem is due to this long-banned but continuing practice.

The first serious water quality study of Boston Harbor was completed in the late 1960s.[2,3]

The findings of this study were that the primary treatment was satisfactory, but the disposal of sludge to the harbor should be stopped and the that the major problem for the harbor was the combined sewer overflows (CSOs). These overflows derive from about 90 sources on the perimeter of the harbor and result from the collection of storm water and sewage in the same pipes throughout much of the older inner city area. Raw sewage is discharged from these sources during wet weather when the treatment plant capacity is exceeded. These discharges of raw sewage occur about 60 times a year. It is generally agreed that the combined sewer overflows are responsible for the frequent closing of the shellfishing and bathing areas within the harbor.

The only positive thing to be said about the water quality situation in Boston Harbor is that the depletion of dissolved oxygen has never been a problem, except near the shoreline. Therefore, it was natural that the Metropolitan District Commission (MDC), the state agency responsible for managing the treatment of wastewater in the Boston area, should apply for a waiver of the secondary treatment requirement so that it could focus clean-up efforts on stopping sludge discharges and the combined sewer overflows.

The Waiver Process

In response to the 1977 Congressional directive, the EPA published preliminary criteria and procedures in the spring of 1978 and final guidelines in the summer of 1979 by which municipalities could apply for waivers of the secondary treatment requirement for discharges into coastal waters.[4] The EPA guidelines stated that applicants "will bear a particularly heavy burden in demonstrating to the EPA that such (less-than-secondary) treatment is sufficient to protect marine waters."[1] Despite the strong language against the waiver process, many communities applied. Boston's application was one of 70 filed prior to the 1980 deadline. Subsequently, the deadline was extended to the end of 1982 and 137 additional applications were filed. In addition, the EPA refused to allow applicants to compare the environmental impacts of less-than-secondary and secondary effluents through the same outfall, thus contradicting an accepted principle of environmental impact analysis. That incremental benefits of secondary treatment might be negligible, or might be unjustifiably costly, was of no apparent interest to the EPA as reflected in the guidelines.

Boston's Waiver Plan

In 1975, well before the federal waiver amendment, the MDC had concluded that the benefits of secondary treatment were minimal and proposed a three-part plan:[5,6]

- an ocean outfall seven miles offshore of Deer Island for the discharge of primary treatment effluent;
- the cessation of ocean sludge discharge; and
- combined sewer overflow controls.

In 1976, while in the process of setting priorities, the MDC determined that providing secondary treatment ranked 42nd in a list of 52 projects the agency was considering that would be required to improve Boston Harbor.[7]

These earlier studies became the basis for the waiver application to the EPA in the fall of 1979. The MDC proposed upgrading the existing primary treatment plants, which were severely deteriorated, and constructing a combined ocean outfall and multi-port diffuser that would have a total length of nine miles and that would terminate in water more than 100 feet in depth. Boston was required to demonstrate that their waiver plan would meet state water quality standards for marine waters. The major standard was that dissolved oxygen not fall below 6 parts per million (ppm), which is approximately 80 percent of its saturation value under summer temperature conditions.

The estimated cost of these proposed facilities, including sludge disposal on land, was $480 million in 1979. The cost of the nine-mile ocean outfall and diffuser accounted for approximately half of the total costs for the project. At that time, the EPA was funding 75 percent of the capital costs for clean-up projects. The MDC settled back to await the EPA's verdict, having no inkling that a decision on the waiver would drag on for the next five-and-one-half years.

The EPA's Response

The EPA, overwhelmed by a mountain of waiver applications, hired a consulting firm to assist in the review process. In middle of 1981, the EPA requested additional information from the MDC. Among the data they requested was a sensitivity analysis of the water quality model, additional sampling, and assessments of sediment deposition and resuspension. The MDC responded in the fall of 1982 with new monitoring data and the analyses requested.

In the summer of 1983, the EPA issued a tentative denial of Boston's waiver application. The denial focused on potential violations of the state dissolved oxygen standard and excessive solids deposition. However, the EPA left the door open by stipulating that the MDC could submit a revised application by July 1984. There was a significant amount of interaction between the two agencies on the nature of new information to be submitted. The revised application was submitted and six months later the EPA's consultant issued its technical review.

One point of contention between the MDC and the EPA's consultant was the proper value of the background or ambient dissolved oxygen in the vicinity of the nine-mile outfall. The MDC asserted 7.4 ppm was reasonable for late summer conditions when dissolved oxygen was observed to be at its minimum (p. 151).[8] The consultant recommended a more stringent value of 6.5 ppm, recalculated the dissolved oxygen impacts in four separate analyses that had been previously performed by the MDC and concluded that "the Massachusetts dissolved oxygen standard will be met" (p. 158).[8] The MDC assumed that the major issue had been resolved and awaited its waiver. Its optimism was short-lived. In March 1985, the EPA regional administrator issued a "tentative decision" that the revised waiver application be denied.[9]

The EPA's denial was based on seven findings. Six of these findings were non-quantitative or procedural in nature and consisted of items such as deficiencies in the monitoring program to assess future impacts and future source control programs to reduce toxics. The single quantitative finding reversed the conclusion of the EPA's consultant in one of the four impact analyses carried out to check the state's 6.0 ppm dissolved oxygen standard (p. 4).[9] This analysis involved the calculation of a dissolved oxygen change due to the resuspension in a storm event of organic particles deposited on the bottom after 90 days of uninterrupted deposition. The EPA calculated a dissolved oxygen concentration of 5.5 ppm (a violation of 0.5 ppm) by means of two "adjustments." The first involved an arbitrary one-third increase in the rate of organic sediment accumulation in the vicinity of the outfall diffuser. The second and more serious adjustment reduced the ambient dissolved oxygen concentration for the resuspension event from 6.5 to 6.1 ppm (p. 17),[9] a value only 0.1 ppm above the standard, which even the effluent from a secondary treatment plant would have violated. The EPA made no effort to defend these adjustments in this critical instance, yet the agency proceeded to rest its case for secondary treatment on them. There were grounds for challenging the EPA's tentative denial, but other events had by this time removed the MDC as the responsible state agency.

New Agency, New Plans

Early in 1983, the city of Quincy, on the southern portion of Boston Harbor, sued the MDC for polluting its beaches. The case was heard by State Supreme Court Judge Paul Garrity who appointed Professor Charles Haar of the Harvard Law School as special master. Haar's report adopted by the court in the fall of 1983 included:[10]

- a strict time-table for stipulated remediation measures which, incidentally, did not include secondary treatment; and
- recommended the creation of a new state agency with the power to issue bonds outside the control of the state legislature.

After a year of the MDC's failure to meet Haar's schedule, Judge Garrity threatened to stop sewer connections for new buildings in Boston. In the last hours of 1984, the legislature created the Massachusetts Water Resources Authority (MWRA) and gave it bonding authority. In January 1985, the EPA sued the MWRA in Federal District Court for polluting

the harbor. Under threat of a huge retroactive fine, the MWRA has been operating under a 1986 federal court-ordered planning and construction schedule that was designed to carry out the EPA's insistence on full secondary treatment by 1999. The MWRA made the judgement that any attempt to reopen the waiver issue was doomed to failure.

Determining the Best Course of Action

It is easy to look back over the past ten years and to say what should have been done to clean up Boston Harbor. The existing primary treatment plants are beyond rehabilitation and, in fact, never performed satisfactorily. Design and construction of new state-of-the-art primary plants and land-based facilities for the disposal of the primary sludge should have begun in 1979. No one ever questioned the need or priorities for these facilities regardless of the waiver decision. Work would have begun in 1979 but for the fact that the EPA would not approve an application for a federal grant while a ruling on the waiver application was pending.

In retrospect, the most serious flaw in the waiver process was the EPA's refusal to consider a comparison of the environmental impacts of primary treatment effluents and secondary treatment effluents through the same outfall. As a result, the incremental environmental benefits of secondary treatment for the harbor were never balanced against the negative environmental impacts of disposing of twice as much sludge on land or by incineration in the air. Through a fortuitous set of circumstances in the spring of 1988, the data necessary for such a comparison became available.

In March 1988, the MWRA published a comprehensive primary and secondary treatment facilities plan.[11] A few weeks later the EPA issued a draft Environmental Impact Statement (EIS) for Boston Harbor based on this plan.[12] The astonishing thing about the 1988 facilities plan is that the length of the outfall and diffuser, about nine miles, is the same as in the 1979 waiver plan. So it is now planned to discharge secondary effluent at essentially the same location as was originally suggested for the primary effluent. If constructed, it would be the longest outfall and diffuser ever designed specifically for secondary treatment effluent.

According to the court-ordered schedule, the new primary plant and the outfall and diffuser are to be completed by 1995 and the secondary treatment stage by 1999. Because of the five-year construction lag, the MWRA and the EPA were required to predict water quality conditions for primary as well as primary plus secondary effluents. Separate consulting firms were employed to carry out the technical analyses for the two agencies.

The MWRA facilities report provides a detailed analysis of all state and federal water quality criteria for conventional as well as hazardous wastes.[11] The MWRA concluded that all of the standards, including dissolved oxygen, would be met by the primary effluent and that, in general, "secondary treatment discharge impacts are not expected to be significantly different from primary impacts" (p. 8-34).[13] Despite these findings, the MWRA again made no effort to reopen the issue of the marginal benefit of secondary treatment. When this course of inaction was explicitly pointed out during the public comment period on the facilities plan, the MWRA response was that "it is not necessary to justify secondary treatment in the Facilities Plan as the MWRA is mandated by Federal law and court agreements to provide this level of treatment" (p. 7).[14]

The EPA performed separate impact analyses of the MWRA facilities plan for primary and secondary effluents through the proposed nine-mile outfall and diffuser. Noncompliance with standards was cited in three instances for primary effluent. The first was based on a predicted dissolved oxygen violation of 5.7 ppm during a 90-day sediment resuspension event. In contrast to the 1985 waiver denial based on a similar event, the EPA now assumed the ambient dissolved oxygen at 6.5 ppm, thereby backing away from its earlier ambient level of 6.1 ppm (p. 5-22).[15] A search was made for reasons why the EPA predicted a dissolved oxygen violation while the MWRA's analysis of the same event did not. It was found that the EPA calculated a sixfold increase (compared to the MWRA) in the average rates of sediment deposition in the ocean area sur-

rounding the multi-port diffuser. The calculations were based on the EPA's determination that a small fraction of sediment in the primary effluent would be in the size range that has a fall velocity of 0.1 cm/sec (p. 5-5).[15] On the other hand, the MWRA determined that the maximum fall velocity of sediment in the effluent would be 0.01 cm/sec (p. 3-63).[13] A calculation of the sediment removal capability of the new primary treatment plant indicated that all particles having a fall velocity of 0.1 cm/sec would be removed in the primary treatment process even when the primary plant was operated at its maximum capacity of 1.2 billion gallons per day, which is two and a half times its average flow rate. The EPA's use of the larger size fraction in the effluent resulted in higher calculated rates of sediment deposition, and, consequently, an overestimation of the dissolved oxygen depression due to resuspension.

The second point in which the EPA indicated that primary effluent impacts were less satisfactory than secondary was in the areal extent of bottom sediment enrichment and toxicity. Again, this contention was the result of overestimating sediment deposition rates.

The third point — aquatic life criteria — is based on a screening of toxic effects of more than 50 non-conventional pollutants. Of these pollutants, the only violations indicated were for mercury and three compounds (two pesticides and PCB) not presently detected in the inflow to Boston's treatment plants (pp. A-41 & A-85).[16]

These issues were pointed out during the public comment period and the EPA's responses were given in the Final Environmental Impact Statement (FEIS) of July 1988 (p. 3-45).[17] On the major non-compliance issue that revolved around the disagreement on sedimentation rates, the EPA acknowledged that the faster settling sizes would be removed in the primary treatment process. However, the EPA justified retaining the high settling rate in order to "account for potential aggregation of the effluent particles in the marine waters, which would cause the aggregate particles to fall faster" (p. 3-45).[17] The suspended solids concentration in the primary effluent is only about 50 ppm before undergoing a hundred-fold reduction in concentration through the multi-port diffuser. Even at the undiluted value, there is no scientific basis for assuming that aggregation is effective at such low particle concentrations. The only evidence for aggregation in marine waters is in the discharge of sludge from outfalls where particle concentrations are more than a thousand parts per million.

The EPA summed up the level of treatment issue in its FEIS:

> "Some commentors questioned the need for secondary treatment, particularly for a discharge as far off-shore and as deep as the outfall location recommended. The suggestion was made that EPA was over-conservative in its analysis, and that money required to construct and operate the new secondary treatment facilities could better be used to address other pollution problems such as discharges of raw sewage from combined sewer overflows" (p. 3-52).[17]

The EPA continued by saying that because the waiver was denied and the MWRA is now committed to secondary treatment:

> "[T]he need for secondary treatment of the MWRA wastewater was not a question addressed by this FEIS, and a comparison of the impacts of primary effluent versus secondary effluent is not required" (p. 3-52).[17]

The assumptions underlying such statements could be viewed as running counter to the spirit of environmentalism that led to the creation of the EPA and to the concept of environmental impact analysis. The EPA's "tentative" waiver denial of 1985 has become incontrovertible law. These actions reflect an abuse of regulatory authority through a process of circular reasoning. Because the waiver was denied, no further information can be considered that might indicate that the basis for its denial was flawed.

Priorities

A vitally important component has not been included in the present federal court construction schedule for the harbor cleanup — namely, the combined sewer overflow control facilities. Preliminary plans have indicated that

more than 20 miles of deep-rock tunnels, 25 feet in diameter will be needed to store wet weather sewer flows so they can be fed into the treatment system in subsequent dry periods. The cost, certain to be a billion dollars or more, will ultimately be added to the $6.1 billion price tag for already-scheduled facilities. While the MWRA has accepted responsibility for the combined sewer overflow problem, its placement in the construction schedule will not be resolved until after final combined sewer overflow plans are submitted in mid 1990. It is very apparent that the MWRA's financial and management capabilities will be stretched to the limit to complete the secondary treatment facilities plan by 1999 and that combined sewer overflow construction will probably extend well into the next century. Without some means of handling the combined sewer overflows, the $6 billion plan, when completed, will not make Boston harbor fishable or swimmable and will be a rude shock to ratepayers in the Boston area.

The cost of the secondary treatment and secondary sludge disposal facilities in the present schedule is about $2.5 billion. This cost is a very high price to pay for the marginal environmental benefits of secondary treatment, especially when the negative environmental impacts of disposing of twice as much sludge that would result from such treatment have yet to be evaluated. The EPA's claimed benefits of secondary treatment relate to the removal of suspended solids rather than to the purpose for which it was designed — *i.e.*, the removal of soluble organic material (BOD).

An innovative treatment process capable of providing levels of suspended solids removal comparable to secondary treatment (but without high BOD removal and increased sludge production) has not been considered in any of the MWRA's post-waiver planning. This process, known as advanced primary treatment, consists of adding very small amounts of polymer chemicals to primary treatment tanks in order to cause the aggregation of particles and increased sedimentation. Los Angeles County, Orange County and the City of San Diego have successfully used advanced primary treatment for more than 10 years. Advanced primary treatment has achieved 80 percent suspended solids removal with only a 30 percent increase in sludge production over conventional primary treatment, as compared to a 100 percent increase in sludge production with secondary treatment. Orange County received a waiver of the full secondary treatment requirement from the EPA in 1985. Los Angeles County's waiver application is still pending. However, they are expecting a favorable decision that will allow them to continue this sensible practice. San Diego's waiver application has a checkered history. It was tentatively approved in 1981. However, in 1986 the EPA announced a reversal of its decision with an option for the city to submit a revised application. In 1987, in response to public pressure, San Diego decided not to resubmit.

A logical set of priorities for Boston Harbor would follow the current schedule through the completion by 1995 of the new primary treatment plant, its sludge disposal facility, and the nine-mile ocean outfall and diffuser. At that time, the cleanup effort should be directed away from secondary treatment in favor of facilities for the control of the combined sewer overflows. Upon completion of the new primary plant and the ocean outfall and combined sewer overflow remediation measures, an intensive monitoring program in Boston Harbor should indicate whether additional treatment is necessary. If so, the sensible step would be to implement advanced primary treatment.

Fortunately, during the next six years, there is time to bring scientific and political pressure to force the priority issue through the new EPA administration and Congress. When the EPA was footing three-quarters of the bill and threatening massive retroactive fines, there was little incentive to argue. Now there is every reason to insist that local funds be used to achieve an optimal environmental solution rather than one that adheres to a viewpoint more attuned to regulations than results.

NOTE — *Page notations are included in parentheses in the text for the reference cited.*

ACKNOWLEDGEMENTS — *The author acknowledges the support and comments provided by fellow members of the John R. Freeman Fund Committee of BSCES: Lee Marc G. Wolman, Chairman; Harry*

L. Kinsell; Edward B. Kinner; George E. Hecker; and Jonathan A. French. This article is an extension of remarks made at the 1988 BSCES Freeman Lecture Symposium on "Boston Harbor: Engineering and Technical Issues," which was held on April 7, 1988.

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REFERENCES

1. Black & Veatch, "Residuals Management Facilities Plan, Draft: Characterization of Residuals, Supplemental Report No. 1 to MWRA," January 1988.

2. Camp, Dresser & McKee, "Improvements to the Boston Main Drainage System: Report to the City of Boston," September 1967.

3. Horsefield, D.R., "The Deep Tunnel Plan for the Boston Area," *Journal of the Boston Society of Civil Engineers*, Vol. 55, No. 4, October 1968.

4. *Federal Register*, Volume 44(117):34797, June 15, 1979.

5. MDC, "Wastewater Engineering and Management Plan for Boston Harbor-Eastern Massachusetts Metropolitan Area," (EMMA Study) 1975.

6. Resources Analysis, "Scientific and Technical Evaluation of the Environmental Protection Agency's Wastewater Facilities Planning: Boston Case Study, Phase I, Water Quality Considerations," July 1976.

7. Metcalf & Eddy, "Wastewater Engineering and Management Plan for Boston Harbor-Eastern Massachusetts Metropolitan Area," March 1976.

8. Tetra Tech, Inc., "Technical Review of Boston's Deer Island and Nut Island Sewage Treatment Plants Section 301(h) Application for Modification of Secondary Treatment Requirements for Discharge into Marine Waters," Bellevue, Washington, 1984.

9. EPA, "Tentative Decision of the Regional Administrator on the Revised Application (of MDC Boston)," Boston, MA, March 29, 1985.

10. Commonwealth of Massachusetts, "Report of the Special Master Regarding Findings of Fact and Proposed Remedies," *Superior Court Civil Action No. 138477, City of Quincy v. Metropolitan District Commission et al.*, August 1983.

11. MWRA, "Secondary Treatment Facilities Plan, EIR," March 31, 1988.

12. EPA, "Boston Harbor Wastewater Conveyance System, Draft Supplemental Environmental Impact Statement," April 1, 1988.

13. MWRA, "Secondary Treatment Facilities Plan, EIR, Volume V, Effluent Outfall: Final Report," Boston, MA, March 31, 1988.

14. MWRA, "G.L.C 30 Section 61, Findings by the MWRA on the Final Secondary Treatment Facilities Plan for Boston Harbor, V, Appendix: Response to Comments," Boston, MA, December 15, 1988.

15. EPA, "Boston Harbor Wastewater Conveyance System; Draft Supplemental Environmental Impact Statement, Volume I," Boston, MA, 1988.

16. EPA, "Boston Harbor Wastewater Conveyance System; Draft Supplemental Environmental Impact Statement, Volume II," Boston, MA, 1988.

17. EPA, "Boston Harbor Wastewater Conveyance System; Final Supplemental Environmental Impact Statement," Boston, MA, 1988.

The History of Boston: The Impact of Geology

Boston's location and geology suited settlement and expansion into a major city.

DAVID WOODHOUSE

Boston is situated on Massachusetts Bay, a large body of water bounded by Cape Ann to the north and Cape Cod to the south (about latitude 42° 21′). Boston is the oldest city in the United States (Florida claims that St. Augustine, settled in 1546, is the oldest permanent European settlement). Boston occupies the smallest land mass of any major city and sits in its sheltered bay with snug harbor and 38 islands that act as a buttress to the Atlantic Ocean (the Pilgrims counted at least 50 islands in 1621). Affectionately called the Athens of America, the Hub, God's Own Capitol, and plain old Bean Town, Boston serves as the capital of Massachusetts and is the largest city in New England.

History of Founding

As one might expect for such an old city, the literature of its history and founding is voluminous. In particular, Winsor, Whitehill, Kaye and, most recently, the *Boston Globe* have provided comprehensive and interesting looks into the city's history.[1,2,3,4]

The earliest settlers of Boston were the Indians in about 2500 B.C. In about the year 990 the Norseman Biarne visited the area and many early traders and explorers came here during the 16th and 17th centuries. Captain John Smith first mapped the Boston shoreline in 1614. Captain Miles Standish with his small group of Pilgrims and Indian guides entered Boston Harbor in a shallop, a large open sailboat, on September 19, 1621, having come up along the shore from Plymouth.

The actual credit for the founding of Boston can be given to John Winthrop and his band of Puritan followers representing the Massachusetts Bay Company. On March 22, 1630, they set sail on the Arbella, Ambrose, Talbot and the Jewell from the exact spot the Pilgrims had departed from, down the 10-mile estuary of Southampton, England, to Cowes on the northernmost tip of the Isle of Wight. The Puritans anchored at Cowes until sailing conditions were favorable. Eventually, as a passenger on the Arbella named Thomas Dudley recorded, a total of 17 ships with 1,000 passengers assembled to make the journey to New England. On April 8, 1630, Winthrop set sail on the Arbella, along with nine other ships, some of which were bound for Newfoundland, and headed on to their perilous journey to escape religious persecution and to found a "city upon a hill." The settlement would be founded under the sponsorship of the Massachusetts Bay Company and its duly elected governor, John

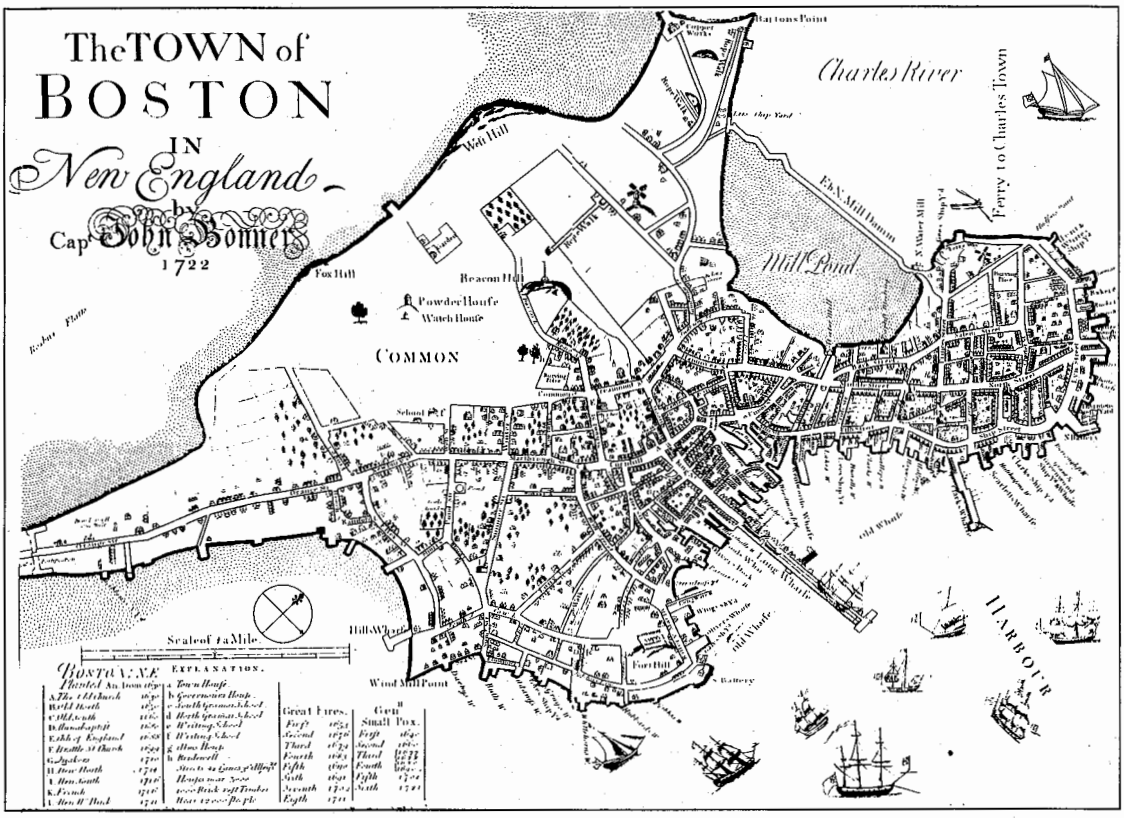


FIGURE 1. Map of Boston on Shawmut Peninsula in 1722, by John Bonner.

Winthrop. Their destination was Salem, about 25 kilometers (15 miles) north of Boston, where more than 100 English settlers had been living for almost one year.

On June 12, 1630, the hardy group reached Salem and was greeted by a landscape of barren rock and little soil, a brutal contrast to their native, fertile southern England. Pressured by his followers, Winthrop, after spending just five days in Salem, left for Charlestown (now a section of Boston) where a group of Englishmen had already settled. By July 6, 1630, eleven ships had arrived at Salem or Charlestown; the people on board those ships were the survivors of stormy seas and sickness that had decimated their ranks.

Within a couple of weeks, the new settlers at Charlestown were nearly defeated by sickness, death and lack of water. Winthrop blamed his troubles on the one source of water available, a brackish spring that was only exposed at low tide. Their sickness was compounded since the English traditionally drank from springs rather than dug wells. Help came in the person of the hermit William Blaxton (Blackstone), a retired Episcopal minister who had been living across the Charles River on Beacon Hill since 1625.

Blackstone had been the chaplain of the unsuccessful Robert Gorges' Plantation that had settled at Wessagusett (now known as Weymouth) in 1623. Blackstone was the first English settler of what would become Boston (then called the Shawmut Peninsula), having built a cottage on the south side of Beacon Hill near the Common, where Spruce Street now meets Beacon Street (see Figure 1). Blackstone told Winthrop of the abundant and refreshing spring water available on his land. Accepting his invitation, Winthrop and his group moved across the river to the Shawmut Peninsula. Shawmut is the Indian name for "living waters" and, indeed, such living waters did exist on Spring Lane, in Louisburg Square and on Pemberton Hill. The group settled near Spring Lane, the North End and at the Great Cove at the head of what is now State Street.

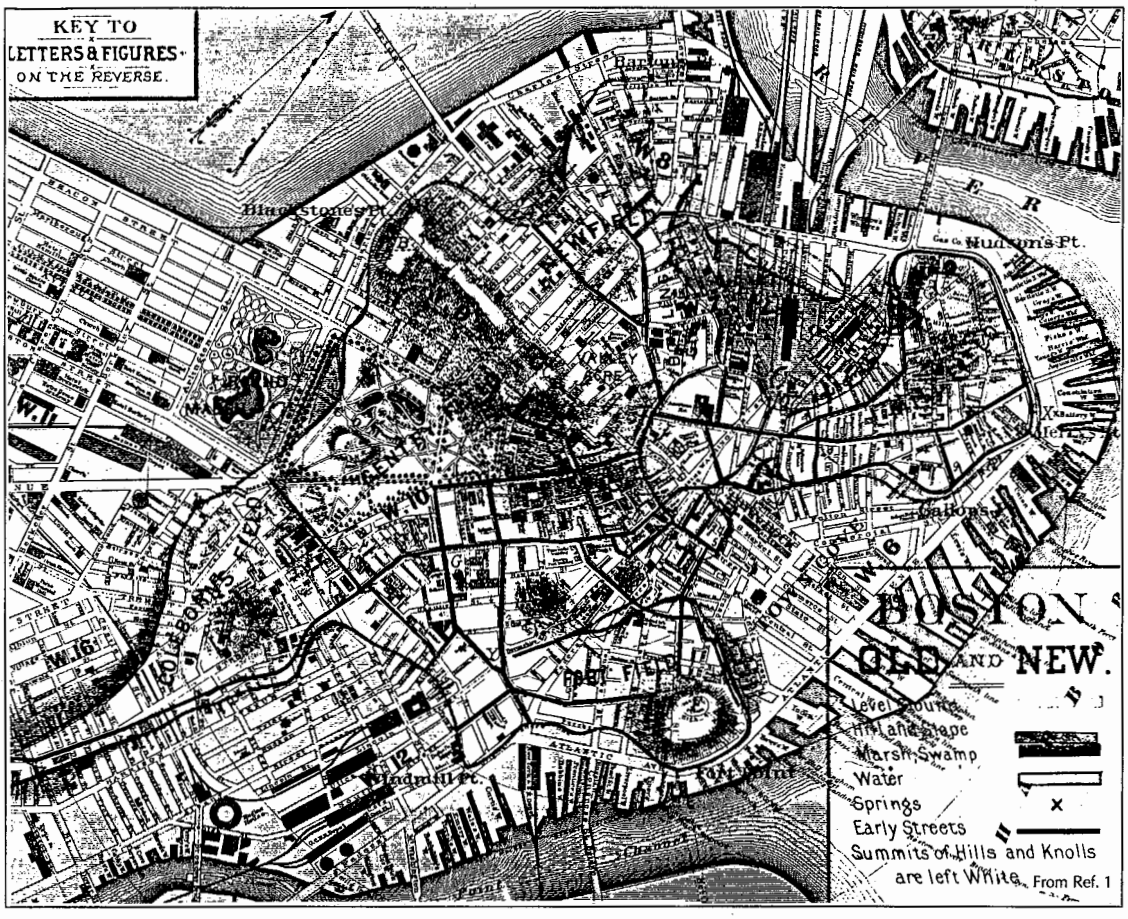


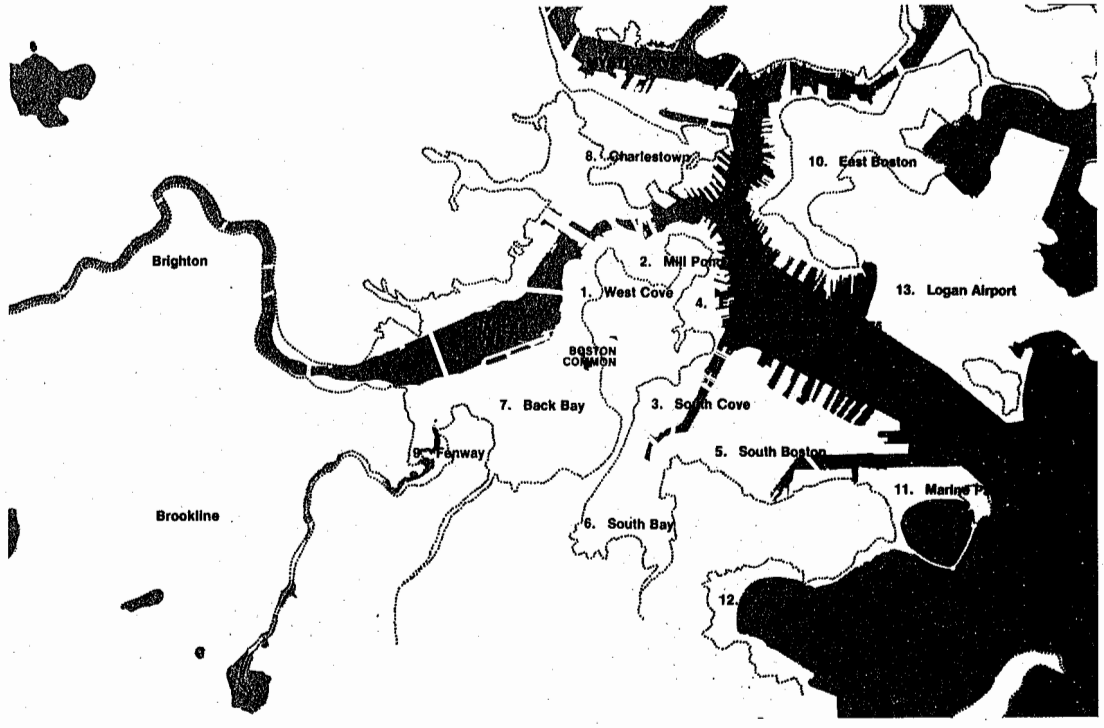
FIGURE 2. Map of the colonial Shawmut Peninsula from the 1700s superimposed on an 1890 map of Boston.

Thus it was that Winthrop had founded his "city on a hill." The "hill" was called the Trimountain (its old spelling was "Trimontaine," and it was sometimes called Tra-mount, Tremount, or Tramountaine, and is now called Tremont), the three prominent hills of Boston (see Figure 2). The westernmost hill was Mt. Vernon (also called West Hill or Copley's Hill), the central peak was Beacon Hill (or Sentry Hill) and the easterly hill was Pemberton Hill (or Cotton Hill). An order of the Court of Assistants, with Winthrop presiding as Governor, proclaimed "[t]hat Trimontaine shall be called Boston," on September 7, 1630 (which later became September 17 after the calendar change of 1752, which added 10 days to the calendar).

Boston was named after Boston, England, which was located in the parish of St. Botolph. The name itself was a contraction of "Botolph's town," after "bot," meaning boat, and "ulph," meaning help. St. Botolph was the patron saint of fishing. Boston's roots were thus established.

Geological Influences Affecting the Founding

There were four major geological influences that affected the founding of Boston. Being a seafaring people, the early settlers looked for a safe harbor. Boston's island-studded harbor is formed by a deep indentation in the coastline of Massachusetts. The indentation exists primarily because the underlying rock of Boston is softer and more easily eroded than the harder, granitic rocks that surround the Boston Basin, as this large topographic depression is called. Glacial ice eroded a valley or depression in these soft rocks; and, with subsequent melting of the ice, the sea level rose and flooded the



Area	Size (acres)	Dates
1. West Cove	80	1803-1863
2. Mill Pond	70	1807-1829
3. South Cove	86	1806-1843
4. East Cove	112	1823-1874
5. South Boston	714	1836-present
6. South Bay	138	1850-present
7. Back Bay	580	1857-1894
8. Charlestown	416	1860-1896
9. Fenway	322	1878-1890
10. East Boston	370	1880-present
11. Marine Park	57	1883-1900
12. Columbus Park	165	1890-1901
13. Logan Airport	150	1922-present

from Ref. 5

FIGURE 3. Map of the Boston metropolitan area showing the history of land filling that expanded the city's original 783 acres to over 3,000 acres today.

depression, thereby forming the harbor.

The second important factor was that the settlers looked to the land for protection from their enemies. Boston was naturally situated for fortifications, with its "Trimountain" rising 46 meters (150 feet) above the surrounding land; Copp's Hill (also called Windmill Hill, from the one built there in August 1632, and Snow Hill) in the North End rising 15 meters (50 feet) above the water; and Fort Hill (also called Corn Hill) rising 24 meters (80 feet). Both Copp's Hill and Fort Hill are situated on points of land facing the sea. Smaller hills or knolls were found, such as Fox Hill and Powderhouse Hill (also called Flagstaff Hill), but probably did not play a significant role as fortifications.

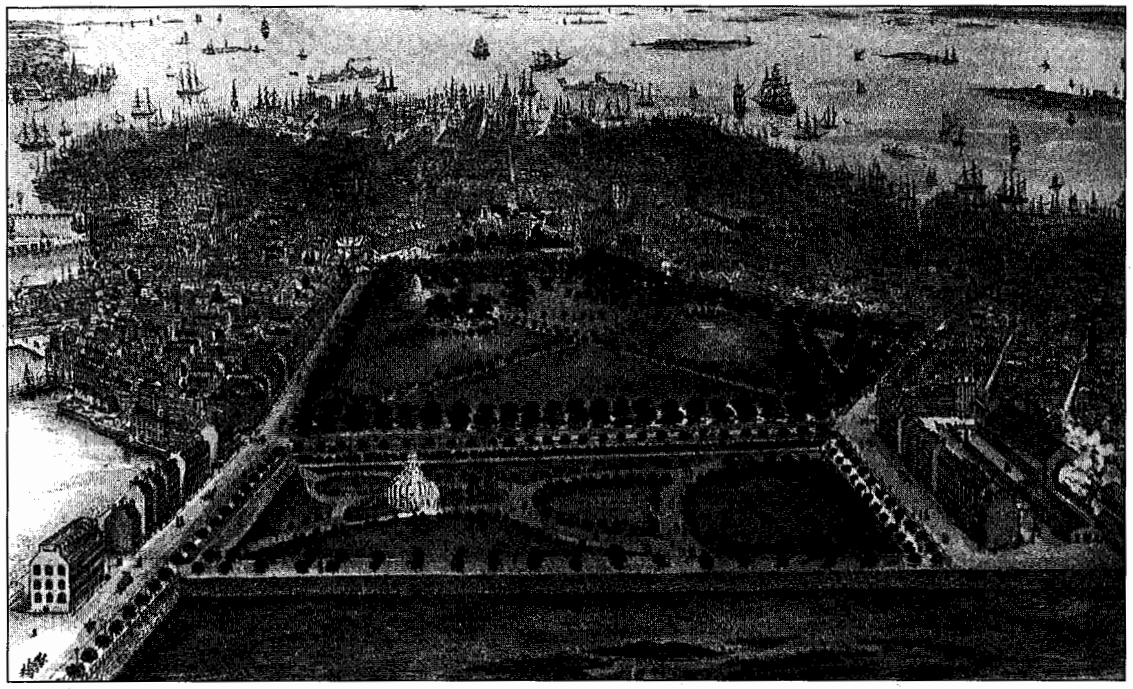


FIGURE 4. A view of Boston prior to the filling of the Back Bay.

Hills around Boston are predominantly drumlins, including many of the harbor islands. However, the "Trimountain," now represented by Beacon Hill, is not a drumlin as originally postulated by geologists. Numerous excavations into the hill have exposed deformed, faulted and folded sediments such as clay, sand and gravel, and till.

Third, the 319-hectare (789-acre) Shawmut Peninsula was an island at high tide. The land to the south, called the Neck, that connected Roxbury and Boston, was very narrow and low, and at best served as a causeway for travelers.

The fourth and last geologic factor, the primary and literally life-giving factor, was the abundant fresh water available to the settlers. The Shawmut Peninsula became the Town of Boston because several good springs were found, and the shallow dug wells that were dug produced water of good quality under artesian pressure. Most of the area is underlain by a sandwich of thick, pervious sand and gravel between lodgement till and marine clay soils.

These factors all contributed to a notable population increase and considerable development of the Shawmut Peninsula during the 17th and 18th centuries (see Figure 3). To reclaim the surrounding marshy lowlands in order to meet the needs of development and expansion, early land developers looked to the three hills of the Trimountain as a ready source of fill.

In 1799, about 15 to 18 meters (50-60 feet) of Mt. Vernon was excavated by the Mt. Vernon proprietors to fill in the cove at its base, thus creating Charles Street. The Mill Pond created in 1643 was the next area attacked in the enthusiasm for increasing the land area of Boston. The central peak of Beacon Hill, in itself a source of gravel since 1758, was lowered by 18 meters (60 feet) by John Hancock's heirs and the pond filled in. The last hill, Pemberton Hill, had its top shaved off by Patrick Tracy Jackson, a railroad man, in 1835 to fill new land north of Causeway Street and develop Pemberton Square. The remaining ridge connecting the former peaks of Pemberton and Beacon Hills was leveled in 1845.

The latter half of the 19th century saw the last major filling as the Back Bay was created from sand and gravel brought in from Needham (a town to the southwest of Boston) by railroad (see Figure 4 for a view of the area before filling). This filling of the bays around

Boston has added character to the city as well as causing engineering problems. It was a factor in social change and created a very peculiar patchwork street network, since each in filled area has its individual street pattern.

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REFERENCES

1. Winsor, J., ed., *The Memorial History of Boston, Including Suffolk County, Massachusetts, 1630-1880*, James R. Osgood & Co., Boston, 4 vols, 1880-1882.

2. Whitehill, W.M., *Boston: A Topographical History*, 2nd ed., Belknap Press of Harvard University, 1973, 249 pp.

3. Kaye, C.A., "The Geology and Early History of the Boston, Massachusetts Area — A Bicentennial Approach," *U.S. Geological Survey Bulletin* 1476, 1976, 78 pp.

4. *Boston Globe*, "The Birth of Boston," March 30, 1980.

5. Boston Society of Architects, *Architecture Boston*, Barre Publishing, Boston, 1976, 182 p.

Geology of the Boston Basin & Vicinity

If the knowledge of the geology of an area is well known, the greater will be the ability to handle engineering and environmental problems with a higher degree of certainty.

PATRICK J. BAROSH, CLIFFORD A. KAYE & DAVID WOODHOUSE

Boston lies in a fault-bounded structural and topographic basin containing a wide variety of strata that have highly irregular stratigraphic and structural relations. Overlying these rocks are glacial and post-glacial soils of such complexity that their study ultimately led to the development of soil mechanics as a science in this country.

The extreme variety in bedrock and surficial deposits, and their structure, makes the region one of the most challenging anywhere for engineering geologists, since nearly every site is different. The growing need to understand more about water movement (both for water supply and hazardous waste management and cleanup), ground conditions for an ever-expanding variety of construction projects and the potential for earthquake hazards requires more detailed knowledge of the geology of the Boston area than ever before. The lack of detailed geologic data in the northeast United States, for example, has resulted in the unnecessary expenditure of millions of dollars and years of delay in the construction of nuclear power plants while seismologists attempted to relate earthquakes to geologic structures on maps that did not contain the critical data.

A virtual explosion of new information on the region has become available over the past 20 years. This new knowledge has radically changed earlier concepts of the regional geology. The data is mainly from mapping by personnel of the Boston Office of the Geologic Division of the U.S. Geological Survey (closed in 1976), investigations of the New England Seismotectonic Study and the work of the Water Resources Division and the Office of Marine Geology of the U.S. Geological Survey. Some excellent work also has been done by engineering geologists and geophysicists working on tunnels and nuclear power plant sites in the region. Much of the geologic literature on the area, unfortunately, reflects this understanding unevenly and is often contradictory. Also, obtaining the new geologic data is not always easy. The recent State Geologic Map of Massachusetts is, unfortunately, more of pictorial than scientific value and contains little practical information.[1] It contains far less information than the larger-scale preliminary compilation for this map.[2] However, most of the quadrangles in eastern Massachusetts have been mapped, although most of these maps are

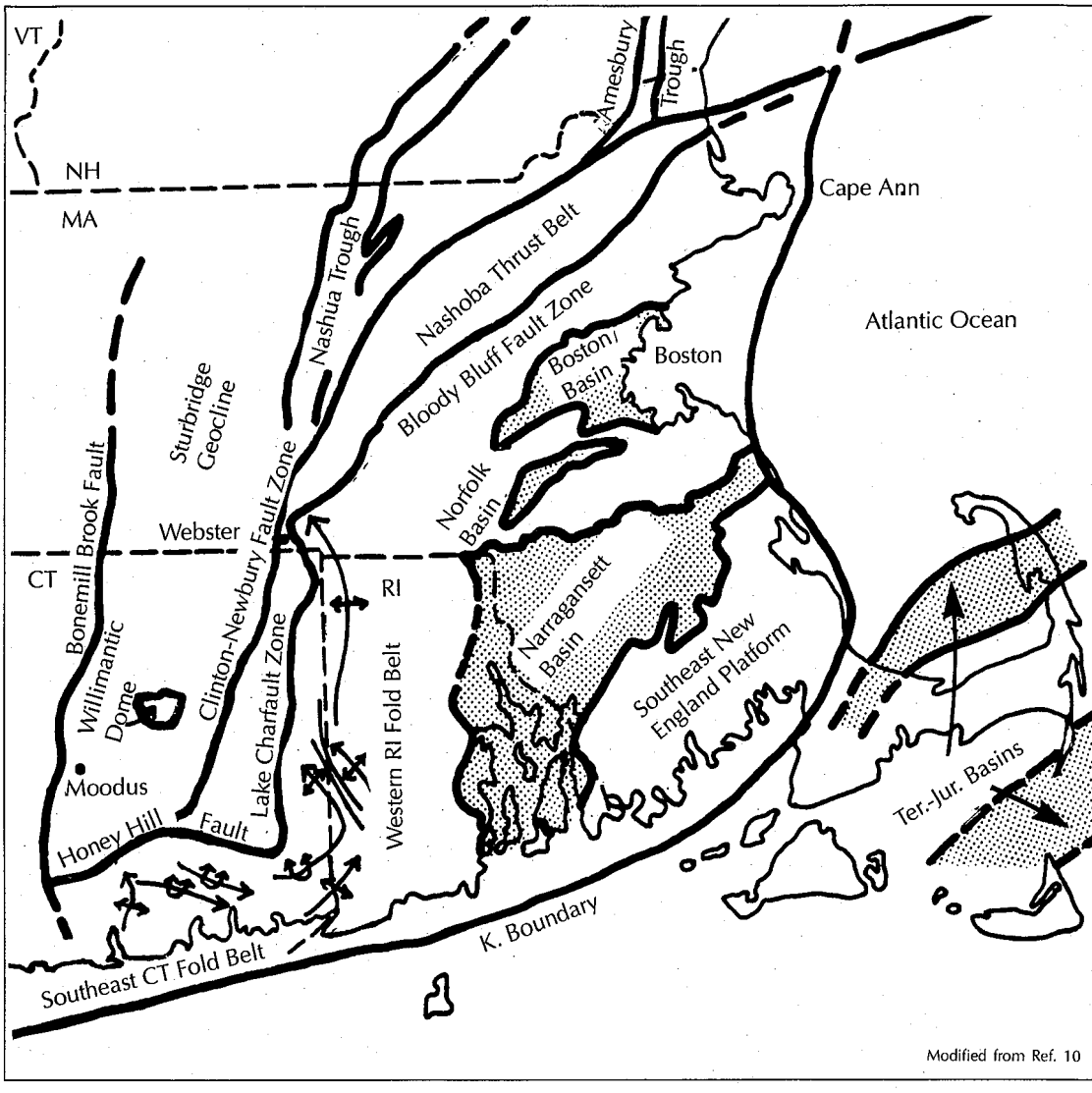


FIGURE 1. Map of southeastern New England showing the major tectonic provinces and structures.

as of yet unpublished and only available in open-file reports. Those that were prepared after 1970 generally have much more structural data, since it took time, preliminary work and excavation of new exposures before many structures were recognized.

Kaye has prepared several reports on Boston summarizing much of this complex geology,[3,4,5,6,7] building on the work of La Forge[8] and has produced a detailed geologic map of central Boston.[9] Information on the regional geology of eastern Massachusetts can be obtained in other recent works.[2,10,11]

Regional Structural Framework & History

Southeastern New England contains some of the most interesting, varied and complex geology in all of North America. It lies astride the eastern border of the Appalachian orogenic belt. This border is the greatest structural zone known in New England. It represents a zone of late Precambrian and early Paleozoic collision between Paleo-North American and Paleo-African plates, and now forms the Nashoba Thrust Belt (see Figure 1). The rifting that later

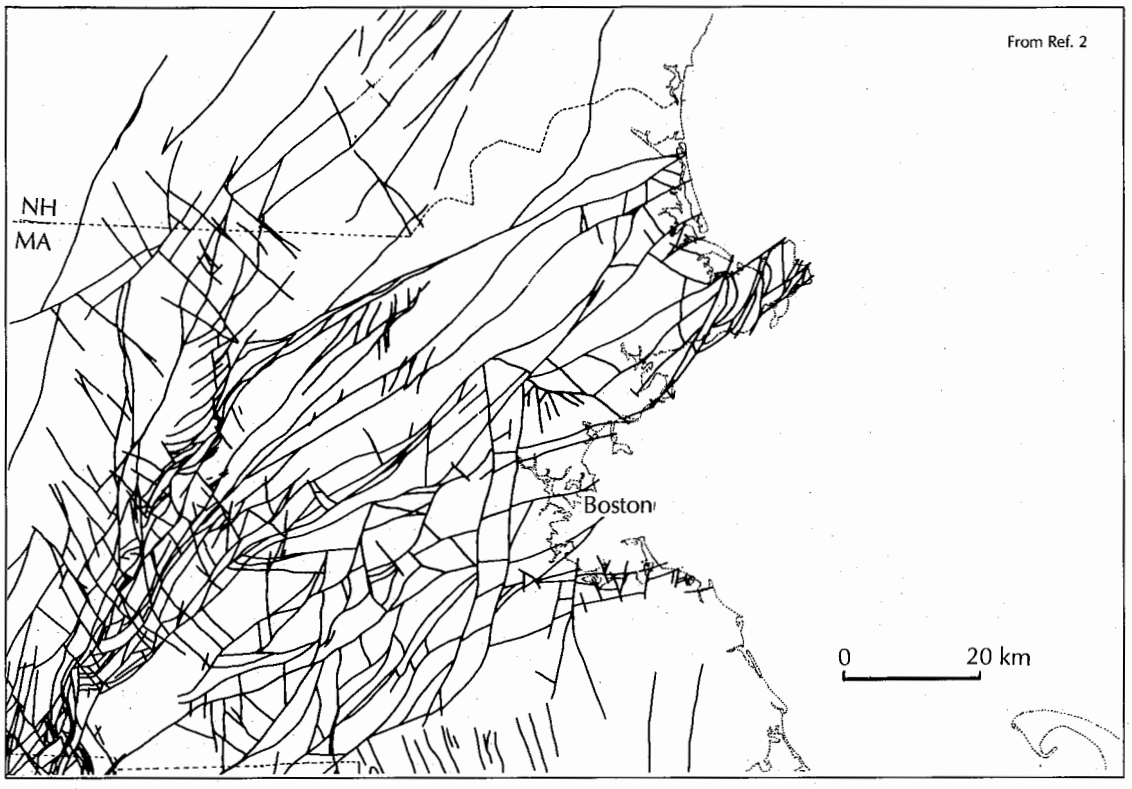


FIGURE 2. Map of eastern Massachusetts and southern New Hampshire showing mapped faults (Boston 2-degree sheet).

formed the present North Atlantic Basin generally followed this collision zone, but split off a piece of northwest Africa that clung to North America and now forms the southeast corner of New England.[10] This fragment became the foundation on which Boston was built. The Boston Basin, as well as other nearby basins, were formed by faulting in this ancient, largely granitic basement (see Figures 2 and 3). Subsequently, the eastern edge of this fragment sagged as the North Atlantic widened and was overlapped by the Cretaceous and Tertiary deposits that lie offshore and form the submerged northern extension of the Atlantic Coastal Plain.[12] Earthquake activity and other indications of crustal movement show that it is still a tectonically active region.

The Boston Basin

Bedrock Geology. The City of Boston is located near the center of the Boston Basin, an east-northeast-trending, wedge-shaped, down-faulted body of sedimentary and volcanic rock (see Figure 4).[7,8,9,13,14,15,16] Onshore, the basin is widest along the coast, where it measures about 24 kilometers (15 miles) north to south.

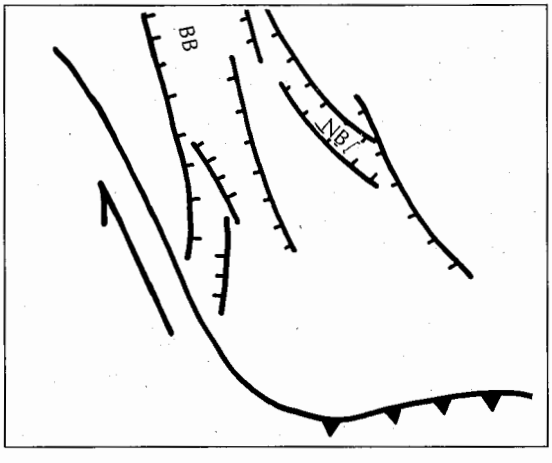


FIGURE 3. Sketch map showing the structural relations of the Boston Basin (BB) and the Norfolk Basin (NB) with the eastern edge of the Nashoba Thrust Belt.

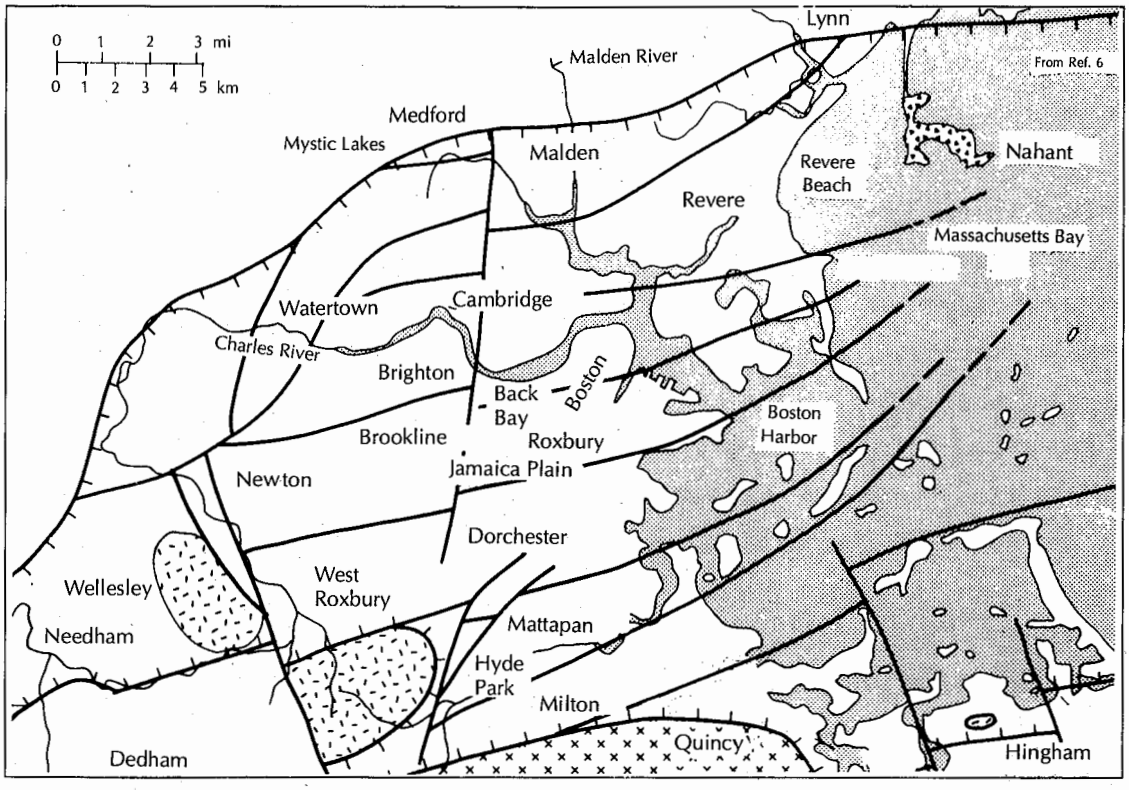


FIGURE 4. Map of Boston showing the limits of the Boston Basin (ticked line), generalized major faults (heavy lines, dashed where inferred) and inliers of plutonic rock (hatched = Dedham granite, crosses = Quincy granite, and triangles = Nahant gabbro).

Offshore, it extends to the east under Massachusetts Bay, where it appears to widen still more. On the west, the basin tapers to a point about 29 kilometers (18 miles) west-south-west of Boston.

The strata in the basin vary in age from the latest Precambrian to Middle or Late Cambrian and perhaps Ordovician.[17] The rocks of the Boston Basin appear to have been deposited in a basin complex that was undergoing active block-faulting. The highest relief and source areas lay to the south and west of the basin. Coarse-grained heterogeneous terrestrial deposits along these borders graded rapidly north and east into fine-grained marine deposits. The sedimentary rocks, therefore, consist of detritus that had been eroded from the surrounding highlands and fault scarps and that had been deposited as interfingering lithofacies. Conglomerate, sandstone, argillite and volcaniclastic sediment grade or interfinger into each other laterally and vertically over short distances. Thin limestones interbedded with argillite and sandstone are locally abundant.

Very late Precambrian volcanic activity was widespread and occurred in at least six intervals. Early eruptions were rhyolitic and later were spilitic and keratophyric. The volcanic rocks occur as flows, flow breccias, explosion breccias, pillow lavas, plugs, necks and diatremes.

Bottom conditions were unstable in the depositional basins. At many stratigraphic levels, the telltale evidence of submarine sliding and turbidity currents is present. This evidence includes convoluted bedding, intraformational breccia, graded-bedding and large lenticular slumped masses of pebbly to bouldery mudstone (mistakenly identified as glacial tillite). Bottom slumps and slides were probably triggered by earthquakes that originated from volcanic eruptions and block faulting.

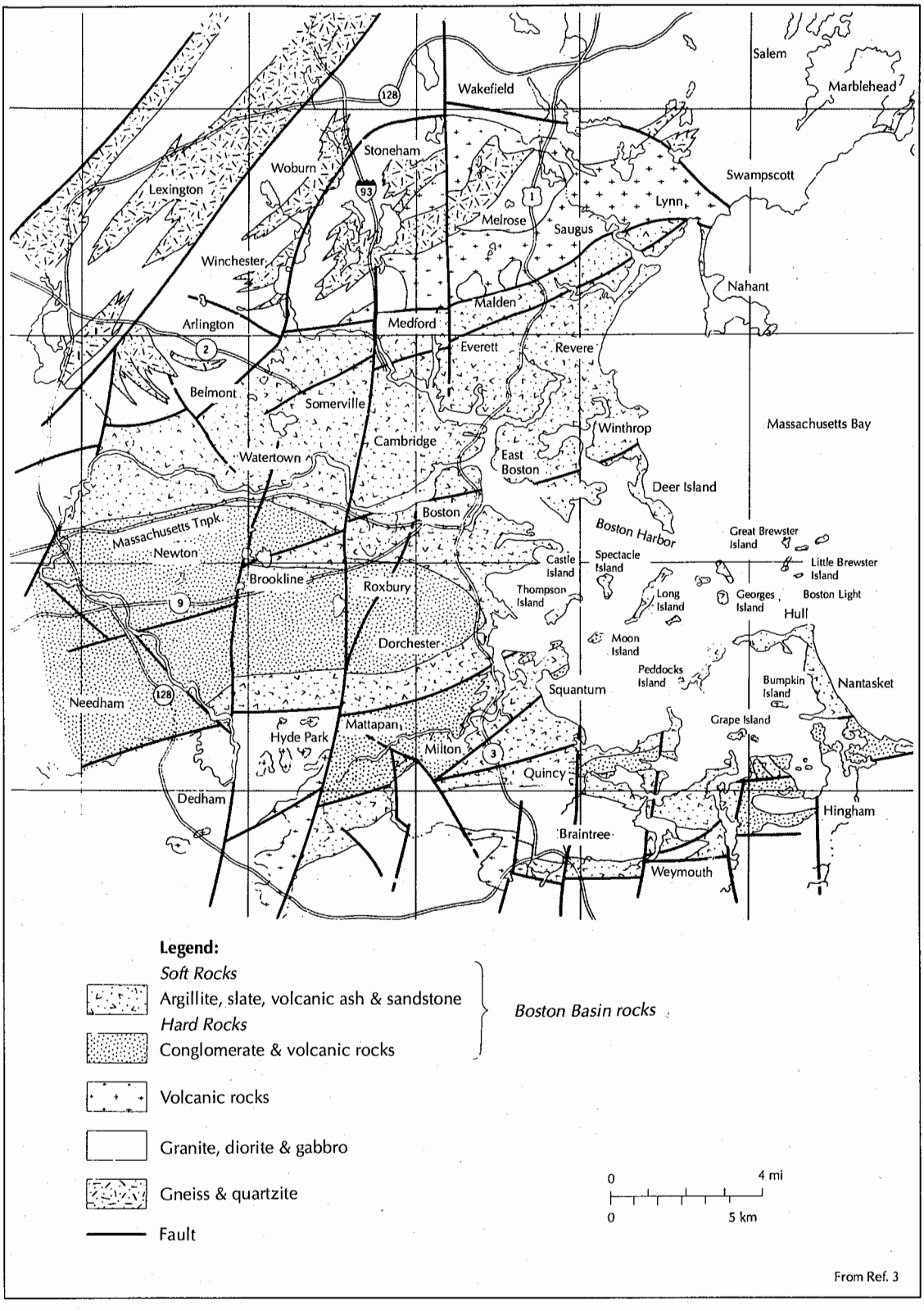


FIGURE 5. Generalized bedrock geologic map of the area surrounding Boston.

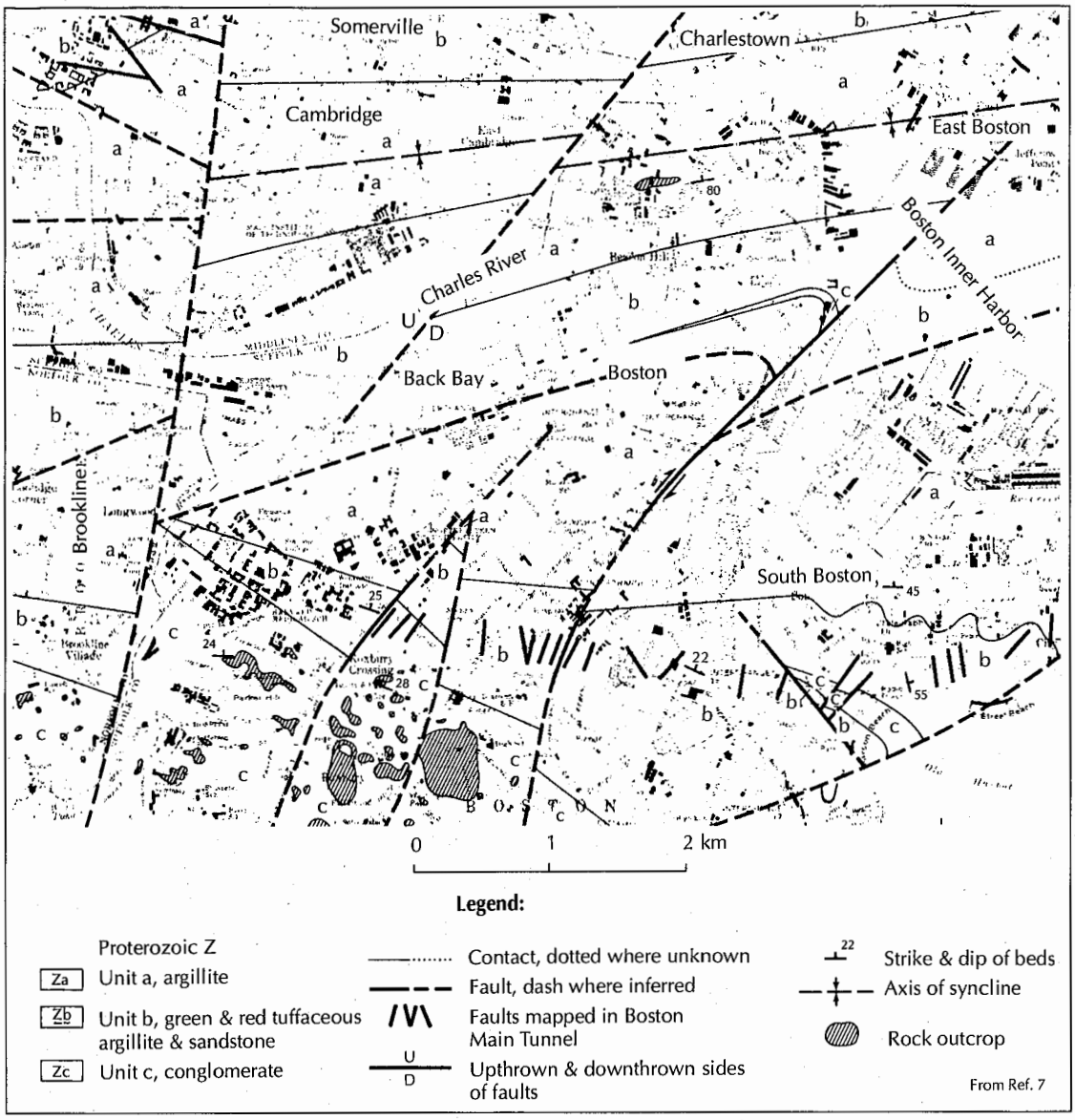


FIGURE 6. Bedrock geologic map of central Boston.

Large volcanic complexes developed on either side of the Boston Basin during the Ordovician period. The remnants of these complexes now comprise the Cape Ann Granite, Salem Gabbro-Diorite and Lynn Volcanics to the north and the Quincy Granite and associated ash-flow and other volcanic rock to the south. This activity probably ended marine deposition in the basin. Rock of this age might also be preserved locally in the basin.

The sedimentary rocks in the basin are divisible into three main facies: coarse-grained (conglomerate and sandstone), fine-grained (argillite) and a mixed facies consisting of maroon and green tuffaceous siltstone and sandstone (see Figure 5). Traditionally, these sedimentary rocks have been called the Boston Bay Group and have been given formational names: Roxbury Conglomerate and the Cambridge Slate, in that order of the previously ascribed Paleozoic age.[8,15]

The lower formation, the Roxbury Conglomerate, was traditionally subdivided into three members which are, in ascending order:

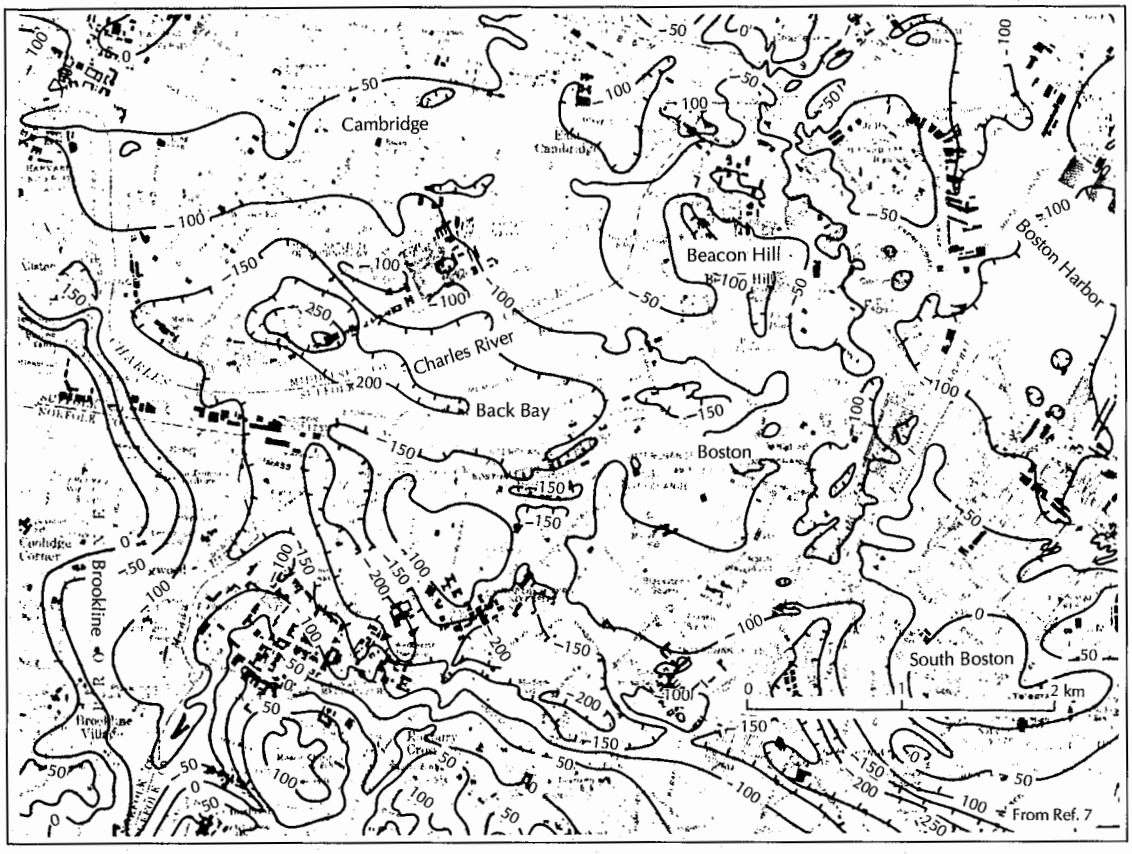


FIGURE 7. Bedrock surface of the central Boston area. The contour interval is 15 m (50 ft). Datum is mean sea level.

the Brookline Conglomerate, the Dorchester Shale, and the Squantum Tillite. The Brookline Conglomerate is comprised of conglomerate, sandstone, argillite, shale, and altered basalts and andesites that are sometimes called the Brighton Metaphyre. The uppermost member, a mudstone with "floating" clasts, has been called a tillite by some,[8,18] but is considered to be a subaqueous flow by more recent workers.[19]

The Cambridge Slate is now called the Cambridge Argillite. Thin quartzites are locally present within it such as the Tufts Quartzite in Medford, on the edge of the basin just north of Boston, and the Milton Quartzite, just south of the city.[14,15]

This simple stratigraphic concept of a layered sequence of decreasing age does not properly portray the complex intertonguing relations of different facies in which the same rock type may have been deposited at different times. The use of these formational terms perpetuates confusion in the basin and they have not been used on recent maps.[9] The assignment of a proper stratigraphic nomenclature awaits further mapping.

The fault boundary of the basin on the south is complex and consists in part of faults that were active during the period of deposition. The much straighter northern boundary fault formed later. The Boston Basin appears to be deformed by compression into a series of long, east-west to east-northeast-trending folds, overturned to the south and, as they can be seen today, plunging east. The southern part of the central Boston area appears to lie on the faulted northern flank of a large east-plunging anticline (see Figure 6).[8,9,14] Cutting across the northern tip of the Boston Peninsula is the east-northeast-trending axis of the large Charles River Syncline,[14] with argillite cropping out in the trough of this long fold. These folds are

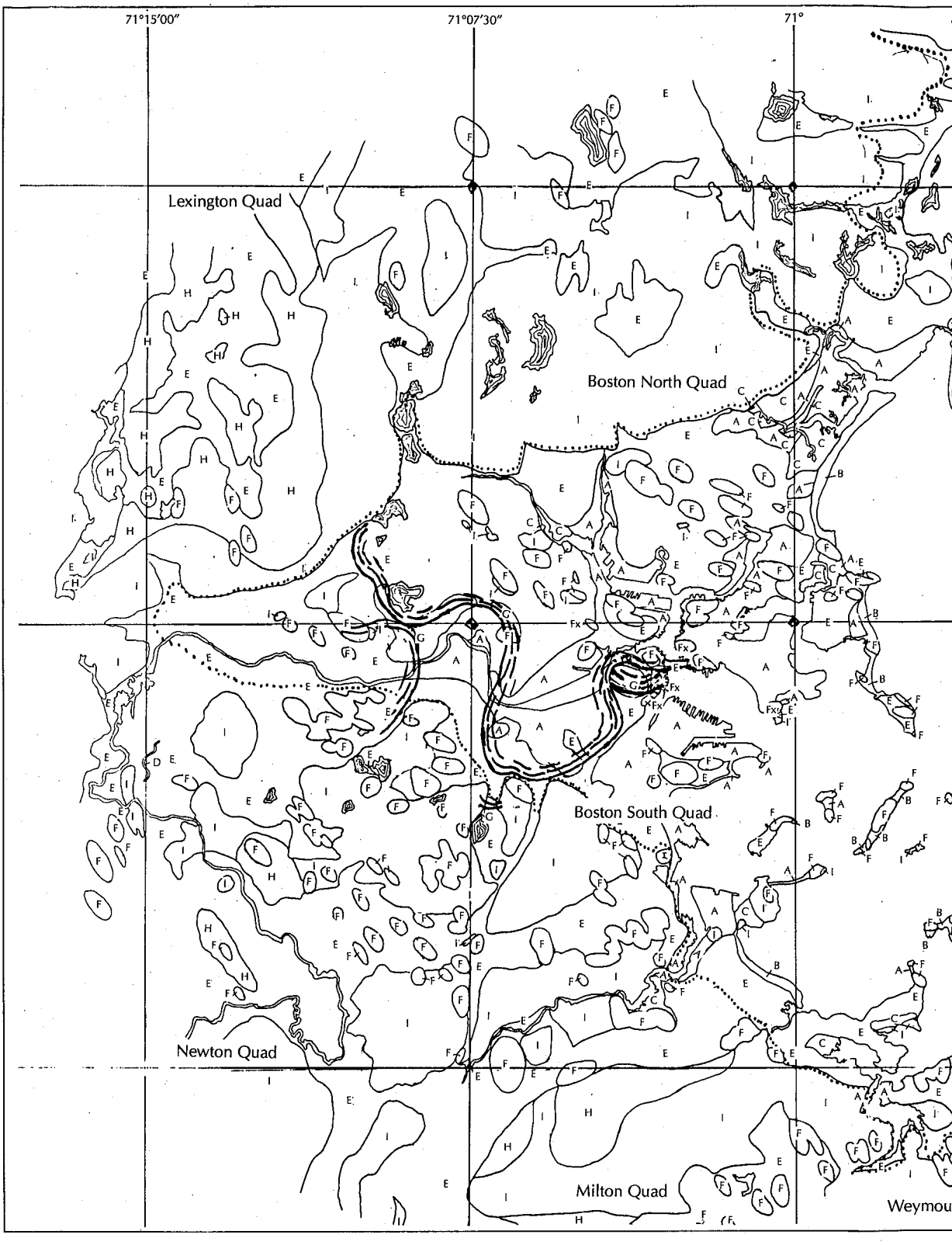
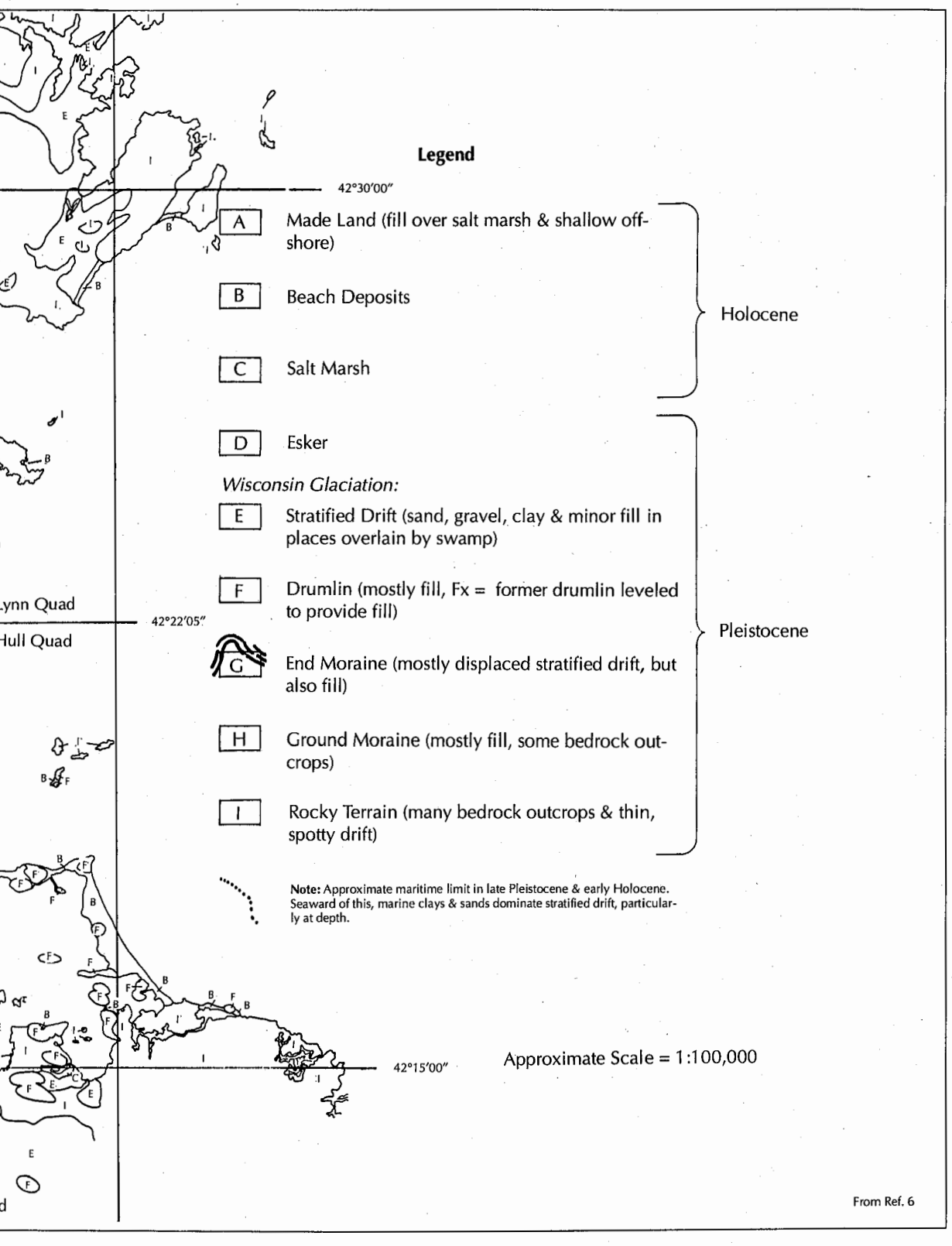


FIGURE 8. Surficial geologic map of the area surrounding Boston.


Legend
42°30′00″
A Made Land (fill over salt marsh & shallow off-shore)
B Beach Deposits
C Salt Marsh
Holocene
D Esker
Wisconsin Glaciation:
E Stratified Drift (sand, gravel, clay & minor fill in places overlain by swamp)
F Drumlin (mostly fill, Fx = former drumlin leveled to provide fill)
G End Moraine (mostly displaced stratified drift, but also fill)
H Ground Moraine (mostly fill, some bedrock out-crops)
I Rocky Terrain (many bedrock outcrops & thin, spotty drift)
Pleistocene
Note: Approximate maritime limit in late Pleistocene & early Holocene. Seaward of this, marine clays & sands dominate stratified drift, particularly at depth.
Lynn Quad
Hull Quad
42°22′05″
42°15′00″
Approximate Scale = 1:100,000

From Ref. 6

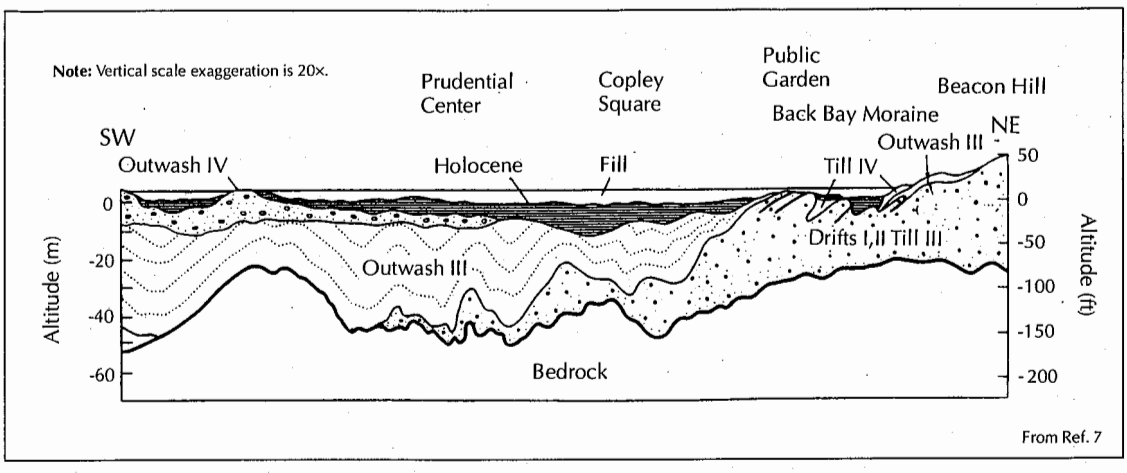


FIGURE 9. Geologic cross-section across part of the Back Bay, extending southwest from Beacon Hill and showing Quaternary deposits.

fragmented by longitudinal faults of large displacement. Recent mapping indicates that there are at least eight large faults, most of which are 15 kilometers (9 miles) or more in length.[9] They break the basin into long, narrow fault blocks, each of which consisting of a single fold, either an anticline, syncline or homocline. In addition, the rocks are broken by a complex of later faults, most of which are transverse to the longitudinal faults. Besides faults with large to small displacement, there are shear zones with various cataclastic effects, but with relatively small displacement. The longitudinal faults are mostly high-angle reverse in nature. Slickensides on fault surfaces show a strong strike-slip component of movement on many

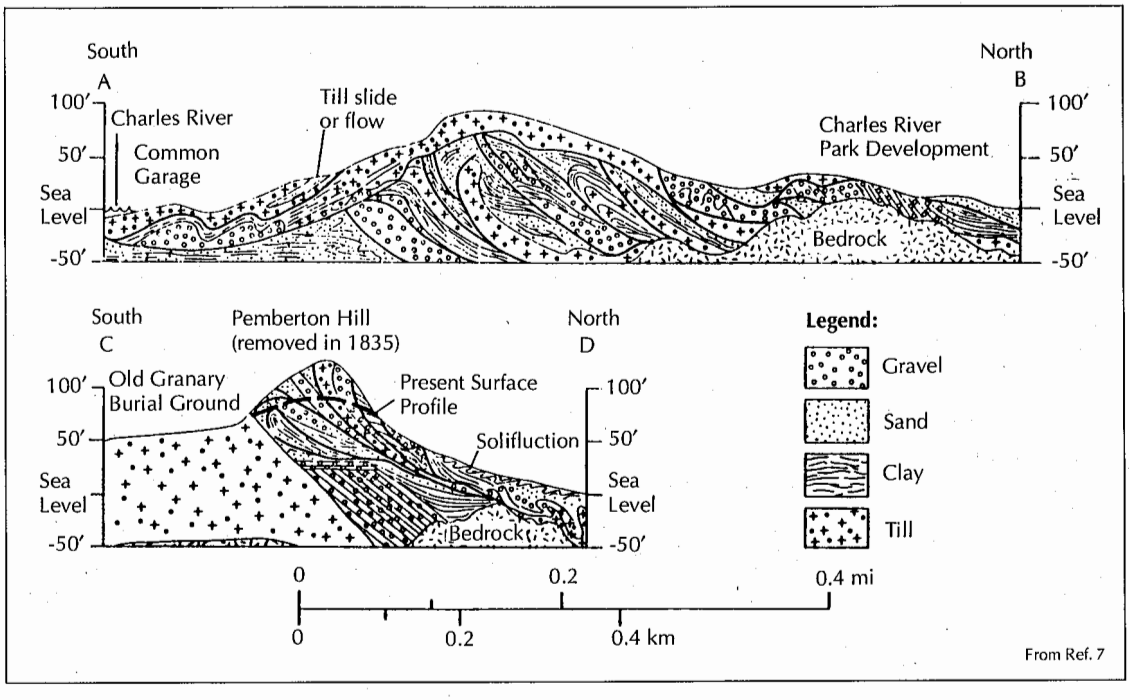


FIGURE 10. Two cross-sections of Beacon Hill showing the type of complexity revealed by deep foundation excavations.

of the transverse faults. There is surprisingly little to no unconsolidated fault breccia or gouge lining many faults. However, there is, in many of the faults, lithified cataclastic material that is difficult to characterize without petrographic study. The present data indicate an average lateral fault-spacing throughout the area of about 150 meters (490 feet) measured in any direction, although the density of faults varies from place to place.

The structural deformation within the Boston Basin probably is the result of latest Precambrian to Ordovician movements with lesser activity from the rest of the Paleozoic, when other nearby basins were formed, as well as faulting and dike-intrusion in the Triassic-Jurassic. The longitudinal faults may represent reactivation of the late Precambrian basin-range faults. The transverse faults are mostly post-Pennsylvanian. Northwest- and north-trending faults are the youngest in the region and some are still active north of the basin.

Bedrock Surface. More than 95 percent of the surface of the bedrock of the central Boston area lies buried beneath Quaternary deposits. The

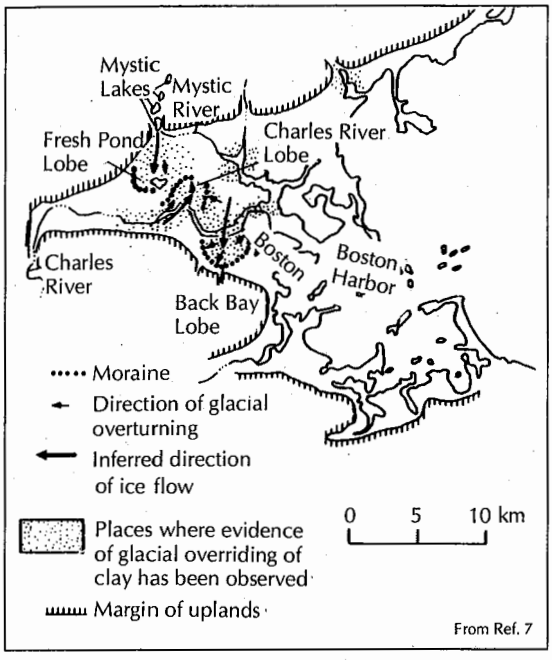


FIGURE 11. Map showing Fresh Pond, the Charles River and Back Bay moraines and areas that have yielded evidence of glacial overriding of clay.

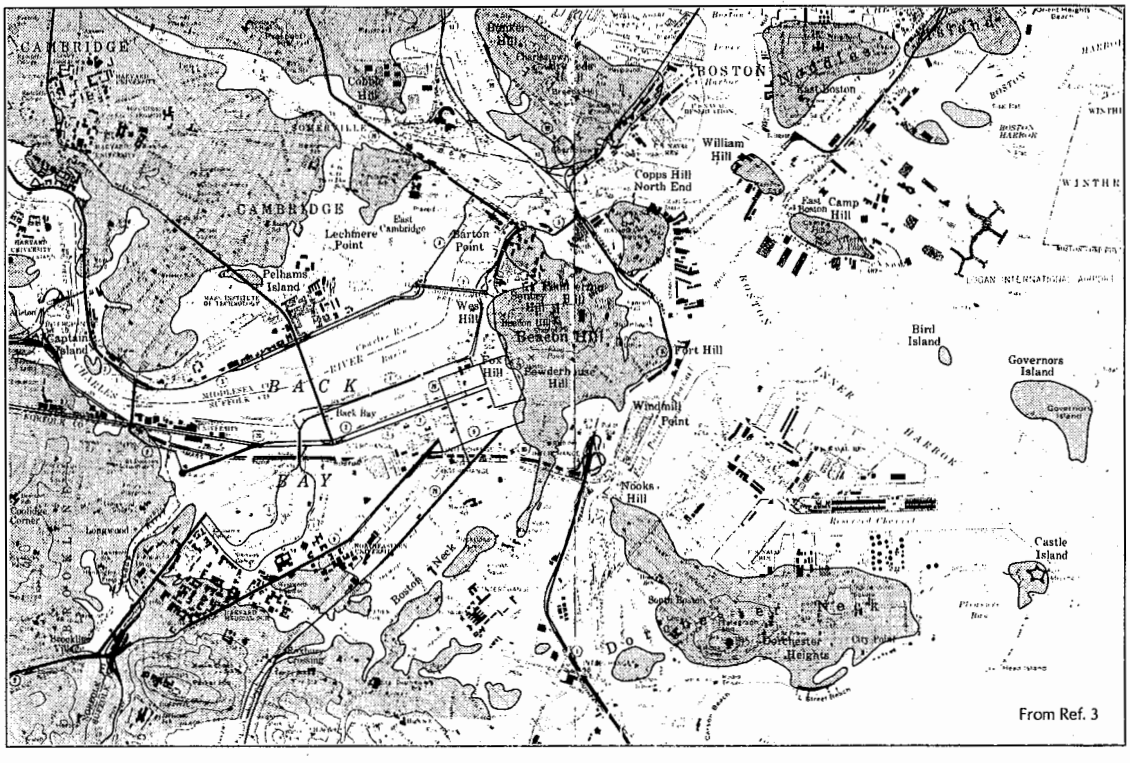


FIGURE 12. The Boston mean-high-tide shoreline as it probably existed in 1630 (shaded).

bedrock surface reflects rock erodibility. The deeper section under the Charles River and Back Bay is underlain by softer strata (argillite, siltstone and sandstone), and the high-standing area along the southern margin marks the outcrop of massive conglomerate (see Figure 7). In detail, the bedrock surface is highly irregular.[7] Dikes stand up as knobs and ridges, major joints are deeply grooved and closed depressions abound.

Pleistocene Geology. The last Wisconsin ice sheet covered the entire region and extended off-shore to the east. Extensive thick outwash plains and moraines developed across Cape Cod and the islands near its southern terminus. As the ice melted and retreated northward, a complex and often bewildering variety of till, drumlin, esker, outwash, delta and lake clay, and sand and gravel deposits were laid down. Sequences of many types of outwash, delta and lake deposits formed in the shallow valleys. Marine clays were deposited along the shore, and sand and gravel deposits lie off-shore. The outwash deposits are several hundred feet thick on outer Cape Cod, but usually the deposits are thin, although quite variable in thickness. Little geologic change has affected the area since the retreat of the ice other than man's activities and the filling of many lowland areas by lake and swamp material. This history has created a highly variable ground condition that may change over a short distance.

Glacial deposits overlie the bedrock almost everywhere in the central Boston area, attaining a maximum thickness of 90 meters (295 feet) in a few places under the Charles River Basin. These deposits include: drumlin till, ablation till, sand, gravel, and silt and clay.[3,4,5,7,20,21] Most of the clay is marine and referred to as the "Boston blue clay," although its moist color is typically light greenish-gray to medium-gray. The Boston area, lying close to or below sea level, was repeatedly flooded by marine waters. Variation in ice thickness, eustatic sea level and isostatic crustal levels were all interrelated factors that affected deposition and erosion (see Figures 8, 9 and 10). Four separate and distinct ice currents occurred in the Boston area. Major wastage of the ice took place about 18,000 years before the present. From that time to about 12,500 years ago, glacial re-advances, such as at Beacon Hill, Back Bay and Fresh Pond, have occurred in the Malden River, Mystic River and Charles River valleys (see Figure 11).

Holocene Geology. The melting and retreat of the glaciers from New England resulted in the surface of the land rebounding from the mass of the ice and a rise in sea level. These events changed the depositional regime in the Boston area. The rising and southward tilt of the land combined with the rise in sea level are in equilibrium just south of Boston, where the Late Pleistocene and present sea-level match. The sea level rise was not uniform, but had at least several fluctuations and was complicated by some crustal subsidence.[22]

Unconsolidated reworked marine clay and organic silt mantle the low areas of Boston and the use of fill, dating back to colonial times, has been extensive around the city.[23] When Boston was first settled, the town lay on a high-tide island that was surrounded in many places by shallow mudflats (see Figure 12). As the town grew into a city, most of the shallow areas have been filled to provide land for the expansion of the city. A record of the stages of filling is reflected in the city's somewhat confusing street pattern, since when each area was filled, it was usually given a pattern of streets different from the earlier ones. Comparison of the area of construction at different times with the original shoreline also shows this phenomenon well.

Engineering & Environmental Considerations

At many places in the Boston Basin, the joints, faults, alteration or type of clay affect the engineering performance of the foundation material. Many rock types have distinct engineering characteristics.[7,16] Secondary alteration has weakened, softened and bleached all basin rock types in places.[24] It has changed the hard rock into a soft, white silty aggregate that can be dug with a hand shovel. These changes are due to the formation of sericite and kaolinite at the expense of all primary minerals, including quartz. The argillites, particularly the maroon and green tuffaceous argillites, seem to be most widely affected, especially under parts of downtown Boston, the Back Bay and the

lower Charles River, where the cause-and-effect relationship of low topography, deep bedrock, and altered rock is notable. The alteration seems to occur in close proximity to certain faults. The cause of this soft-rock alteration is conjectural and may be the result of hydrothermal activity, but it may also represent the roots of lateritic weathering in Tertiary time.[24] Soft-rock alteration probably is responsible for the fact that most of the Boston Basin is ill-suited for tunnelling. Unfortunately, much of the alteration is restricted to relatively narrow zones or beds that are easily missed by exploratory borings. In tunnels, the altered rock almost always requires steel support. This altered rock is often referred to as "shale" in geotechnical reports.

Understanding the regional geology around Boston and its development is important in many ways. Faults of different ages have different trends and characteristics that affect their bearing strengths, and water-bearing and earthquake generating capabilities. They are important for water exploration and understanding the direction of flow of hazardous waste. Some stratigraphic units weather and loose their strength very rapidly in cuts so that they pose problems in slope stability; other units may contain natural groundwater contaminants such as arsenic and iron sulfides. Thus, their recognition and distribution is important. Granites with different ages and tectonic environments at time of intrusion vary greatly in degree of homogeneity, foliation, jointing and potential for radon gas production. These characteristics affect blasting, slope stability, the potential for high-level radioactive waste and other types of underground storage and indoor air quality. Understanding the early tectonic movements helps predict areas of high residual strain that may cause rock bursts in deep excavations. The type of Pleistocene deposit largely determines its water-bearing and groundwater flow characteristics, bearing strength, slope stability and potential for earthquake-induced liquefaction.

The general geology thus contains considerable practical data for construction, groundwater and hazardous waste projects in the region and needs to be considered even for very small sites.

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CLIFFORD A. KAYE *(1916-1985) was an uncompromising scientist and an outstanding geologist. He received his undergraduate degree from Cornell University in 1938 and performed graduate work at the University of California at Berkeley and at Harvard University. Virtually his entire professional career over a 40-year span was spent with the United States Geological Survey and most of that working as an Engineering Geologist in the Boston area. He devoted large amounts of time and energy, even after his retirement in 1981, trying to understand and decipher the complex geology underlying Boston.*

DAVID WOODHOUSE, *CPG, is Vice President and Principal of QUEST Environmental Sciences, Inc., located in Manchester, New Hampshire. Since receiving his undergraduate and graduate degrees in geology from Boston University, he has worked as an engineering geologist for over 20 years with major geotechnical firms in the Boston area. This experience has given him valuable insights into the complex geological and soil problems in the Boston area. He has also developed a keen interest in its history since the colonial times.*

REFERENCES

1. Zen, E-an, "Bedrock Geologic Map of Massachusetts," U.S. Geologic Survey, 1983, scale 1:250,000.

2. Barosh, P.J., Fahey, R.J., & Pease, M.H., Jr., "Preliminary Compilation of the Bedrock Geology of the Land Area of the Boston 2-Degree Sheet, Mas-

sachusetts, Rhode Island, Connecticut, New Hampshire," *U.S. Geol. Survey Open-File Rept. 77-285*, 1977, 91 p., scale 1:125,000.

3. Kaye, C.A., "The Geology and Early History of the Boston, Massachusetts Area — A Bicentennial Approach," *U.S. Geolog. Survey Bull. 1476*, 1976, 78 pp.

4. Kaye, C.A., "Beacon Hill End Moraine, Boston — New Explanation of an Important Urban Feature," in Coates, D.R., ed., *Urban Geomorphology*, Geological Society of America Special Paper 174, 1976, pp. 7-20.

5. Kaye, C.A., "Outline of the Pleistocene Geology of the Boston Basin," in Cameron, B., ed., *Geology of Southeastern New England*, New England Intercollegiate Geological Conference, 68th Annual Meeting, Princeton Science Press, 1976, pp. 46-63.

6. Kaye, C.A., "Surficial Geologic Map of the Boston Area, Massachusetts," *U.S. Geol. Survey Open-File Rept. 78-111*, 1978, approx. scale 1:1,000,000.

7. Kaye, C.A., "Bedrock and Quaternary Geology of the Boston Area, Massachusetts," *Geological Society of America, Reviews in Engineering Geology*, Vol. V, 1982, pp. 25-40.

8. LaForge, L., "Geology of the Boston Area, Massachusetts," *U.S. Geological Survey Bulletin 839*, 1932, 105 pp.

9. Kaye, C.A., "Bedrock Geology, Boston North, Boston South, and Newton Quadrangles, Massachusetts," *U.S. Geological Survey Map MF-1241*, 1980, scale 1:24,000.

10. Barosh, P.J., "Regional Geology and Tectonic History of Southeastern New England," in Hanson, L. S., ed., *Geology of the Coastal Lowlands from Boston, MA to Kennebunk, ME*, New England Intercollegiate Geologic Conf., 76th Ann. Meeting Guidebook, Salem State College, Salem, MA, 1984, pp. 1-35.

11. Barosh, P.J., Pease, M.H., Jr., Schnabel, R.M., Bell, K.G., & Peper, J.D., "Aeromagnetic Lineament Map of Southern New England Showing Relation of Lineaments to Bedrock Geology," *U.S. Geol. Survey Misc. Field Studies, MF-855*, 1977.

12. Grow, J.A., "Structure of the Atlantic Margin of the United States," in Bally, A.W., *et al.*, eds., *Geology of Passive Continental Margins*, Amer. Assoc. of Petroleum Geologists Ed. Course, Note Ser. N. 19, 1981, pp. 3-1 to 3-41.

13. Emerson, B.K., "Geology of Massachusetts and Rhode Island," *U.S. Geol. Survey Bull. 597*, 1917, 289 pp., map scale 1:250,000.

14. Billings, M.P., "Structural Geology of the Eastern Part of the Boston Basin," *American Jour. Sci.*, 5th ser., V. 18, 1929, pp. 97-137.

15. Billings, M.P., "Bedrock Geology of the Boston Basin," in Cameron, B., ed., *Geology of Southeastern New England: A Guidebook for Field Trips to the Boston Area and Vicinity*, New England Intercollegiate Geol. Conf., 68th Ann. Mtg., Boston, Oct. 8-10, 1976, Science Press, Princeton, pp. 28-45.

16. Kaye, C.A., "Engineering Geologic Framework of the Boston Basin," in Hatheway, A.W., ed., *Engineering Geology in New England*, American Association of Civil Engineers, Boston Convention, Preprint 3602, 1979, pp. 1-17.

17. Kaye, C.A., & Zartman, R.E., "A Late Proterozoic Z to Cambrian Age for the Stratified Rocks of the Boston Basin, Massachusetts, U.S.A.," in Wones, D.R., ed., *The Caledonides in the U.S.A.*, Virginia Polytechnic Institute and State University, Department of Geological Sciences, Memoir 2, 1980, pp. 257-264.

18. Sayles, R.W., "The Squantum Tillite," *Museum of Comparative Zoology Bull.*, V. 56, Harvard College, 1914, pp. 141-175.

19. Dott, R.H., Jr., "Squantum 'Tillite' Massachusetts — Evidence of Glaciation or Subaqueous Mass Movements?" *Geological Society of America, Bull.*, V. 72, 1961, pp. 1289-1305

20. Judson, S., "The Pleistocene Stratigraphy of Boston, Massachusetts, and Its Relation to the Boylston Street Fishweir," in Johnson, F., ed., *The Boylston Street Fishweir II*, Robert S. Peabody Foundation for Archeology, V. 4, No. 1, 1949, pp. 7-48.

21. Kaye, C.A., "Pleistocene Stratigraphy of Boston, Massachusetts," *U.S. Geological Survey Professional Paper 424-B*, 1961, pp. B73-B76.

22. Kaye, C.A., & Barghoorn, E., "Late Quaternary Sea-level Change and Crustal Rise at Boston, Massachusetts, With Notes on the Auto-compaction of Peat," *Geological Society of American Bulletin*, V. 75, 1964, pp. 63-80.

23. Whitehill, W.M., *Boston: A Topographical History*, 2nd ed., Belknap Press of Harvard University, 1973, 299 pp..

24. Kaye, C.A., "Kaolinization of Bedrock of the Boston, Massachusetts, Area," in *Geological Survey Research 1965: U.S. Geological Survey Professional Paper 575-C.*, 1967, pp. C165-C172.

Geotechnical Characteristics of the Boston Area

The wide range of geologic conditions influences the methods of testing a particular site as well as the type of foundation to be constructed.

EDMUND G. JOHNSON

Subsurface conditions, as created by both the natural geologic events of the past as well as the more recent activities of man, have played a contributing role in shaping the development of constructed facilities in the City of Boston since colonial times. The present-day urban planner, developer or architect who is contemplating initiating an underground construction project in this city should be sure to include on a design team members who have a firm understanding of these relationships. A thorough and well planned investigation of the site by qualified personnel is of extreme importance, since local subsurface conditions are often very complex and unpredictable.

A thorough assessment of the Boston metropolitan area's geotechnical characteristics should take into account these following factors:

Those conditions created by nature. These conditions include the depths, thicknesses, characteristics and properties of the natural soil overburden deposits and of the underlying bedrock.

Those conditions controlled by man. These conditions include location, depth, quality and information on the history of man-placed fill materials, such as those placed in the Back Bay, marginal waterfront areas and at other sites within the city.

Those conditions controlled jointly by nature and man. These conditions include the current, past and future (predicted) ranges of water levels. The water levels that result from natural runoff and infiltration often are modified by control structures (such as the Charles River Basin), sewer and tunnel routes, underdrainage systems, plus temporary construction dewatering or recharging activities.

Other such conditions include the presence of contaminants in the soil or water that would create potential environments that may be corrosive or destructive to underground construction and/or create human health hazards. These contaminants

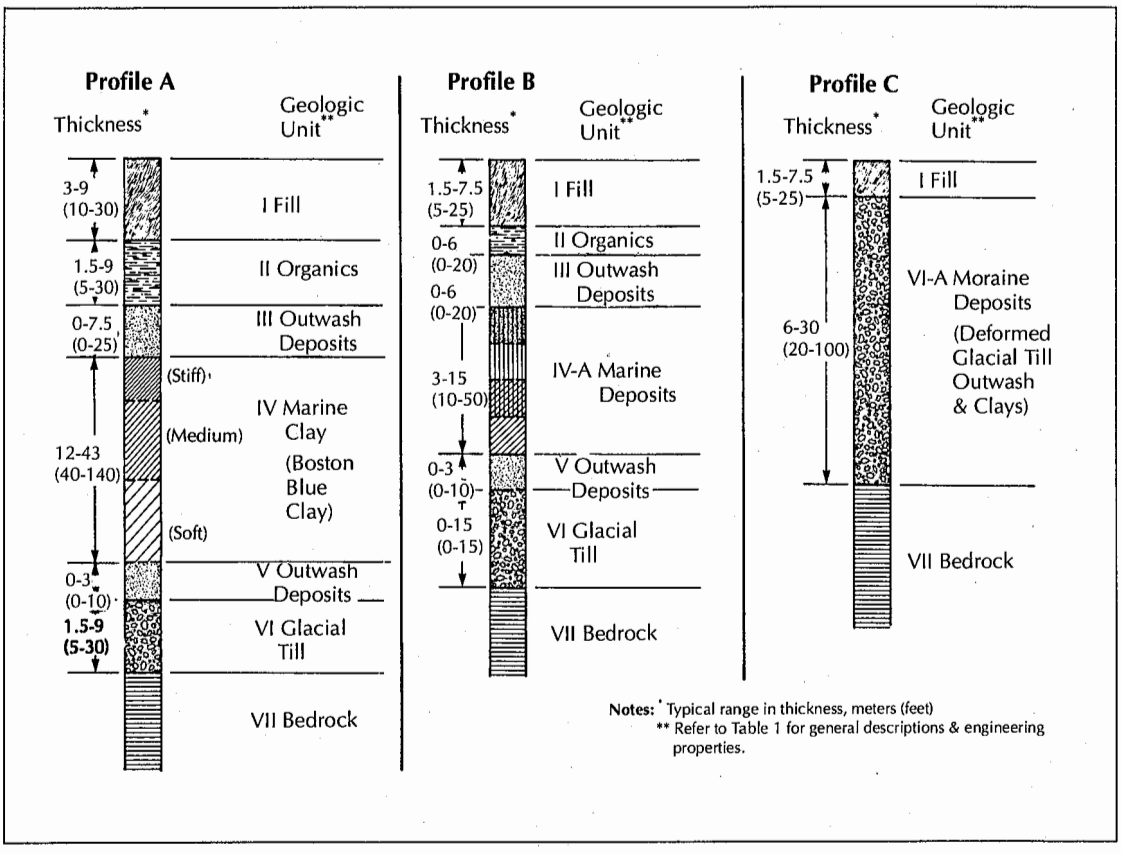

FIGURE 1. Geologic units encountered in typical major foundations.

can also be generated by organic soil deposits, decomposing landfills or uncontrolled hazardous waste sites.

Foundation Materials & Their Engineering Properties

Bedrock. Only the relatively shallow bedrock is of significance for foundation engineering purposes. The predominant upper bedrock that underlies much of Boston is argillite, referred to locally as the Cambridge Argillite.

In its fresh, unweathered condition, the argillite is typically a hard, blue-gray, finely-laminated rock. Local layers of tuff and sandstone are also typical of this formation, as well as numerous intrusive sills and dikes of diabase, diorite or basalt.

However, in many areas, the argillite is highly weathered or altered to the degree that the material can be readily crumbled between the fingers. The explanation of the processes by which the argillite was softened in these local areas is uncertain,[1] but is believed to reflect either hydrothermal alteration or extensive weathering. The altered argillite may vary from light gray to dark green in color. The distribution of the alteration is commonly quite erratic, and the alteration tends to follow the layering of the steeply folded strata. However, the intrusive rock units within the rock mass were apparently less affected by the alteration, and are predominantly moderately hard to hard. These harder intrusive layers may range in thickness from several centimeters to a meter or more (a few inches to many feet).

Where the entire rock mass has been altered, it generally retains a recognizable bedrock fabric such as foliation, bedding or jointing. The rock mass near the top of the altered zone can often be most accurately described as a soil. With increasing depth, the fabric of the rock becomes more evident.

The bedrock surface topography below Boston is quite irregular. A map showing the ap-

proximate contours of the bedrock surface has been developed from interpretation of data from boring logs and construction activities by Kaye.[2] Generally, the rock surface is at a depth of 23 to 53 m (75 to 175 ft) below the surface. The Back Bay borders the eastern edge of a deep bedrock valley that extends to known depths of at least 67 m (220 ft). On the other hand, the rock nearly crops out at the surface in a local area to the northwest of Beacon Hill.

The Cambridge Argillite, which predominates throughout the Boston Basin, has extremely variable engineering properties. The unweathered, unaltered rock may be quite sound. It is so sound that vertical cuts will remain stable with little or no support, and bearing intensities of up to 5,800 kiloNewtons per square meter (kN/m^2) (60 tsf) or more may be appropriate. On the other hand, highly altered zones may have properties similar to a medium or soft cohesive soil. Because of the potential variability, both vertically and laterally, within short distances, a very thorough, well-planned exploration program is warranted if foundation support or other construction is planned on or within the rock.

Conglomerates may be encountered locally in such places as the south and west of the city in portions of Roxbury and Brookline. In contrast to the argillite, it is a very hard, durable stone that was often used in the late 19th century for building and retaining wall construction.[3] It is usually a mottled brown in color, with embedded round to angular pebbles, and resembles a dense concrete material. Surfaces on the conglomerate may be extremely uneven, since these materials were not easily eroded by subsequent periods of glaciation. Construction excavation or drilling of this massive rock may be very difficult, due to its hardness and lack of natural jointing or fracture planes.

Overburden. The overburden in the Boston area is typified by three general sequences, or profiles, for the purposes of foundation construction (see Figure 1):

- Profile A is the most typical and is found below the filled-in Back Bay and marginal waterfront areas.
- Profile B is representative of intermediate areas adjacent to the original Boston Peninsula.
- Profile C is most complex and is found typically within the limits of the original colonial shoreline of the Boston Peninsula.

A description of the geologic units in these sequences, together with their typical engineering properties, is presented in Table 1.

Glacial Till (Unit VI): This unit directly overlies the bedrock throughout much of the Boston area. It is usually of the lodgement variety and it forms a very compact, unsorted, generally non-stratified mixture, of rock fragments and minerals of all sizes, ranging from clay and silt-size particles to cobbles and boulders. The rock fragments are often broken pieces of the underlying bedrock material. The till is extremely variable as a result of the very complex processes of deposition. Pockets and layers of pervious sands and gravels, as well as zones of plastic silts and clays, are often encountered within the mass.

The Standard Penetration Test (SPT) is often the only practical field test to determine an indication of the in-situ density. N-values of over 80 blows per 30 cm (1 ft) are typical where there is more than 15 m (50 ft) of overburden. Lesser values of N, from 40 to 80, are obtained in reworked till, glacial overthrust deposits and at shallower depths. It must be emphasized that the method by which N-values are determined are not precise and unusually high values for individual tests may reflect the presence of gravel, cobbles or boulders encountered by the sampler.

Sample recovery is often poor, and visual examination and classification are often made on very limited quantities. In-situ testing with a pressuremeter device may be appropriate for certain projects. Whenever possible, grain-size and hydrometer tests should be performed as well as Atterberg Limits on cohesive portions. Typical grain-size distribution curves usually indicate a widely graded material with 10 to 25 percent or more of the grains finer than a number 200 sieve.

Outwash Deposits (Unit V): These glaciofluvial deposits consist of medium dense, stratified sands and gravels of a discontinuous nature that overlie the lodgement till.

TABLE 1
Typical Engineering Properties of Foundation Material in Boston

Geologic Unit	General Description	Saturated Unit Weight kg/m³ (lb/ft³)	Natural Water Content (percent)	Atterberg Limits (percent) LL	PI	Undrained Shear Strength kg/m² (lb/ft²)	Other	Allowable Bearing Pressure kg/m² (lb/ft²)
I. Miscellaneous Fill	Loose to very dense sand, gravelly sand or sandy gravel, intermixed with varying amounts of silt, cobbles or boulders, & miscellaneous brick, rubble, trash or other foreign materials.	1600-2000 (100-125)	—	—	—	—	—	—
II. Organics	Very soft to medium stiff, grey clayey organic silt or brown fibrous peat with trace amounts of shells, fine sand & wood.	1440-1760 (90-110)	40-100	—	—	1465-3900 (300-800)	Organic Content 5-25%	—
III. Outwash Deposits	Medium dense to dense, brown coarse to fine or medium to fine sand with varying amounts of gravel & silt.	1760-2160 (110-135)	—	—	—	—	—	19500-48800 (4000-10000)
IV. Marine Clay	Stiff, yellow-grey silty clay.	1840-2000 (115-125)	25-35	40-55	15-30	3900-9760 (800-2000)	Compression Ratio = 0.15-0.25	14650-39000 (3000-8000)
	Medium stiff, grey silty clay, occasional layers of fine sand or silt.	1824-1920 (114-120)	30-40	40-55	15-30	2930-5860 (600-1200)	Recompression Ratio = 0.02-0.04	9760-19500 (2000-4000)
	Soft to very soft, gray silty clay, occasional layers of fine sand or silt. (Note: This unit sometimes becomes stiffer at lower levels.)	1810-1890 (113-118)	30-50	40-55	15-30	1950-3900 (400-800)		4880-9760 (1000-2000)
IV-A. Marine Deposits	Interbedded grey silty or sandy clay, silty fine sand & fine sandy silt	Too variable	—	—	—	—	—	Variable
V. Outwash Deposits	Medium to dense, stratified sands & gravels in discontinuous layers.	—	—	—	—	—	—	Variable
VI. Glacial Till	Dense to very dense, heterogenous mixture of sand, gravel, clay & silt with cobbles & rock fragments.	2000-2240 (125-140)	10-20	15-30	10-20	9760-39000 (2000-8000)	—	39000-98000 (8000-20000)
VI-A. Moraine Deposits	Miscellaneous deposits of deformed glacial till, outwash & clays.	Too variable	—	—	—	—	—	Variable
VII. Bedrock	Cambridge Argillite.	—	—	—	—	—	—	78000-195000 (16000-40000)
	Roxbury Conglomerate.	—	—	—	—	—	—	195000-975000 (40000-200000)

Note: Metric units above English units in parentheses.

Marine Clays (Unit IV): These clay deposits are usually referred to locally as the Boston Blue Clay. Its properties have been investigated thoroughly for foundation design. There is generally very little clay directly below the original downtown Boston peninsula, but in the Back Bay, as well as along marginal waterfront areas, the clay is typically 15 to 38 m (50 to 125 ft) thick. Even greater thicknesses, up to 60 m (200 ft), have been found to the west of Massachusetts Avenue and in the City of Cambridge.

A weathered crust is present at the top of the clay. This crust is the result of desiccation, oxidation and capillary stress. It is yellowish or brownish in color, in contrast to the normal

gray or olive-gray color of the lower clay. The presence of the stiffer crust plays an important role in the support of structures in the area.

Extensive laboratory test programs have been performed on the clay by researchers and practitioners over the past 40 years. Tests on samples from the site of the Prudential Center[4] and the Massachusetts Institute of Technology (MIT) in Cambridge,[5] reveal that the stiff yellow clay has been pre-consolidated (compacted) to four or more times the present overburden stress. The overconsolidation ratio decreases quite rapidly with depth, so that the clay below a depth of about 21 to 27 m (70 to 90 ft) is considered to be normally consolidated.

Discontinuous layers and lenses of sand and silt are often encountered within the clay. Thus, horizontal permeability is generally several times greater than the vertical. Typical ranges of undrained shear strength and other engineering properties are given in Table 1.

Marine Deposits (Unit IV-A): These deposits were formed in areas that were inundated with marine waters at locations close to the shoreline, creating a complex depositional environment. Quantities of silt and clay-sized particles, discharged by glacial meltwater streams into the sea, slowly settled out of suspension to form strata of clay. Simultaneously, sands and silts were deposited by meltwater streams and near-shore currents. As a result, a highly complex marine deposit of alternating and interfingering layers of fine sand, or silt and clay, developed in these areas. It is not practical to try to typify the engineering properties for this unit since its composition varies so widely.

Outwash Deposits (Unit III): Sand and gravel was deposited over the surface of the weathered clay in some areas, following another advance of glacial ice. These well-stratified sands and gravels range in thickness from 3 to 7.5 m (10 to 25 ft). They are medium to compact and are considered an important bearing stratum for supporting light to medium weight structures. Their relatively high permeability is also important.

Organic Deposits (Unit II): Organic silt and clay deposits were formed throughout much of the lower lying areas surrounding the Boston Peninsula following the ice age. The organic deposits vary greatly in overall thickness and content, but are generally from 1.5 to 7.5 m (5 to 25 ft) thick. In those filled-in areas of the Back Bay, this layer has been compressed considerably due to the weight of the fill. Marsh gas, resulting from the decomposing organic matter, is sometimes encountered in excavations.

Man-Placed Fills. Low-lying areas began to be filled starting in the late 18th century when colonial Boston outgrew the limited area of the original peninsula.[6] Previously, the entire Back Bay area was a mud flat and the Charles River was a tidal estuary. A mill dam was first constructed in 1820 along what is now Beacon Street, from Charles to Kenmore Square, to harness tidal power. Subsequently, railroad embankments were built across Back Bay. This construction, in effect, created stagnant water areas that eventually were filled in for development purposes. Between 1856 and 1890, the entire Back Bay between Charles Street and the Fenway was filled. The fill materials consisted of clean sands and gravels, brought by rail from a source in Needham, about 15 km (9 miles) to the west. Following the construction of a tidal dam across the Charles River in 1910 that controlled the water level in the Basin, embankment fill was placed along the river and Storrow Drive was completed in 1951. On the Cambridge side, the tidal marshes were filled in and the area was developed, including the present campus of MIT, following the construction of a granite seawall about 1890. The waterfront areas facing Boston Harbor were also filled in by stages. This process was quickly followed by pier and bulkhead construction. In general, the materials used for these fills were earth remnants from several high land areas within the original Boston Peninsula, such as Fort Hill, plus dredged materials and demolition rubble.

Groundwater Levels

General Conditions. As might be expected, the normal groundwater level in Boston and the Back Bay area is generally close to mean sea level. Although the normal tide range in the harbor is about 1.5 m (5 ft) above and below mean tide, similar fluctuations in groundwater levels below the city are usually not observed, except along marginal waterfront areas. A

stabilizing factor results from the Charles River Basin being maintained at about 0.73 m (2.4 ft) above mean sea level.

Variations and anomalies in the piezometric surface are often related to the dewatering implemented for construction projects or possibly pumping from deep basements. Leakage from or into storm sewers is another factor. The many subway tunnels and deep utilities in the area often form either barriers or drainage paths that interrupt or control normal groundwater flow.

There is evidence that the sea level was considerably lower in the past. Freshwater peat and tree stumps, as well as ancient Indian fish weirs have been found in excavations at levels from 5 to 7.5 m (15 to 25 ft) or more below the present mean sea level.

Influence on Constructed Facilities. Groundwater levels are a key factor in any geotechnical assessment of conditions in the Boston area. The determination of realistic present water levels, as well as past and potential future variations, are of major significance. In Boston, the consequences of lowering the water levels below normal, even temporarily, fall in two general categories:

- General subsidence of the land — including streets, utilities or buildings founded at shallow depths — may occur if the water is depressed in areas underlain by soft compressible layers, such as the filled-in Back Bay. The settlement would occur very slowly and the magnitude would reflect the relative thickness of the soft underlying soils.
- Individual buildings supported on untreated wood piles may settle if the pile butts are exposed to drying and decay. As long as the piles are constantly submerged, and not exposed to air, they will not be attacked by fungi. Also, the lowered water level may result in the consolidation of the soils surrounding the piles, and thus the frictional forces that develop along the piles may create additional loads for which the piles were not designed.

The maintenance of "normal" water levels has been very important concern to city officials during the past century. After the filling of the Back Bay, most structures were supported on untreated wood piles, driven to bearing in the sand layer below the organic deposits or as friction piles in the clay. Since the groundwater level was at that time found at approximately 0.7 m (2.3 ft) above mean sea level, the piling was usually cut off at 0.6 to 0.9 m (2 to 3 ft) below this grade. However, with the subsequent effects of decreased surface infiltration and as the areas were developed (for example, dewatering for tunnels and drainage systems or other local pumping activities), it was discovered that the wood piles below many structures were no longer permanently submerged, and that these piles became exposed to drying and decay.

A notable example of the problem occurred in 1929, when major cracks were discovered in the walls of the Boston Public Library at Copley Square.[7] Upon investigation, it was discovered that the tops of wood piles were decaying. A major underpinning effort was required to restore the foundation system. The apparent reason for the lowering of the water table was traced back to the earlier construction of storm and sanitary sewer lines, with invert levels about 1.8 m (6 ft) below water table. Steps were taken to control the infiltration and restore the levels to normal.

More recently, problems with foundation distress and rotted piles have occurred in the lower Beacon Hill area. Investigations have revealed that the groundwater levels were as much as six feet below the water level in the Charles River. These lowered levels are attributed to leakage into sewers. Aldrich and Lambrechts provide an excellent historical perspective on groundwater fluctuations in the Back Bay and on the adverse effects of lowered levels.[8]

Due to the adverse effects of a drop in the groundwater level, it is a requirement that an adequate cutoff system be installed to control drawdown beyond the site during any construction excavation below the groundwater level. Adjacent areas must be monitored and, if necessary, remedial action must be taken such as modifying the pumping operation or installing a recharge system.

Exploration & Testing Practices

Subsurface Investigations. Local exploration practice, for the most part, consists of boring and sampling methods that are performed in accordance with American Society for Testing and Materials (ASTM) standards. These methods are considered to be "direct", and consist of borings that penetrate the overburden soils, the recovering of rock and physical samples for laboratory testing, and determining the stratigraphy and geotechnical properties. Other "indirect" and principally geophysical methods such as seismic refraction, resistivity and cross-hole seismicity tests are less likely to be used in the urban area. The exploration work is generally contracted to one of several qualified independent drilling firms, with the field work being monitored by representatives of the consultant for the construction project.

Most standard borings are made using a 6.3 or 7.6 cm (2.5 or 3.0 in) diameter steel casing to maintain the hole through unstable soils. Larger diameter casing is used when undisturbed piston samples are required. The casing is advanced by driving and the soil within is washed out with chopping bits and clean-out tools to the desired sampling depth. When penetrating cohesive soils, such as the clay, the casing is generally not required and the hole may be stabilized by drilling mud. For deep borings that are to penetrate bouldery glacial tills, rotary drilling techniques are generally used to advance the flush-joint casing by using a core barrel or tricone bit.

An alternate procedure is the use of hollow stem helical flight augers, mounted on large mobile truck rigs in order to advance the hole and provide for soil sampling after the removal of a closure plate at the bottom. This method achieves only limited success in pervious soils that are under hydrostatic pressure.

Conventional sampling procedures are usually employed, wherein disturbed samples are obtained by driving a 5 cm (2 in) outside diameter split-spoon sampler at 1.5 m (5 ft) intervals, or at change in soil type, using a 63 kg (140 lb) hammer dropping 76 cm (30 in). Continuous sampling is sometimes used when it is important to detect frequent changes in the stratigraphy. In the clay, relatively undisturbed samples are recovered with a 7.6 cm (3 in) inside diameter stationary piston tube sampler or 5 cm (2 in) Shelby tube.

Core drilling in the rock is accomplished with either BX- or NX-size core barrels. In weathered or altered argillite, the best sample recovery method is the use of an NX-size double tube barrel with a split inner liner.

Field permeability tests are performed below the casing in boreholes at selected depths. The use of observation wells or piezometers, or both, are often necessary in order to determine the long-term stabilized water levels. Seals are required where different piezometric levels may occur at various depths within the boring. Pressuremeter tests to determine the in-situ properties are useful in measuring the stress-strain properties of the glacial till, since undisturbed sampling of the till is not practical.

Available Boring Data. A most valuable resource with regard to the available subsurface information is the collection of boring data published by the Boston Society of Civil Engineers Section/ASCE.[9,10,11,12] These volumes contain the tabulations of the logs of several hundred borings that are located in the Boston Peninsula as well as in South Boston and Roxbury. The data were collected from many sources such as architects, engineers, contractors, public agencies and others. A similar effort was undertaken to publish data for the Cambridge area.[13]

Laboratory Testing. Laboratory testing is performed primarily in the private laboratories of geotechnical consultants in connection with specific projects. During the past 30 years, Boston area soils have been tested extensively at both Harvard University and MIT, either for particular projects or for general research. The Boston Blue Clay is considered one of the most thoroughly tested and researched soils in the world. Of particular note is the work done by Arthur and Leo Casagrande in the mid-1950s during design investigations for the Prudential Center in the Back Bay.[4] The prediction of the consolidation behavior of the clay by this work was critical for the project. During the 1960s there was considerable construction activity on the MIT campus, across the Charles River in

Cambridge. A program called Foundation Evaluation and Research-MIT (FERMIT) included extensive laboratory investigations performed on campus subsoils, particularly the blue clay. The published results from the work of this program are useful in understanding the behavior of the local soils.[5]

Foundation Types Used for Local Geologic Conditions

Selection of Appropriate Foundation Systems. Foundation designs in Boston must comply with the Massachusetts Building Code. The first state-wide Building Code was issued in 1975. Article 7, Structural and Foundation Loads and Stresses, was incorporated, nearly intact, from the then-existing City of Boston Code. Subsequent code revisions have been made, which allow for increased loads in piles and other changes as well as provisions for the design of foundations to withstand earthquakes. The seismic criteria were the first such criteria developed specifically for a jurisdiction in the eastern United States.

The soil and foundation seismic criteria in Section 716 of the code are innovative and comprehensive. The design philosophy recognizes that the probable maximum earthquake intensities for Massachusetts may be as large as those for California, but have much longer return periods. The criteria aim to minimize the loss of life in the event of major earthquakes, but without imposing excessive construction costs. The code emphasizes the ductility requirements for structures and it prescribes relatively modest lateral forces. Most "model" codes simply apply a "zone factor" to the lateral force requirements that were developed for California. The code also has specific provisions regarding potential liquefaction, earthquake-induced lateral earth pressures and the effects of local soil conditions.

The state Board of Building Regulations and Standards is responsible for the administration of the code. Technical assistance is provided by Loads, Geotechnical and Seismic Advisory Committees. These committees are composed of selected volunteer practicing professionals who periodically review code provisions and recommend revisions and additions. Changes are incorporated into the code following a public review process.

Foundation Types Used in Various Geologic Units. The foundation types considered appropriate for bearing within each of the geologic units found in each of the three typical geologic sequences are illustrated in Figure 2. These foundation types are keyed to the three sequences, or profiles, labeled A, B and C in the figure. Those shown are considered as representative of most, but not all, of the foundation types used in the Boston area.

Fill & Organic Deposits: The fill and the organic layers are not suitable for the support of any significant structures.

Outwash Deposits: Light to medium weight structures may be supported on short piles or caissons (cast-in-place shafts or piles) in profiles A or B. If the sand layer is relatively shallow, it may be feasible to use spread footings. For profile A, estimates must be made of the post-construction settlement of the underlying clay (Unit IV). Usually, the settlements of buildings with up to 10 to 12 stories and having one basement level will be nominal. Higher buildings may be possible if more than one basement level is provided, since the stress relief from deeper excavations compensates for the additional foundation loading.

Untreated wood piles (III-1) were used predominantly for early construction in the Back Bay. Typical pile capacities of 62 to 89 kN (8 to 10 tons) were most common when driven to bear in the sand layer. However, problems can develop if the pile butts are exposed to drying. Therefore, they must always remain submerged below the groundwater level. Otherwise, the use of pressure-treated piles can overcome this problem.

Pressure-injected footings (PIFs) (III-2) that offer individual capacities up to 1070 kN (120 tons) or more are feasible where the layer has a proper grain-size distribution (less than 15 percent fine material) and the layer thickness is at least 3 m (10 ft). The PIFs are a unique pile type, and consist of advancing a heavy steel drive tube into the sand surface and then driving out one or more batches of very dry concrete mix, 0.14 m^3 (5 ft^3) each, to form an expanded base within the granular material. A concrete shaft is then formed above the base to complete the unit.

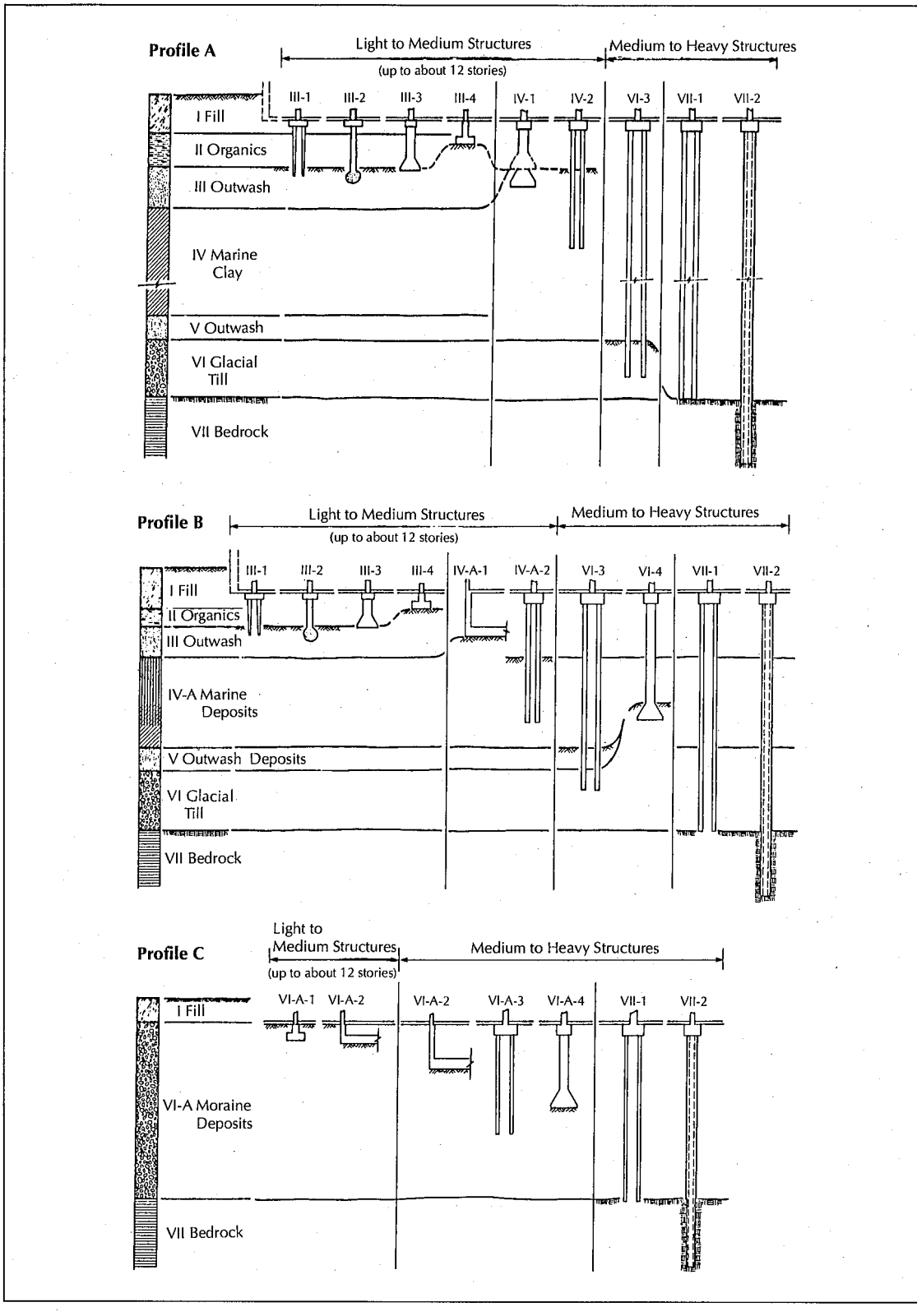

FIGURE 2. Typical foundation types used in Boston.

Belled caissons (III-3) may be installed to bear on the sand layer only if it is practical to make undercuts in the overlying organic layer. Otherwise, straight-shaft units would be required, which are less economical. During construction, it is generally required that the base be dewatered before the concrete is placed. It is sometimes necessary to dewater the sand bearing layer in the vicinity by installing wells, provided there will be no adverse effects from the dewatering in the adjacent area.

Spread footings (III-4) are feasible where the depth to the top of the bearing layer is only a meter or so (few feet) and dewatering does not pose a serious problem. The units are usually sized for a bearing value of 240 to 480 kN/m^2 (2.5 to 5.0 tsf).

Marine Clay: For profile A, some light to medium structures are founded directly on or within the Boston blue clay. It is important to note, however, that estimates of potential settlement must be made.

Belled caissons (IV-1) are perhaps the most common foundation type in Unit IV. Steel casings are advanced through the upper soils and sealed into the organic deposits or clay surface. The belled portions are then undercut by a rotary machine to a diameter of 1.8 to 3 m (6 to 10 ft) or more. In the early days, this was done by hand labor. Usually there is little or no dewatering required. The units are typically designed for an end-bearing capacity of 190 to 380 kN/m^2 (2 to 4 tsf) in the upper stiff clay zone. If that zone is fully penetrated, the caissons bearing on the softer clays below would have a reduced design capacity. In all cases, the strength of the clay should be verified in the field by competent geotechnical personnel.

Friction piles (IV-2) are also used to provide support in the upper clay. Wood piles with a capacity up to 196 kN (22 tons) are allowed by the Massachusetts Building Code. Other pile types have also been used, based on a typical design friction value of 24 kN/m^2 (500 psf) for the portion embedded in the clay. Any such installation should be verified by on-site pile-load tests.

Marine Deposits: Light to medium structures are founded on or within profile B, especially when the overlying sand layer (Unit III) is absent or thin. Because conditions in this sequence may be very erratic, such as a combination of granular and cohesive units in discontinuous layers and lenses, each site must be carefully evaluated. Soil-bearing footings or mat foundations are usually the most feasible foundation system. Occasionally, friction piles are used. There is one known case where pressure-injected footings have been used. A very careful determination of the location and quality of granular deposits was required prior to installation.

Footings or mats (IVA-1) are feasible, especially where the design requires that basement excavations extend down to this unit. Where the foundation extends below the water level, a reinforced mat and waterproofed wall system are usually required.

Friction piles (IVA-2) may be considered when other foundation types are not practical. A conservative design would be to assume that all of the material is cohesive and allows a frictional resistance of 24 kN/m^2 (500 psf) for the exposed pile surface in the marine deposit.

Glacial Till: Where suitable portions of the moraine are close to the surface in profile C, the most feasible foundation type, regardless of the size of the structure, would be soil-bearing footings (VI-1) or mat (VI-2). Heavy structures extending below the water table would probably require a mat (VI-2). In these conditions, permanent underdrainage systems may have to be considered in order to relieve hydrostatic pressures. In some instances, where the upper portion of the deposit is weak, piles (VI-3) or belled caissons (VI-4) are used.

Medium to heavy structures, for which shallower foundations are not practical, may be supported on piles (VI-3) or caissons (VI-4) bearing in the glacial till in profiles A or B. The depth to the till is usually 23 m (75 ft) or more. In this case, it is advantageous to select a unit with as high a load capacity as possible.

Footings or mats (VI-1, VI-2) are usually designed for soil bearing pressures of up to 960 kN/m^2 (10 tsf). Even higher values may be possible, taking into consideration that the code allows an increase of five percent per foot of depth of penetration below the bearing soil, up to two times the design value at the surface.

Piles (VI-3) are usually designed for end-bearing in the glacial till at design values up to

about 1,335 kN (150 tons). Concrete-filled steel pipe piles have been used extensively as piles, but the design must take into account an allowance for corrosion if the piles must pass through a layer of organic material. In the past several years, prestressed concrete piles have been widely used. The current Massachusetts code allows capacities up to 872, 1,192 or 1,558 kN (98, 134 or 175 tons) for 30.5, 35.5 or 40.6 cm (12, 14 or 16 in) square sections, respectively.

Belled Caissons (VI-4) may be appropriate where a single caisson unit can be installed below any column. Multiple units are generally not economical because of the large cap required. They may be designed for end-bearing in the lodgement till at capacities of up to 960 kN/m^2(10 tsf). Higher values may be used for deeper penetrations into the till. In some instances, straight-shaft caissons have been used, with support from side friction as well as end-bearing.

Bedrock: Normally, the bedrock is located 23 m (75 ft) or more below the surface. Unless the structure is quite heavy, shallower foundations are usually more economical. The rock level is close to the surface west of Beacon Hill where a 36-story apartment structure is founded on spread footings that rest directly on the argillite.

Piles (VII-1) may be driven to end-bearing on the rock surface where the overlying units do not provide adequate driving resistance. In those areas where the argillite may be weathered, the piles may penetrate into the rock. Close attention must be given to the selection of an appropriate design capacity for this case.

Drilled-in caissons (DIC) (VII-2), as described in Section 739 of the code, are limited to unique situations where very high column loads must be accommodated and other foundation types are not feasible. DIC design usually calls for a combination of end-bearing and side friction in a rock socket. A permanent heavy steel open-end casing is advanced by driving and internal cleaning to the rock surface and then seated. A socket is advanced into the rock using a churn drill or other methods to a depth of 3 to 7.5 m (10 to 25 ft). A heavy steel H-section is lowered to the bottom and concreted. If the water is unable to be dewatered, it must be inspected by remote video camera prior to concreting. Total capacities of 11,600 to 14,700 kN (1300 to 1650 tons) per unit were developed for a major tower structure in the Back Bay.[14]

More recently, drilled shafts or piles have been advanced into the rock using temporary casing or bentonite slurry to stabilize the hole. A steel core or reinforcement is installed and the hole is backfilled with cement grout in order to develop the load in friction as well as in end-bearing.[15]

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REFERENCES

1. Kaye, C.A., "Kaolinization of Bedrock of the Boston, Massachusetts, Area," U.S. Geol. Survey, Professional Paper 575-C, 1967, pp. C165-C172.

2. Kaye, C.A., "Bedrock Geology, Boston North, Boston South and Newton Quads," U.S. Geol. Survey, Misc. Field Studies Map, MF 1241, 1980, scale 1:24,000.

3. Kaye, C.A., "The Geology and Early History of the Boston Area of Massachusetts — A Bicentennial Approach," *U.S. Geol. Survey, Bulletin 1476*, 1976, 78 pp.

4. Casagrande, A.L., "Foundation Investigation for Prudential Center in Boston," unpublished report, Metcalf & Eddy, Engineers, 1958.

5. Ladd, C.C., & Luscher, U., "Engineering Properties of the Soils Underlying the MIT Campus," MIT Dept. of Civil Engineering, Research Report R65-58, Soil Mechanics Publication 185, 1965.

6. Aldrich, H.P., Jr., "Back Bay Boston, Part 1," *Journal of the Boston Society of Civil Engineers*, V. 57, 1970, pp. 1-33.

7. Aldrich, H.P., Jr., "Preserving the Foundations of Older Buildings: Technology & Conservation," Summer 1979.

8. Aldrich, H.P., Jr., & Lambrechts, J.R., "Back Bay Boston, Part II: Groundwater Levels," Boston Society of Civil Engineers, *Civil Engineering Practice*, V. 1, N. 2, 1986, pp. 31-64.

9. Boston Society of Civil Engineers, "Boring Data from Greater Boston," *Journal of the Boston Society of Civil Engineers*, 1961, 280 pp.

10. Boston Society of Civil Engineers, "Boring Data from Greater Boston: Sec. 1, Boston Peninsula," *Journal of the Boston Society of Civil Engineers*, V. 56, 1969, pp. 131-291.

11. Boston Society of Civil Engineers, "Boring Data from Greater Boston: Sec. 2, Roxbury," *Journal of the Boston Society of Civil Engineers*, V. 57, 1970, pp. 185-291.

12. Boston Society of Civil Engineers, "Boring Data from Greater Boston: Sec. 3, S. Boston," *Journal of the Boston Society of Civil Engineers*, V. 58, 1971, pp. 51-63.

13. Boston Society of Civil Engineers, "Boring Data from Greater Boston: Sec. 4, Cambridge," *Journal of the Boston Society of Civil Engineers Section/ASCE*, V. 70, Nos. 1 & 2, 1984.

14. Ball, Donald G., "Prudential Center Foundations," *Journal of the Boston Society of Civil Engineers*, V. 49, 1962, pp. 229-244.

15. Johnson, E.G., & Schoenwolf, D.A., "Foundation Considerations for the Expansion & Renovation of the Hynes Auditorium," Boston Society of Civil Engineers, *Civil Engineering Practice*, Vol. 2, No. 2, 1987, pp. 35-62.

The Hazard From Earthquakes in the Boston Area

The varied underground conditions in the area result in a range of effects that can be expected from a seismic event.

PATRICK J. BAROSH

Northeastern Massachusetts is one of the relatively more seismically active areas in New England and where the greatest earthquake recorded for New England occurred in 1755. Boston was damaged by this earthquake and an earlier nearby one in 1727. The city also suffered, in 1638 and 1669, from earthquakes at La Malbaie in Quebec. The recurrence of earthquakes of these magnitudes, and the possibility of an even larger one, is of considerable concern due to the large population in the region, the larger number of buildings with poorer foundation conditions than in the past, and potential affects on power plants, dams and tall structures.

Several pioneering earthquake studies were made in the area. The investigation of the 1755 earthquake by Winthrop is one of the first earthquakes studied in a modern descriptive manner without a strong religious bias of God's punishment of sinners.[1] A masterful study of the regional control of earthquakes in the region was made by Hobbs in 1907.[2] The effect of the varying ground conditions on the hazard from earthquakes was investigated in Boston by Crosby.[3,4,5] Despite these excellent studies, Boston was later placed in an unwarranted high hazard category on a national map[6] due to an error that placed the 1755 earthquake within metropolitan Boston.

The need to determine the actual hazard for nuclear power plant safety in the region ushered in a series of evaluations of the seismic hazard for specific sites such as at Seabrook, New Hampshire.[7] The New England Seismotectonic Study was formed by the U.S. Nuclear Regulatory Commission in 1976 to investigate the cause of seismicity and formulate an earthquake zonation map of the northeast United States.[8,9] The results of this study, plus an analysis of the tectonic stability of the region for the purpose of locating storage sites for highlevel radioactive waste[10] and additional studies for the U.S. Army Corps of Engineers to evaluate dam safety,[11,12,13,14] have now produced reliable earthquake zonation for the region. Unfortunately, some earthquake building codes in the region follow a simplified probabilistic study that is used by the Applied Technology Council that is based largely on

TABLE 1
Modified Mercalli Intensity Scale of 1931 (Abridged Version)

I. Not felt except by very few people under especially favorable circumstances.

II. Felt only by a few persons at rest, especially on the upper floors of buildings. Delicately suspended objects may swing.

III. Felt quite noticeably indoors, especially on the upper floors of buildings, but many people do not recognize it as an earthquake. Standing cars may rock slightly. There are vibrations like that of a passing truck. Duration estimated.

IV. During the day it is felt indoors by many, outdoors by few. At night some are awakened. Dishes, windows and doors are disturbed; walls will make cracking sounds. It will create a sensation like a heavy truck striking a building. Standing cars rocked noticeably.

V. Felt by nearly everyone; many are awakened. Some dishes, windows, *etc.*, are broken; a few instances of cracked plaster; unstable objects are overturned. Trees are disturbed, as well as poles and other tall objects. Pendulum clocks may stop.

VI. Felt by all; many are frightened and run outdoors. Some heavy furniture is moved; a few instances of fallen plaster or damaged chimneys. Damage slight.

VII. Everybody runs outdoors. Damage is negligible in buildings with good design and construction; slight to moderate damage in well-built ordinary structures; considerable damage in poorly built or badly designed structures; some chimneys broken. Noticed by persons driving cars.

VIII. Damage is slight in specially designed structures; considerable in ordinary substantial buildings with partial collapse; great in poorly built structures. Panel walls are thrown out of frame structures. Chimneys, factory stacks, columns, monuments and walls will fall. Heavy furniture is overturned. Sand and mud are ejected in small amounts. Changes in well water. It will disturb persons driving cars.

IX. Damage is considerable in specially designed structures; well designed frame structures are thrown out of plumb; great damage in substantial buildings, with partial collapse. Buildings are shifted off foundations. Ground cracks conspicuously. Underground pipes break.

X. Some well-built wooden structures are destroyed; most masonry and frame structures are destroyed; the ground is badly cracked. Rails bend. Landslides are considerable from river banks and steep slopes. Sand and mud shifts. Water splashes (slops) over banks.

XI. Few, if any, masonry structures remain standing. Bridges are destroyed. Broad fissures develop in the ground. Underground pipe lines are put completely out of service. Earth slumps and land slips in soft ground. Rails bend greatly.

XII. Damage is total. Waves are seen on ground surfaces. Lines of sight and level are distorted. Objects are thrown upward into the air.

From Ref. 18

assumptions and may not be accurately relied on.[15] Thus, again local knowledge is not utilized in producing a national zoning map. More detail on the earthquake hazard in Boston and the regional movements and cause can be found in other studies.[9,14,16,17] All earthquake intensities are given in the Modified Mercalli Scale (see Table 1).[18]

Seismicity of Eastern Massachusetts

New England is a region of moderate earthquake hazard that experiences an earthquake every couple of days (see Figure 1). This rate of activity is, at least, an order of magnitude less than that for Southern California. These earthquakes, as shown by the 400 year-long record,[19] are concentrated in certain areas, whereas other areas remain very quiet.[8,20] The general areas of relatively high activity now appear well defined. The earthquakes whose depths have been determined, generally have depths less than 10 kilometers and many have a depth of only a few kilometers.

Most of the seismic activity in eastern Massachusetts is concentrated in the Cape Ann seismic area in a great arc around Cape Ann that extends from the south side of Boston to southern Maine (see Figures 2 and 3). The two largest recorded earthquakes in the area oc-

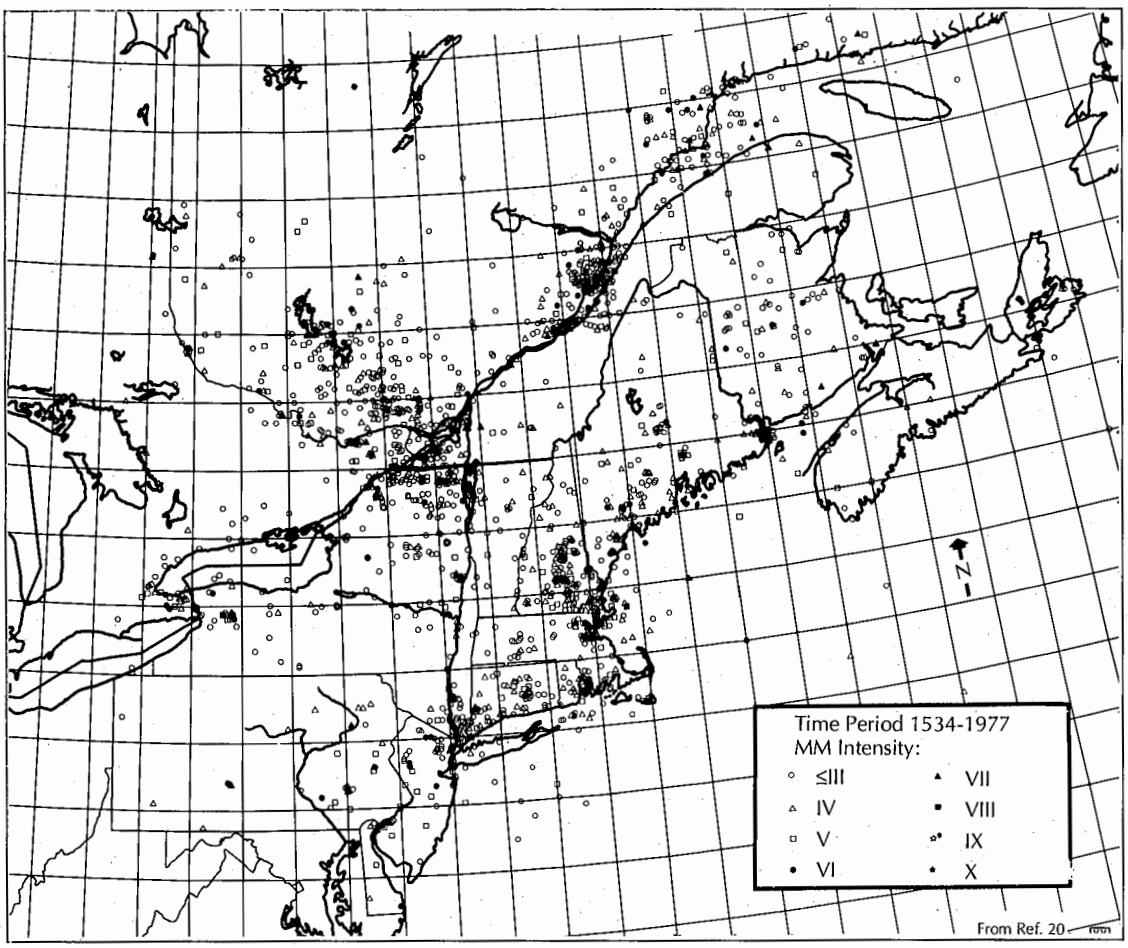


FIGURE 1. Map showing seismicity of the northeastern United States and adjacent Canada from 1534 to 1977.

curred offshore of the north side of Cape Ann. The earthquakes in the area tend to decrease in size away from this site towards Boston.

The two principal historic seismic events are the 1755 earthquake of probable epicentral intensity VIII and the 1727 event of intensity VII.[7] The earthquake of November 18, 1755 was located about 80 km (50 mi) offshore of Boston. It came 17 days after the Great Lisbon earthquake and was felt from Halifax, Nova Scotia, to Annapolis, Maryland (see Figure 4).[1] It caused damage across eastern Massachusetts and thoroughly frightened the inhabitants. In Boston it reached a high intensity VII. In the city the tops of up to 1,500 chimneys were demolished and others cracked and received some sort of damage. Bricks, tiles and slates were scattered in the streets, and large quantities of mortar and rubbish spread almost everywhere. Several houses suffered large cracks and breaches in their foundations. In some places, especially on the low, loose ground made by filling in around the harbor, the streets were almost covered with fallen bricks.

The smaller earthquake of November 9, 1727 that was similarly located offshore north of Cape Ann caused minor damage along the coast near Newbury, Massachusetts, and adjacent areas in New Hampshire.[7] It greatly startled the residents of Boston, but caused little damage (intensity VI).

Cause of Seismicity

The New England Seismotectonic Study showed that the earthquakes are occurring at

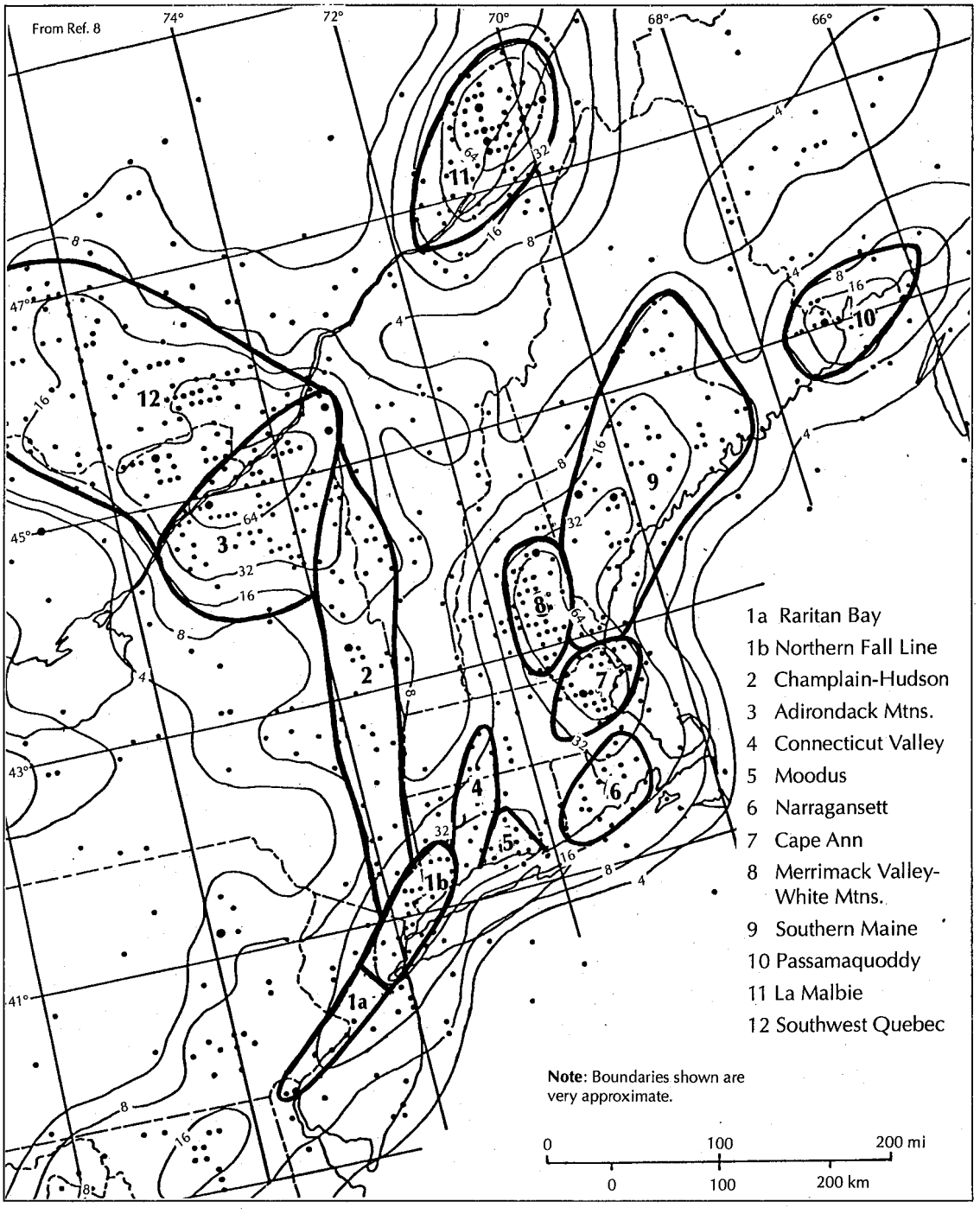


FIGURE 2. Map showing the areas of relatively higher seismicity in the northeastern United States and adjacent Canada.

several local areas of subsidence along the Atlantic Coast of the United States. This subsidence takes place along basement faults whose movements produce earthquakes.[9,14,16,17] The New Hampshire embayment, centered on the New Hampshire coast north of Cape Ann, is one of these subsiding areas and its resulting seismicity appears to be

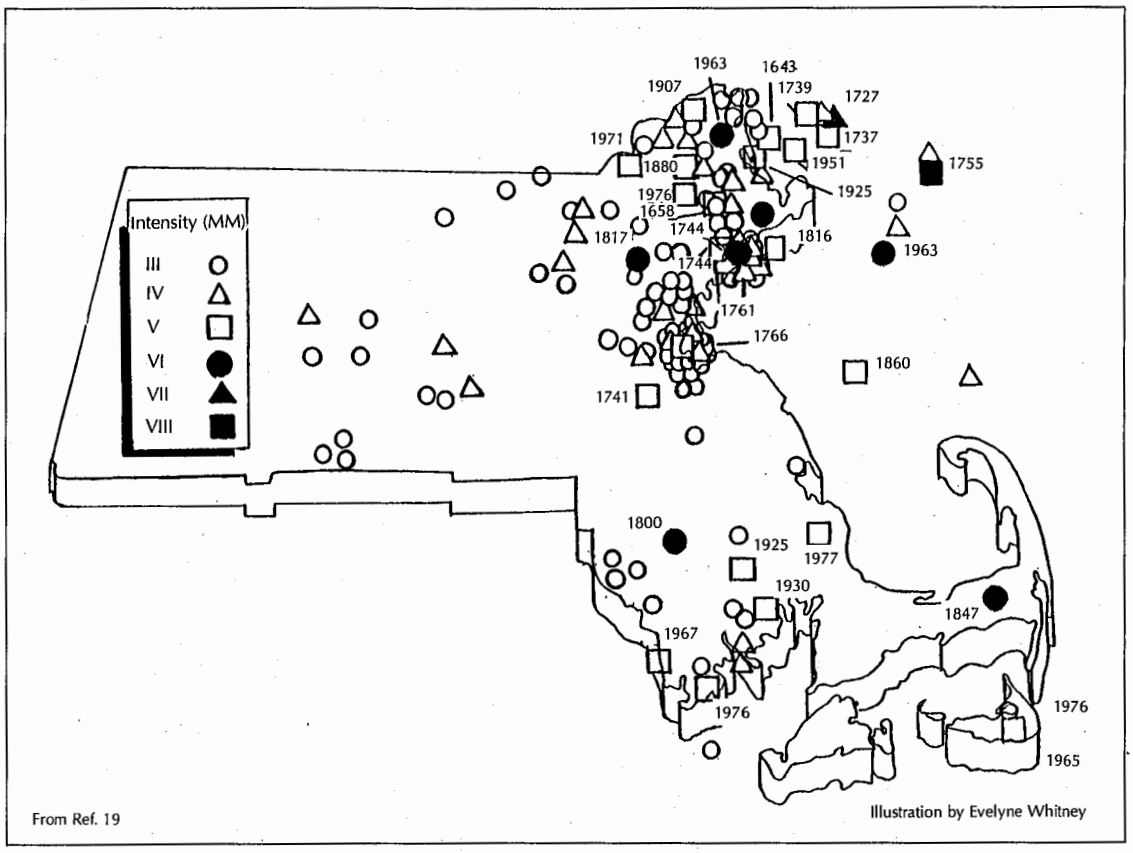


FIGURE 3. Epicentral map of Massachusetts 1534 to 1980 (courtesy the *Worcester Telegram*).

associated with a major structural intersection along a northwest-trending zone of active fractures. The peripheral earthquakes may be caused by local adjustments on various other faults due to the subsidence (see Figure 5).[13,14]

Seismic Hazard

Once the source areas of potentially damaging earthquakes have been located, the seismic hazard to specific sites from activity at these source areas can then be evaluated. This evaluation is achieved by investigating the largest earthquake that is recorded at each source area, determining the maximum credible earthquake that is likely to occur at these areas, establish the rate for attenuating the effects of earthquakes between these areas and the site of concern, and identifying ground conditions at the site that may amplify or dampen these effects.[21]

The time element, or how frequently such a maximum credible event will occur, is not taken into account in this approach. It merely assumes that the maximum credible earthquake may occur "sometime." It thus presents the potential hazard and not the risk. Determining risk involves determining the reasonable return times, a process that has yet to be achieved. The frequency rate is not important for critical structures such as nuclear power plants, dams, hospitals and certain storage tanks that need be built to withstand the maximum credible earthquake, but it may be in non-critical short-term facilities.

The maximum recorded earthquakes for seismic areas in southern New England and La Malbaie, Quebec, are shown in Table 2. The maximum credible earthquakes for those areas that might be a threat to the Boston region are shown in Table 3.[13] The results indicate the maximum expected earthquake for the offshore Cape Ann area is IX and that in the southern part of the area, within the greater Boston metropolitan area, the maximum ex-

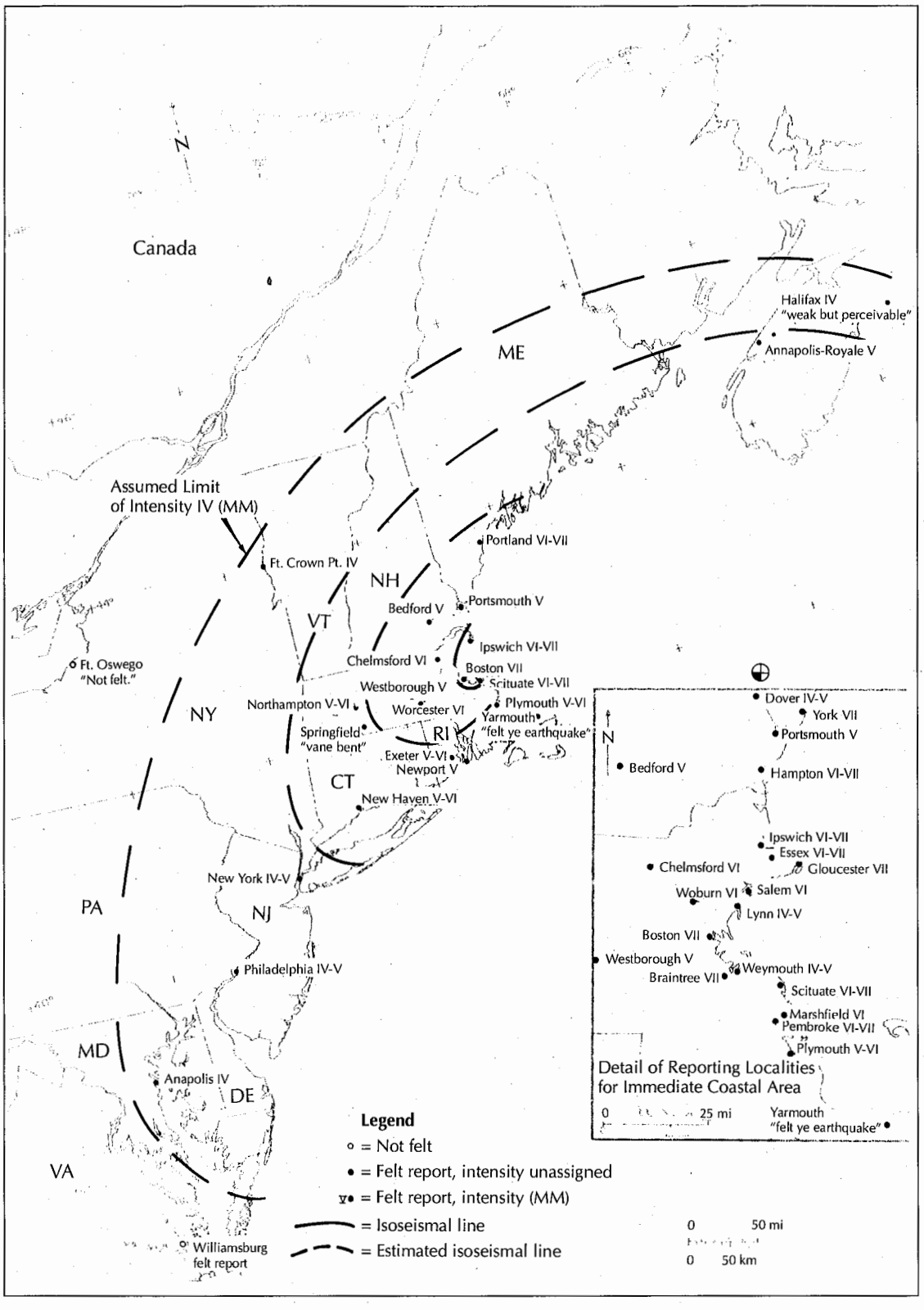


FIGURE 4. Isoseismal map of the November 18, 1755 Cape Ann earthquake.

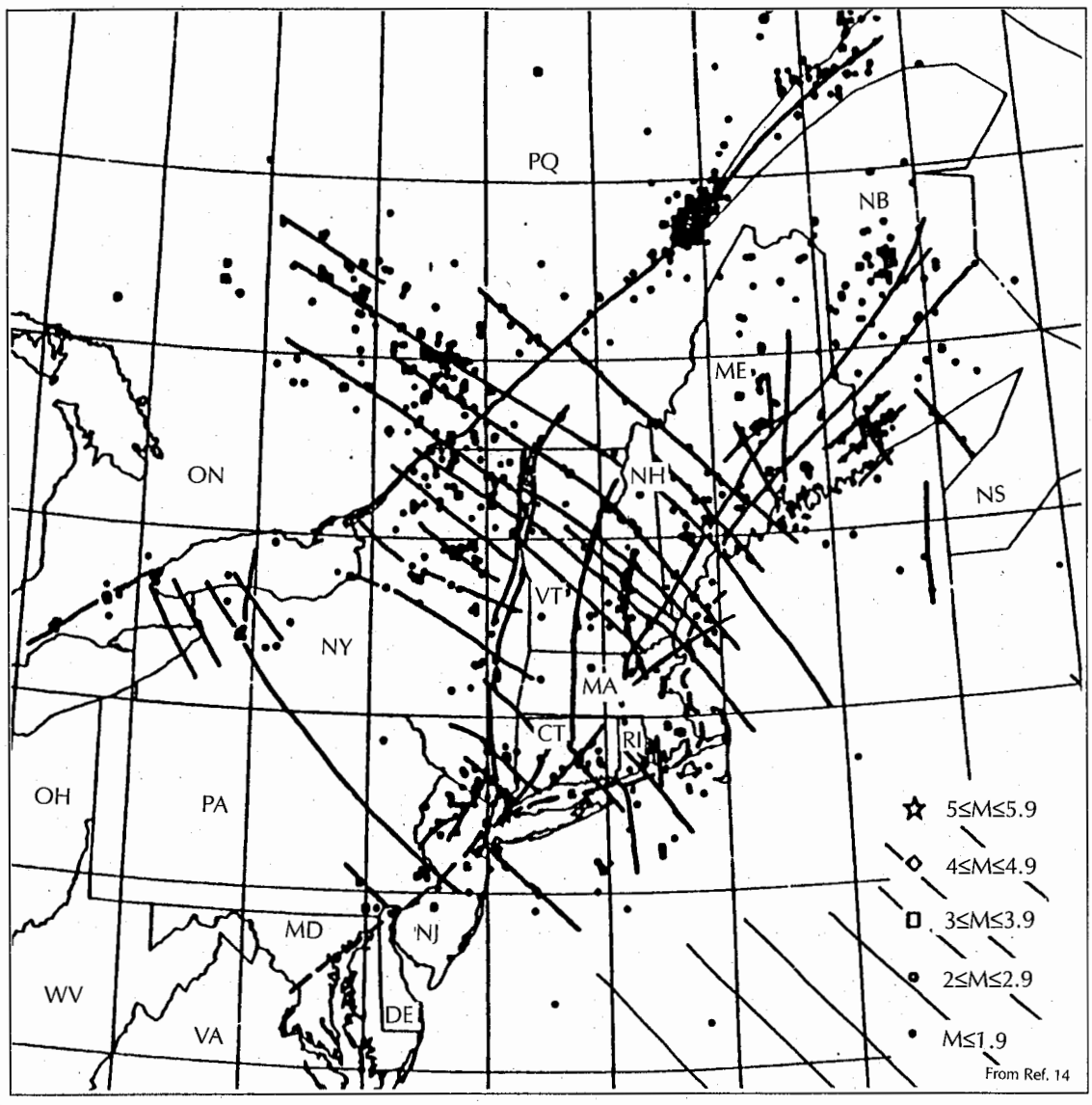


FIGURE 5. Map of New England and adjacent areas showing earthquakes for the period October 1975 to March 1984, the generalized fault and probable fault zones spatially related to earthquakes (solid lines) and the border of the coastal plain deposits (dashed line).

pected is VII (see Figure 6). The intensity X earthquakes experienced in the La Malbaie area are probably near their maximum size.

A maximum credible earthquake in the greater Boston area would produce very local moderate damage. One near Cape Ann off the Massachusetts coast would produce considerable damage in parts of Boston and in select areas of eastern Massachusetts. One in the Ossipee area would cause only very minor damage and one from La Malbaie would cause minor damage with perhaps some moderate damage in structures that are very sensitive to long period waves (see Table 4).

Local Foundation Conditions

The intensity at a site may increase or decrease from the average by as much as two intensity units for extreme variations in ground conditions.[22,23] The different responses to earthquakes depend on the type, density and thickness of the surface layer, degree of water

TABLE 2
Maximum Epicentral Intensities Recorded for Source Areas Near Boston & La Malbaie

Area	Earthquake	Epicentral Intensity	Magnitude (estimate)
Cape Ann			
Greater Boston	Woburn, October 5, 1817	V-VI	M_{bLg} 4.0
	Salem, June 14, 1744	VI	M_{bLg} 4.7
Offshore Cape Ann	Cape Ann, November 18, 1755	VIII	m_b 5.8
Merrimack Valley-White Mountains	Ossipee, December 20 & 24, 1940	VII	m_b 5.4
Narragansett Bay	Wareham, December 25, 1800	VI	
Moodus	East Haddam, May 16, 1791	VI-VII	
La Malbaie	La Malbaie, February 5, 1663	X	

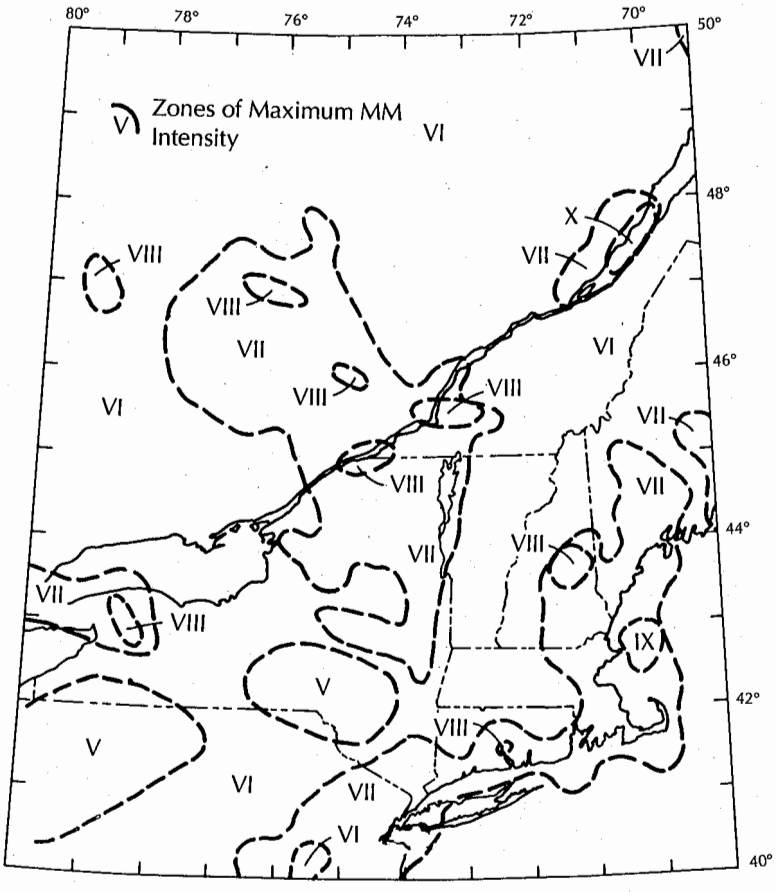
saturation and other factors.

A wide variety of foundation conditions exists in eastern Massachusetts. The Boston metropolitan region contains areas of filled land, marine clay, stratified drift, drumlin till, beach deposits, lake deposits, bedrock and other types of surficial material.[24] If stratified drift or ground moraine is assumed to represent the average foundation conditions with average intensities, then the variation in intensity due to the other material may be approximately as shown in Table 5 which was developed from the general observations found in some other studies.[22,23]

The average intensities from a potential earthquake would be adjusted according to such changes as indicated in Table 5 to approximate the expected local intensities. However, the listing in Table 5 is still too approximate and simplified for an accurate value. It does not indicate thickness, degree of compaction, underlying material, groundwater condition and slope — all of which are all important in ascertaining a reasonably accurate value. Nor does the listing take into account possible liquefaction of sands and clays. Liquefaction of unconsolidated material due to an earthquake load may occur in the area and is addressed in the Massachusetts seismic code.[25] Sand blows, apparently from liquefac-

TABLE 3
Maximum Credible Earthquake for Source Areas Posing the Greatest Hazard to Boston

Area	Epicentral Intensity	Magnitude (m_b)	Distance (for event & range)
Cape Ann			
Greater Boston	VII	5.1	0
Offshore Cape Ann	IX	6.6	80 km (50 mi) (48-112 km; 30-70 mi)
Merrimack Valley-White Mountains	VIII	5.8	176 km (110 mi) (128-184 km; 80-115 mi)
La Malbaie	X	7.3	600 km (375 mi) (536-632 km; 335-395 mi)



Note: Explanation of the values used in describing the maximum epicentral intensity expected on firm ground for seismic source zones in the eastern United States is as follows:

V. Areas of no known earthquakes, generalized boundaries that avoid large known or suspected fracture zones, but otherwise not drawn on geologic basis. The level is chosen at V since rapid freezing may produce this intensity locally.

VI. Areas that have only experienced scattered intensity V earthquakes. Most of the area is considered to rate at intensity V level, but it contains more active local areas with potential for intensity VI (might separate with more detailed zonation).

VII. Areas with identifiable clusters of earthquakes forming a seismic zone that has only experienced intensity VI and probably has potential for intensity VII in the general vicinity of intensity VI or areas where an experienced intensity VII is the estimated maximum. Boundaries are related to geologic features and are generally drawn on them, but modified locally to conform to earthquake distribution.

VIII. Areas with clusters of epicenters within a seismic zone with an experienced intensity at, or near, VII, or areas where an experienced intensity VIII is the estimated maximum. Related to geologic source zone and boundaries generalized around source zone and related earthquakes.

IX. Areas with clusters of epicenters within a seismic zone with an experienced intensity of VIII. Probably approaching the limit of strain build-up in areas of shallow earthquakes. Related to the geologic source zone and boundaries generalized around source zone and related earthquakes.

X. Areas with clusters of numerous epicenters within a seismic zone with an experienced intensity of IX or X. Probable limit of strain build-up on the east coast of the United States. Related to the geologic source zone and boundaries generalized around source zone and related earthquakes.

From Ref. 12

FIGURE 6. Map of the northeastern United States and adjacent Canada showing the maximum expected epicentral intensity.

TABLE 4
General Intensities at Boston from Maximum Credible Earthquakes from Sources Areas Posing the Greatest Hazard

Source Area	Intensity at Boston for Average Foundation Conditions
Cape Ann	
Greater Boston	VI & VII
Offshore Cape Ann	VII
Merrimack Valley-White Mountains	V
La Malbaie	VI

tion, were reported for both the 1727 and 1755 Cape Ann earthquakes. In one case, the sand came up through a clay layer that was reported to be at least 6.5 m (25 ft) thick.

The areas around Boston Harbor that experienced the more severe damage in the 1755 earthquake were built on fill. The enormous amount of filling since 1755 greatly increases the extent of the areas that are more susceptible to earthquake damage.

A map showing the relative stability of ground in earthquakes was prepared for Boston in the 1920s (see Figure 7).[5] A new, but very similar, version was recently prepared for the Massachusetts Civil Defence Agency[26] (see Figure 8) in which basic subsurface geologic groups developed by the Applied Technology Council were followed (see Table 6 and Figure 9).[15] These groups show a slightly more narrow range and an overall greater increase in intensity than shown in Table 5. This increase is mainly due to their having the bedrock set at zero increase in intensity value, rather than at a minus value; in other words, their values are to be added to those values derived from being on relatively better ground rather than as adjustments from the average ground. Because an attenuation rate based on average ground was given for the project, the intensity values are too high. However, the maximum credible earthquake given for this project by the Massachusetts Civil Defense Agency is too low and thus balances the higher values.

TABLE 5
Estimates of Change from Average Intensity Due to Different Foundation Materials in the Boston Area

Foundation Type	Change in Intensity (MM)
Made Land (A), Salt Marsh Deposits (C)	+1.0 to +2.0
Marine Clay	+0.5 to +1.5
Beach Deposits (B), Lake Deposits, Eskers (D)	+0.5 to +1.0
Stratified Drift (E), End Moraine (G)	0 to +0.5
Ground Moraine (H), Drumlin Till (F)	0
Bedrock, Rocky Terrain (I)	0 to -1.0

Note: Letter symbols keyed to Figure 8 in "Geology of the Boston Basin & Vicinity," on page 46.

Local Intensity Estimates

A maximum credible earthquake off the Massachusetts coast in the vicinity of the 1755 Cape Ann event might produce the following intensities for eastern Massachusetts as modified by local conditions. Most of Cape Ann, which has bedrock at or close to the surface, would experience intensity VII effects. However, some areas of stratified drift would experience intensity VIII effects and local fill might fail. The areas west and south of Cape Ann also may have intensities of VII over average ground conditions. Around Boston, where the highest concentration of filled land is located, intensity VIII effects would be widespread (see Figures 7 and 8). There is even the possibility that intensity IX effects would be experienced on very poor ground. The areas of higher intensity include the Back Bay of Boston, much of Cambridge, South Boston, Winthrop and parts of Everett and Lynn and Logan Airport. Damage in these areas would be considerable in ordinary buildings and great in poorly built structures. Tsunamis should pose no problems. Further south and to the west of Boston, including parts of Milton, Newton, Belmont and Lexington, intensity VII effects would predominate as they would on Cape Cod. Local areas south of Boston, particularly where inten-

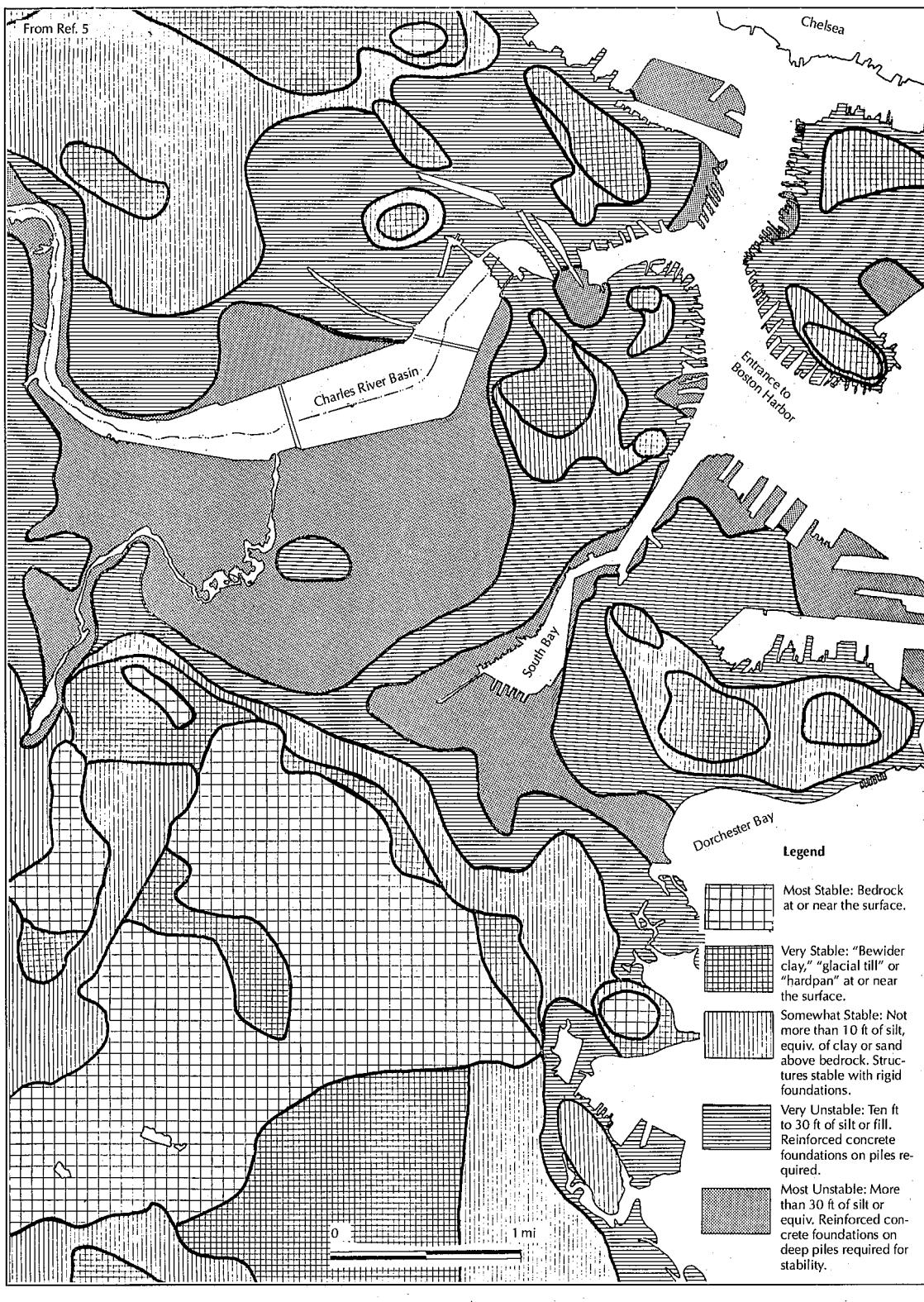


FIGURE 7. Map of Boston and vicinity showing the probable relative stability of the ground during an earthquake.

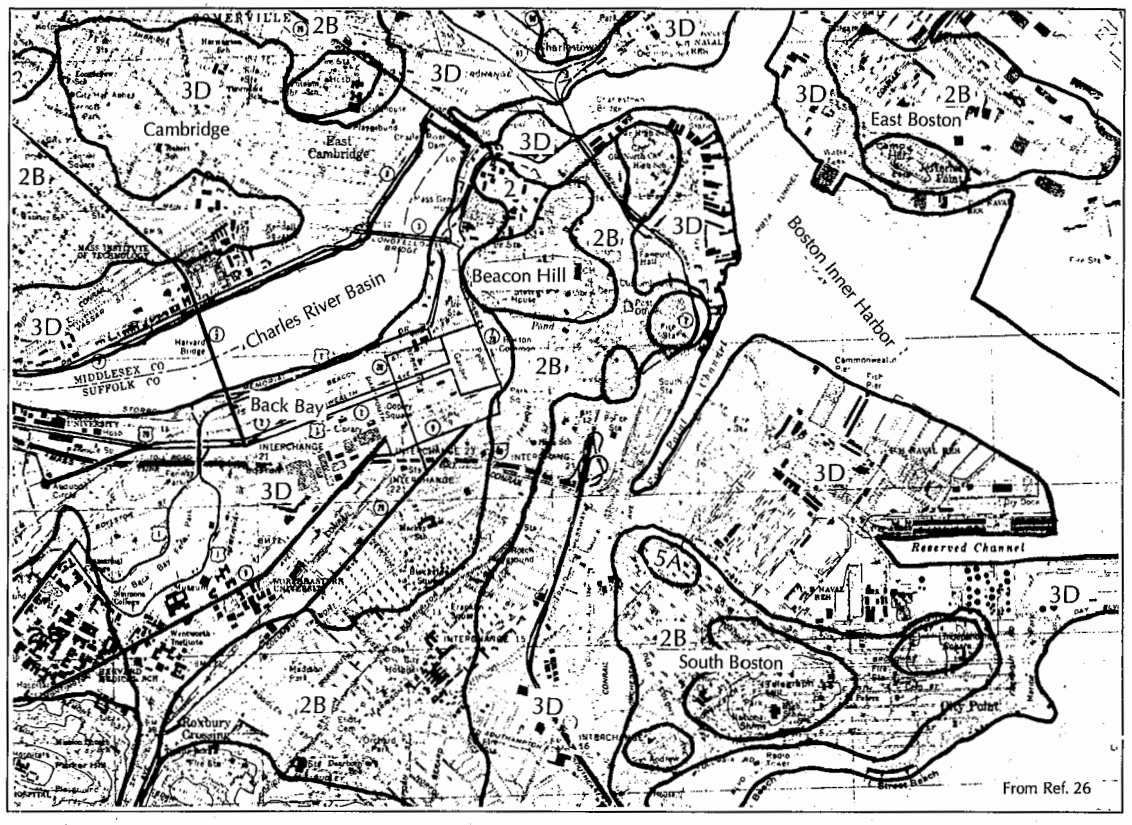


FIGURE 8.Map of the metropolitan Boston area showing numbered areas where the overburden may result in a site intensity of +2. The number (and letter if present) indicates generalized subsurface geologic conditions as shown on the figure. Such areas have the potential to experience ground shaking corresponding to Modified Mercalli intensity VIII for an earthquake magnitude 4.25 m_b from a source off Cape Ann.

TABLE 6
Estimates of Unit Increase in Intensity Due to Different Foundation Materials

Foundation Type	Changes in Intensity (MM)
Geologic conditions where combined thickness of non-engineered fills, organic soils, soft silts or clays, & loose granular soils exceeds a 30 to 50 ft (9.1 to 15.2 m) range in thickness.	+2
Organic soils less than 30 ft (9.1 m) in thickness overlying glacial till or bedrock.	+1
Material at all sites not included in other groups of this list.	+1
Bedrock, glacial till (exclusive of ablation till) & deposits of other dense or stiff soils less than a 30 to 50 ft (9.1 to 15.2 m) range in thickness & overlying bedrock.	0

From Ref. 26

sity VII was recorded in 1755, should expect intensity VIII. Intensity VIII effects would also occur very locally to the west of Boston and locally on the very thick outwash deposits forming Cape Cod.

Ground Motion

The actual ground motion arising from a maximum credible earthquake affecting the Boston area can be estimated by applying desired ground motion-intensity relations to the expected intensities. Relations for acceleration, velocity and duration can be used as a guide in designing construction to withstand a potential earthquake. Such relations have been formulated for the central United States.[27] Krinitzsky and Chang reviewed such strong motion data and produced a series of curves using some

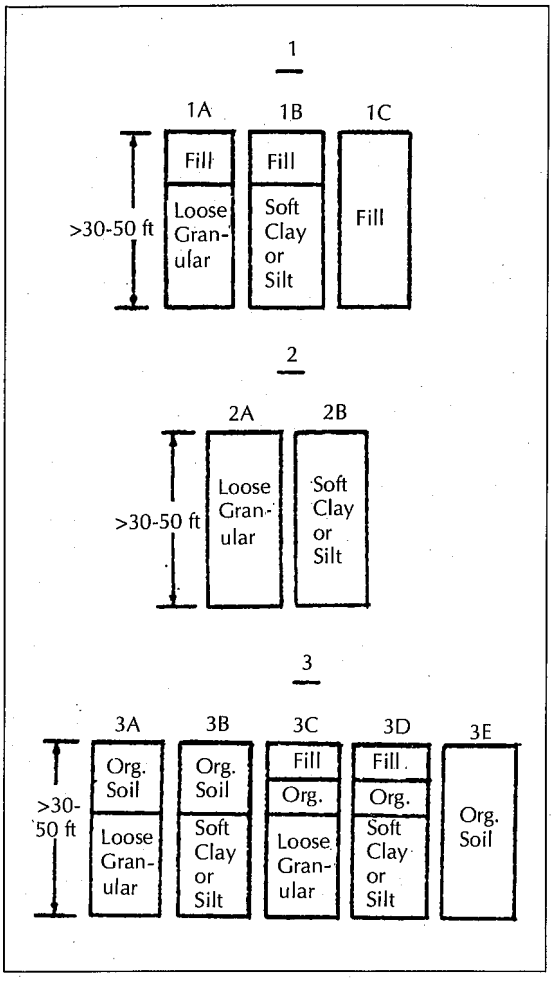


FIGURE 9. Diagram showing the overburden sequences in the Boston area that result in a site intensity increase of +2.

data from New England relating ground motion parameters to intensity for both near and far earthquakes at soft and hard sites.[28,29]

Summary

Eastern Massachusetts is a region of moderate earthquake hazard. The nearly 400-year historical record lists damaging earthquakes in the region in 1638 and 1663 from earthquakes in La Malbaie, Quebec, and in 1727 and 1755 from events north of Cape Ann. These same source areas continue to be seismically active.

Potential earthquakes in the Cape Ann area pose the greatest threat to Boston and eastern Massachusetts. A maximum credible event there of intensity IX would cause a general intensity VII effect in the Boston region with intensity level VIII, and possibly level IX, effects over the extensive areas of filled ground. Large earthquakes at La Malbaie may cause average intensities of VI in Boston and from V to VII in eastern Massachusetts. Long period motions from future La Malbaie events might possibly cause damage to tall buildings and other structures that are susceptible to such motions. Potential earthquakes in central New Hampshire may only produce minor damage in eastern Massachusetts.

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REFERENCES

1. Winthrop, John, "An Account of the Earthquake Felt in New England and the Neighboring Parts of America on the 18th of November 1755," *Phil. Trans. Roy. Soc.*, V. 50, Pt. 1, 1757, pp. 1-18.

2. Hobbs, W.H., *Earthquakes: An Introduction to Seismic Geology*, D. Appleton & Co., 1907, New York.

3. Crosby, I.B., "The Earthquake Risk in Boston," *Journal of the Boston Society of Civil Engineers*, 1923, V. 10, No. 10.

4. Crosby, I.B., "The Danger of Earthquakes in New England," *Sci. New Ser.*, V. 63, 1926, pp. 186-187.

5. Crosby, I.B., "Map of Boston, Massachusetts, Showing Probable Relative Stability of Ground in Earthquakes With Bed-Rock and Surface Contours," in Freeman, J.R., *Earthquake Damage and Earthquake Insurance*, McGraw-Hill, N.Y. & London, 1932.

6. Heck, N.H., "Earthquake History of the United States, Part I," *U.S. Coast Geodetic Survey, Ser. N. 609*, 1938.

7. Boston Edison Company, "Historical Seismicity of New England, Boston Edison Co., Preliminary Safety Analysis Rept. Pilgrim Unit No. 2, BE-SG7601," U.S. Nuclear Regulatory Commission Docket No. 50-471, 1976, 641 pp.

8. Barosh, P.J., "Earthquake Zonation in the Northeastern United States," in Hatheway, A.W., ed., *Engineering Geology in New England*, ASCE Ann. Meeting, Boston, MA, Preprint 3602, 1979, 22 pp.

9. Barosh, P.J., "Cause of Seismicity in Eastern United States: A Preliminary Appraisal," in Beavers, J.E., ed., *Earthquakes and Earthquake Engineering: The Eastern United States*, Ann Arbor Science, Ann Arbor, Mich., V. 1, 1981, pp. 397-417.

10. Harrison, W., *et al.*, "Geology, Hydrology, and Mineral Resources of Crystalline Rock Areas of the Northeastern United States," Argonne Nat'l Lab., Rept. ANL/ES-136, Parts 1 and 2, 1983, 413 pp.

11. Krinitzsky, E.L., & Dunbar, J.B., "Geological-Seismological Evaluation of Earthquake Hazards at Franklin Falls Damsite, New Hampshire," U.S. Army Corps of Engrs., Waterways Experiment Station Tech. Rept. GL-86-16, 1986, 50 pp. & appendices.

12. Barosh, P.J., "Seismic Source Zones of the Eastern U.S. and Seismic Zoning of the Atlantic Seaboard and Appalachian Regions," U.S.Army Corps. of Engineers, Waterways Experiment Stations, Misc. Paper S-73-1, *State-of-the-Art for Assessing Earthquake Hazards in the United States, Rept. 21*, 1986, 302 pp.

13. Barosh, P.J., "Summary and Analysis of the 1982 Gaza, New Hampshire, Earthquake," in Krinitzsky, E.L., and Dunbar, J., *Geological-Seismological Evaluation of Earthquake Hazards at Franklin Falls Damsite, New Hampshire*, US Army Corps of Engineers, Waterways Experiment Station Tech. Rept. GL-86-16, 1986, Appendix C, pp. C1-C75.

14. Barosh, P.J., "Neotectonic Movement, Earthquakes and Stress State in the Eastern United States," *Tectonophysics*, V. 132, 1986, pp. 117-152.

15. Applied Technology Council, "Tentative Provisions for the Development of Seismic Regulations for Buildings," Applied Tech. Council Rept. ATC 3-06, NBS Spec. Pub. 510, NSF Pub. 78-8, 1978.

16. Barosh, P.J., "Seismicity and Earthquake Hazard of Boston and Adjacent Eastern Massachusetts," *Association of Engineering Geologist, Bull.*, in press.

17. Barosh, P.J., "Neotectonic Movement and Earthquake Assessment in the Eastern United States," Geolog. Society of America, Special Paper, E. L. Krinitzsky and D. B. Slemmons, eds., in press.

18. Wood, H.O., & Neumann, F., "Modified Mercalli Intensity Scale of 1931," *Seismol. Soc. America Bull.*, V. 21, N. 4, 1931, pp. 277-283.

19. Nottis, G.N., ed., "Epicenter Map of Northeastern United States and Southeastern Canada, Onshore and Offshore; Time Period 1534-1980," New York Geol. Survey, N.Y. State Museum Map and Chart Ser. N. 38, 1983, 2 sheets, scale 1:1,000,000.

20. Chiburis, E.F., "Seismicity, Recurrence Rates and Regionalization of the Northeastern United States and Adjacent Southeastern Canada," U.S. Nuclear Reg. Comm. Rept. NUREG/CR-2309, 1981, 76 pp.

21. Barosh, P.J., "Use of Seismic Intensity Data to Predict the Effects of Earthquakes and Underground Nuclear Explosions in Various Geological Settings," *U.S. Geol. Survey Bull. 1279*, 1969, 93 pp.

22. Popov, V.V., "Engineering-Geologic Criteria of Detailed Seismic Regionalization," in *Problems in Eng. Seismology*, No. 2, Akad. Nauk SSSR Inst. Fiziki Zemli Trudy 5 (172), 1959, pp. 81-93; also in Ref. 21.

23. Medvedev, S.V., "Determination of the Intensity Scale (Number) of Earthquakes in the USSR," Moscow, Akad. Nauk SSSR Council on Seismology, 1961, pp. 103-138; translation, Nov. 1962, U.S. Atomic Energy Comm. Transl. Ser. AEC-tr-5424, p. 124-156.

24. Kaye, C.A., "Surficial Geologic Map of the Boston Area, Massachusetts," U.S. Geol. Survey Open-File Rept. 78-111, 1978, approx. scale 1:1,000,000.

25. Soydemir, C., & LeCount, P.L., "Massachusetts Building Code on Liquefaction: A Design Engineer's Overview," in Beavers, J.E., ed., *Earthquakes and Earthquake Engineering: The Eastern United States*, Ann Arbor Sci., Ann Arbor, Mich., V. 2, 1981, pp. 665-681.

26. Haley and Aldrich, Inc., "Isoseismal/Geologic Conditions Maps for Eastern Massachusetts," unpublished report for the Mass. Civil Defense Agency, 1983, scales 1:2,500 and 1:24,000, 6 pp.

27. Nuttli, O.W., "The Relation of Sustained Maximum Ground Acceleration and Velocity to Earthquake Intensity and Magnitude," U.S. Army Corps of Engrs., Waterways Exper. Sta., Misc. Paper. S-73-1, Rept. 16, 1979, 100 pp.

28. Krinitzsky, E.L., & Chang, F.K., "Parameters for Specifying Intensity-Related Earthquake Ground Motions," Dept. of the Army, U.S. Army Corps of Engineers, Waterways Experiment Station Misc. Paper S-73-1, *State-of-the-Art for Assessing Earthquake Hazards in the United States, Rept. 25*, 1987, 43 pp., Appendix 41 pp.

29. Krinitzsky, E.L., & Marcuson, W.F., III, "Principles for Selecting Earthquake Motions in Engineering Design," *Asso. Eng. Geol.*, V. 20, 1983, pp. 253-265.

Massachusetts Earthquake Design Codes

Geologic conditions at a site allow for alternatives in meeting seismic design regulations.

S.A. ALSUP & K.E. FRANZ

Design requirements for new or altered structures are included in Chapter 780 of the Commonwealth of Massachusetts Regulations under Article 7, Sections 716.0 and 720.0.[1] The design considerations for earthquake resistant construction, with reference to those sections, are included in Sections 725.0 and 2204.0 of Article 22. The codes were adopted as of September 1, 1980 and no formal changes to them have been made since then. A lengthy and very thorough preparation of the seismic design requirements preceded adoption of the codes. The critical elements were considered by design panels consisting of academicians, seismologists, structural design engineers, architects, soils engineers, geologists and government representatives who were familiar with the procedures and requirements for formalizing technical design criteria.

The objective of the provisions in the codes is to protect the lives and safety of the population of Massachusetts. The provisions apply to all structural designs except one- and two-story structures and minor accessory buildings. The rehabilitation or modification of existing structures falls under the codes — *i.e.*, when a building is undergoing renovations and/or additions the new seismic design rules apply. Technical requirements in the codes are often somewhat complicated and cannot be readily applied by the non-technical person. In fact, the satisfaction of the requirements necessitates the services of registered professional engineers in a number of instances. The codes also require careful consideration of the specific geological conditions that may be encountered in the Massachusetts terrain.

The primary factor considered in the codes for ground motion from earthquakes is the horizontal force that might be introduced into a structure. This force is determined in terms of a horizontal force factor (related to the structural bracing and framework), base shear (related to the fundamental period of oscillation of the structure or structural elements), a foundation material factor (material class and thickness beneath the structure) and the weight of the structure (dead load with consideration for non-permanent loadings).

TABLE 1
Assumed Bearing Values of Foundation Materials

Class of Material	Tons/sq. ft.
Massive crystalline bedrock including granite diorite, gneiss, trap rock and dolomite (hard limestone)	60
Foliated rock including limestone, schist and slate in sound condition	40
Sedimentary rock including hard shales, sandstones and thoroughly cemented conglomerates	20
Soft or broken bedrock (excluding shale) and soft limestone	20
Compacted, partially cemented gravels, and sand and hardpan overlying rock	10
Gravel, well-graded sand and gravel mixtures	6
Loose gravel, compact coarse sand, loose sand	4
Loose coarse sand, loose sand-gravel mixtures and compact fine sand (confined)	2
Loose medium sand (confined)	1
Loose fine sand	(+)
Hard clay	4
Medium stiff clay, stiff varved silt	2
Soft clay, soft broken shale	1
Soft inorganic silt, pre-loaded material, shattered shale or any natural deposit of unusual character not provided for herein	(+)
Disturbed varved silt	0
Compacted granular fill	(2-5+)

From Ref. 1

Each of these factors may be derived through the application of formulas and numerical factors for general classes of conditions, or alternate factors may be developed based on the measured or modelled dynamic reactions of the structure assuming a given level of dynamic input. It is interesting to note that if an essentially "pre-fabricated" design were being considered (one that would include the horizontal force factor, base shear and weight), the critical element that would make the design acceptable or not acceptable under the codes would be the geological conditions present (soil factor).

The codes also identify design provisions for various types of construction materials (concrete, steel, masonry, timber, pre-fabricated, *etc.*) and for the different design elements that may be employed (beams, columns, walls, foundations and interconnections). All such design considerations are based on the potential lateral accelerations that could be applied to a structure under the occurrence of a "design earthquake" that includes a basic 0.12 g lateral acceleration modified by a height-related fundamental period spectrum. Different methods may be used to demonstrate a satisfactory design that would not cause risk to life and public safety in the opinion of a qualified registered engineer. Sophisticated dynamic methods may be employed, or the analysis may be as simple as demonstrating that similar structures on a similar geologic foundation have been able to withstand the basic level of disturbance without damage.

For earthquake design purposes in the Commonwealth of Massachusetts, foundation materials (or "soil" in engineering parlance) are considered to be either Class A or Class B materials.

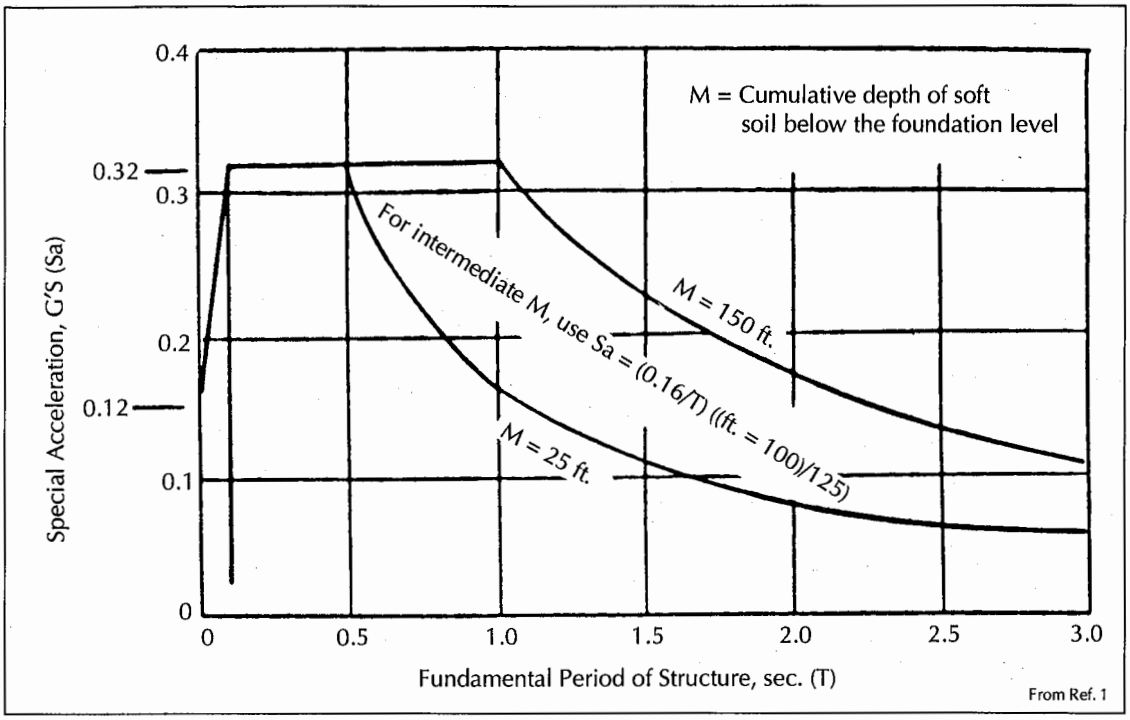

FIGURE 1. Diagram showing the acceleration-period relation of a structure for different depths of soft soil below the foundation level.

Class A materials include massive igneous rocks and conglomerate, slate and argillite, glacial till, gravel or well-graded sand and gravel, coarse sand, medium sand, clay and compacted granular fills. Granular materials must be dense to very dense; clay must have an undrained shear strength of at least 488 g/cm^2 (1,000 lbs/ft^2); and, compacted granular fills must have proper placement and demonstrate a bearing strength. Typical bearing strengths for these materials are shown in Table 1.[1]

All other materials are considered to fall within Class B. This classification requires an increase in the lateral load factor by 50 percent for the evaluation of the design. The thickness of soft foundation material affects the acceleration-frequency relations as well as other ground motion factors at any site from an earthquake (see Figure 1). Modifications to this requirement may be made if satisfactory investigations and evaluations of the actual material conditions are performed that demonstrate that a lesser increase is proper.

Liquefaction is given special consideration in the codes. The presence of clean saturated fine to medium sands in the subsurface may indicate the need for considering liquefaction in earthquake-resistant designs. The Standard Penetration Test (blows/30 cm) in such material, with an adjustment for the depth of such material below the ground surface, is considered the primary guideline for determining liquefaction potential. Material of this type is excluded from consideration if it is present beneath level ground at depths in excess of 18 meters (60 feet). Compacted granular fills may also be excluded, provided that tests or other procedures demonstrate that the risk of liquefaction is not present. Sites where such materials are present, and a clear exclusion of the liquefaction risk cannot be established, must be investigated in some detail. Alternate solutions are given in the codes if liquefaction poses a risk. These alternatives include using foundation designs that can accommodate liquefaction, densification of the deposit to reduce the risk or even total removal of the materials to make the site safe. Sites that are underlain by saturated sands and subject to potential slope instabilities, such as sloping ter-

rain and man-made excavations, must be evaluated by a registered professional engineer for liquefaction and slope instability risks.

Building codes are well known to be excessively rigid, sometimes not very applicable to the average situation, and even counterproductive when enforced by well-intentioned but poorly-informed inspectors. The seismic design codes for earthquake load in Massachusetts, however, have been prepared to furnish a basic guideline that will provide a minimum standard for the evaluation of designs in terms of resistance to damage in the event an earthquake occurs. Additional beneficial features, not always common to the building codes, include flexibilities that permit, and encourage, participation by qualified professionals to propose variations to the basic guidelines. This mandate permits the use of information from well-documented site investigations to modify design requirements, as well as the application of more sophisticated analytical methods in weighting the suitability of a design in terms of the planned structure and the geological conditions present.

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REFERENCES

1. Commonwealth of Massachusetts, *Massachusetts State Building Code*, 3rd ed., Massachusetts State Building Commission, Boston, April 1979.

Environmental Concerns Imposed by Boston Area Geology

The geologic conditions that have affected settlement and expansion have had a significant effect on the area's environmental management system.

DAVID WOODHOUSE

Many environmental concerns arose with the growth of the Boston area. The expansion of the City of Boston into a large metropolitan area during the 19th century made Boston a leader in the development of a city-wide water and sewerage system. Boston also led the nation in developing a structured solid waste program that was built in conjunction with the water and sewage system. In recent years, a host of new laws regulating the operation of these environmental management systems have resulted in a need for enlarging them further. In addition, new state and federal regulatory statutes have had a significant impact on the treatment of the environmental problems in the metropolitan Boston area.

Water Supply

Boston owes its founding to its abundant fresh water springs that attracted John Winthrop and his Puritan followers (see "The History of Boston: The Impact of Geology" on pp. 33-38). They had abandoned their settlement across the Charles River in Charlestown because of the sickness that was caused by drinking water from a spring that was exposed only at low tide. The settlers soon learned that the drumlin till that composes most of Charlestown consisted of a material that was not likely to provide a good source of fresh water.

The colonists living on the Shawmut Peninsula derived their water from shallow dug wells that produced water of good quality under artesian pressure.[1] Some households in colonial Boston drew water from the town well located at what is now called Washington Mall and the town spring in Spring Lane downhill from Washington Street. Wells in the lower part of Boston tended to be somewhat brackish since they were close to sea level and were influenced by salt water intrusion.

The early colonial wells were lined with slabs of local rock, later ones progressing to cobbles retained by wooden sheathing and then to loosely mortared brick. Pumps consisting of hollow log pipes with a moving wooden

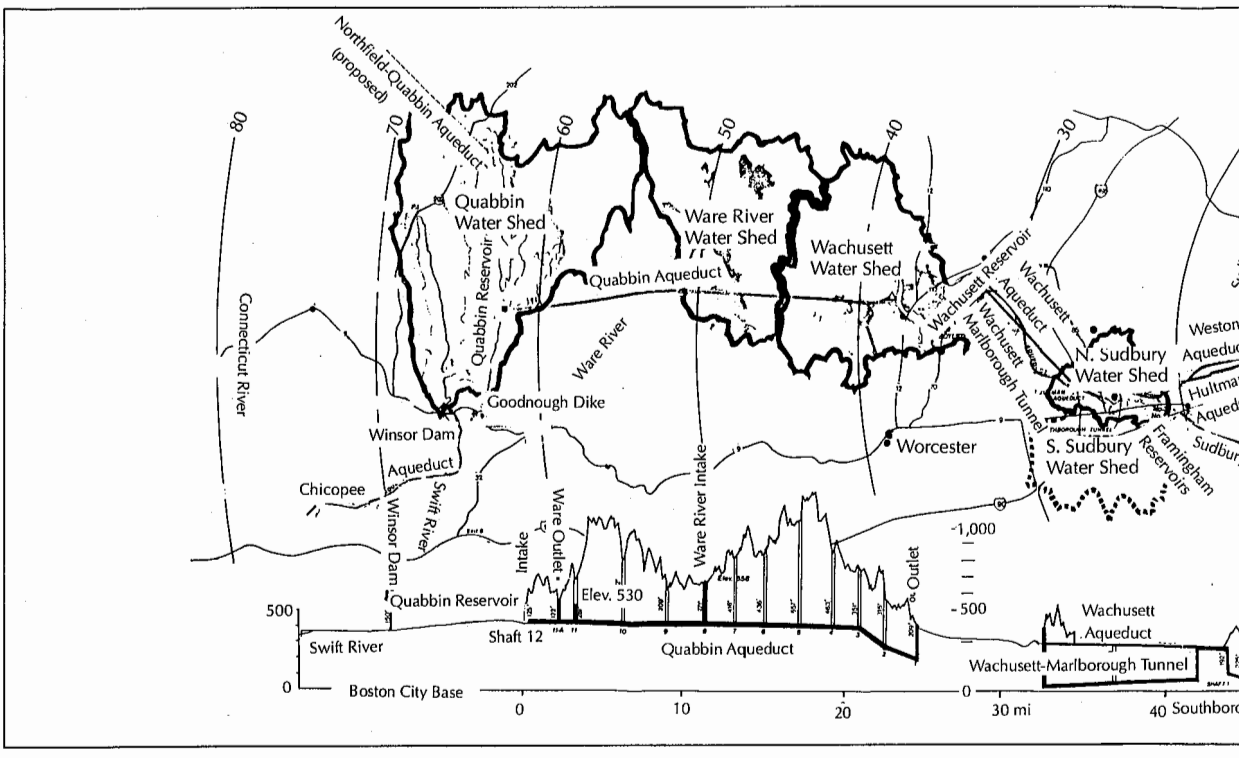


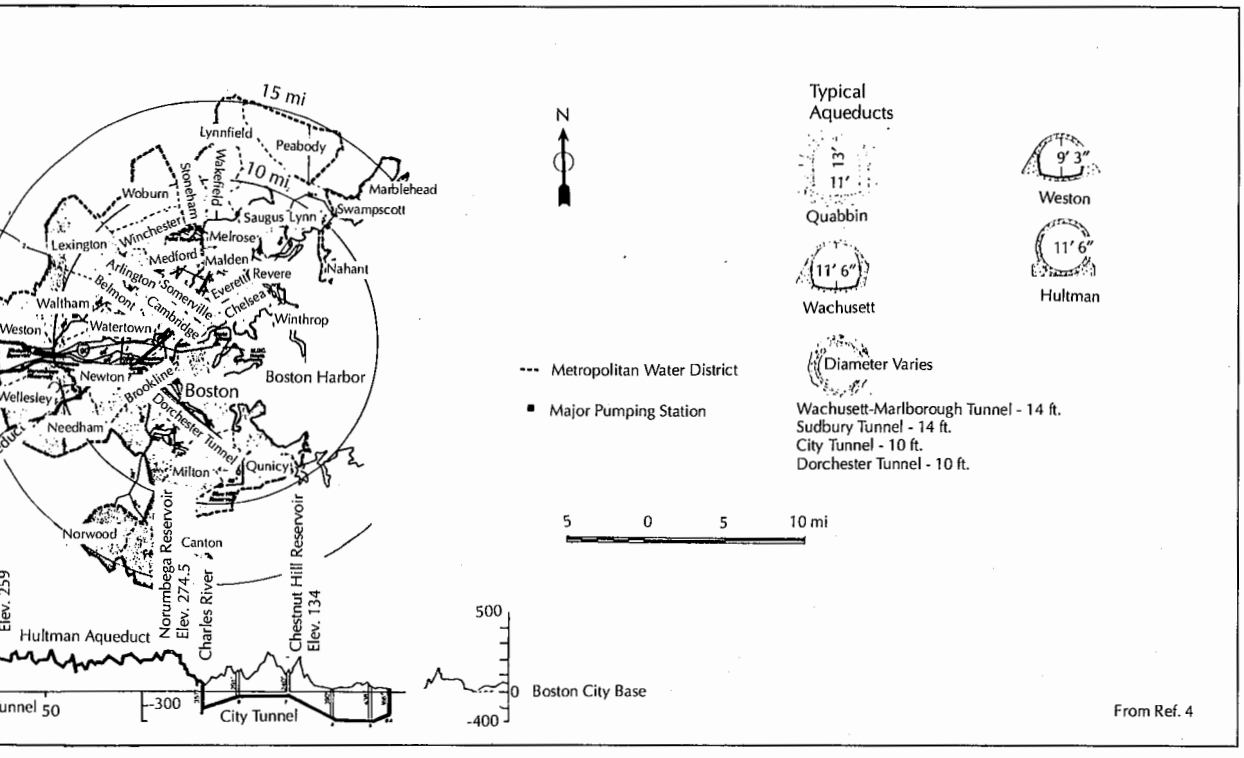
FIGURE 1. Map of eastern Massachusetts showing the aqueduct tunnels and the reservoirs of the Metropolitan District Commission water system.

piston and flap-valve have been found in the deep wells. Thin lift-rods were attached to the valved piston and activated by a pump handle at the surface. Log pipes were used for water until the mid-19th century.

A privately owned water company was organized in Boston in 1795 that brought water downslope from Jamaica Pond through a 64-km (40-mi) long distribution system of hollow pipes. However, wells continued to be used on the higher elevations such as Beacon Hill because of a lack of pressure. In 1848, the city initiated a new public water supply system from Lake Cochituate, about 24 km (15 mi) west of the city. A reservoir was built on top of Beacon Hill and water was pumped up to it, to flow from there by gravity to all parts of Beacon Hill. Water was also brought in from the Sudbury River, about 40 km (25 mi) west of Boston.

In the 1890s, it was realized that the water supply problem could only be solved on a metropolitan basis. The Metropolitan Water District (MWD) was created in 1895 and work began on a major expansion: the building of the Wachusett Reservoir 64 km (40 mi) west of Boston and an aqueduct to connect the new reservoir to Boston (see Figure 1). The water quality of the Wachusett Reservoir is now endangered because the Metropolitan District Commission (MDC), which now manages the reservoir, owns only eight percent of the watershed. Development is taking place in the watershed at a rate unequaled in any other part of the state. Algal growth in the reservoir recently has given Boston's system an odor.

In 1926, the demand for more water led to a 20-year program for the construction of the Quabbin Reservoir 104 km (65 mi) west of Boston in a valley in central Massachusetts that was occupied by six towns. The water supply for the reservoir was provided by impounding the water of the Swift River, a tributary of the Connecticut River. The impounding of the river was accomplished by the construction of an earth dam referred to as the Winsor Dam and Goodnough Dike. In 1946, when the reservoir was constructed and filled, it had a capacity of 412 billion gallons, and covered a surface area



of 10,000 hectares (39 square miles). Today, it provides water for 44 cities and towns, 34 of which are located within the metropolitan area.

The water is transported to the metropolitan area by a series of aqueducts and tunnels. The tunnels and aqueducts are 3.7 to 4.3 m (12 to 14 ft) in diameter, and local distribution tunnels within Boston are 3 m (10 ft) in diameter (see Figure 1).

Historically, the water quality at the Quabbin Reservoir has been excellent and contains none of the road salt or organic chemicals that plague other water supplies within the state. The reason for the excellent quality is due to the remote location of the reservoir and the superior management of the watershed that feeds the reservoir.

However, recently trace metals such as mercury have been found in the waters of the Quabbin and the Wachusett Reservoirs. The metals are concentrating in fish and has resulted in at least a temporary ban on fishing and consumption of the fish. It is not known whether the metals are originating from materials in the towns inundated by the dammed Swift River or from airborne pollution. The only other water quality problems that have occurred have been related to local distribution systems. Elevated levels of lead occur in Boston and Cambridge where much of the old distribution system consists of lead pipes. Tetrachlorethylene has been detected in some of the most recently installed dead-end sections of the distribution system where pipes with a plastic coating to reduce corrosion were used. Current water treatments include chlorination, ammoniation and fluoridation.

Wastewater Management

Wastewater management in the greater Boston area is not unlike that of other communities of similar age. In concert with the development of the city and its environs, which was complicated by the filling of the mudflats and other tidal areas, a sanitary sewer system evolved. The tidal flats of the Charles River in the Back Bay became so polluted and offensive to the City Fathers that the area was filled in during the latter half of the 19th century. The need for a sewer system to serve the entire area was emphasized in a mid-19th century report from the Consulting Physicians of the City of Boston, which recommended a plan that would "carry the sewerage out so far at sea that point of

discharge will be remote from dwellings, and beyond the possibility of doing harm to the citizenry."

A plan developed in 1875 called for two main drainage systems: one to serve the area north of the Charles River, and one to serve the area south of it. Both systems were to discharge untreated sewage into the ocean on the outgoing tide. The southerly system, termed the Boston Main Drainage System, was constructed first and was placed into operation in 1885, discharging into Boston Harbor at Moon Island. The North Metropolitan Sewer District was created to serve the northern area and was placed into operation in 1895, and it discharged into Broad Sound from Deer Island near the town of Winthrop. In the same year, both districts were united under one administration, called the Metropolitan Sewer District (MSD), which served 18 cities and towns.

By 1904 the MSD had expanded and constructed another flow release point at Nut Island. The Nut Island plant was to serve the additional southern and western towns that were accepted into the system. In 1907, new regulations were formulated that prohibited the combined sewer construction. In 1910, a new treatment plant was constructed at Deer Island for better waste dispersion. In 1919, the Metropolitan District Commission (MDC) was established. It was replaced by the Massachusetts Water Resources Authority (MWRA) in 1986.

Over the years, the MDC upgraded the disposal plants: Nut Island in 1949, Deer Island in 1952, and remote headworks for Deer Island in 1968. The Moon Island plant was phased out with the operation of the new remote headworks. In conjunction with the upgrading of the various treatment plants, new collection, routing and outfalls were constructed (see "Tunnelling Projects in the Boston Area" on pages 100-117 for a summary of the deep tunnels constructed for the MDC). Today, the MWRA is studying and implementing various systems for the upgrading of the treatment and disposal system for compliance with new environmental requirements for tertiary treatment (see "Boston Harbor Cleanup: Use or Abuse of Regulatory Authority?" on pages 25-32 for a critique of the proposed cleanup plan.)

Boston remains the only major city in the United States to still dump raw sewage. This sewage is dumped mainly into Boston Harbor. The treatment plants do not meet Clean Water Act standards since they are antiquated and stressed beyond their capacity. The city also mixes storm water and sewage that is discharged at high tide at many outfalls along the waterfront.

The U.S. Environmental Protection Agency (EPA) has cited the city and the Commonwealth of Massachusetts for non-compliance with federal regulations and the MWRA was formed in 1986 to clean up the harbor in response to a Federal Court Order. The EPA now has approved a plan by the MWRA to pump sewage treated at a new Deer Island Plant into Boston Harbor about 13 km (8 mi) east of the island, maximizing the dispersion of the effluent and minimizing potential adverse water quality and shoreline impacts.

The EPA has approved locations proposed by the MWRA for tunnels to bring sewage from Nut Island to Deer Island and to transport effluent from the Deer Island plant to the outfall pipe in the harbor beyond the Graves, the outermost island in Boston Harbor. The proposed tunnel would be the longest of its kind in the world, about 13 to 16 km (8 to 10 mi) long and 7.6 m (25 ft) in diameter. The invert would be in the bedrock about 30 m (100 ft) below the ocean floor.

The tunnel is expected to discharge 500 million gallons of treated effluent daily and to be operational by 1995. The existing Deer and Nut Island treatment plants currently discharge 450 million gallons of inadequately treated wastewater and 70 tons of dry sludge into the harbor every day. Projected costs for the ocean outfall range from $389 to $468 million.

The MWRA today serves 104,000 hectares (400 square miles) and 43 member communities that make up the Metropolitan Sewer District. The system's 2.1 million inhabitants generate 400 million gallons a day of flow. The MWRA operates 8,900 km (5,578 mi) of interceptors, which provide routing to the Deer Island and Nut Island treatment plants.

Solid Waste Disposal

Throughout its long history, Boston has been

faced with the problem of waste disposal that dates back to colonial times. Solid waste had been dumped in the marshes and tidal flats around Boston. Causeway Street near North Station is built over a fill of street sweepings. Old dumps exist throughout Boston, some of which are still generating methane gas and burning underground. The University of Massachusetts at Boston was built on the old Columbia Point dump with a foundation that was designed to allow for the venting of gas.

During the 20th century up to the enactment of the Clean Air Act, Boston either trucked its wastes for incineration to the dumps situated on the tidal marshes in Lynn and Saugus, north of the city, or to the city incinerator located off Massachusetts Avenue. Boston now trucks its waste, using private contractors, to eleven landfills and transfer stations located in the surrounding suburbs. The city also ships garbage to Spectacle Island in Boston Harbor where it is open-burned since the city chose not to upgrade its incinerator to comply with the Clean Air Act because of costs. A resource (RESCO) recovery plant in Saugus accepts some of the Boston waste, burning it to generate steam for the General Electric Company.

With many landfills closing because of the new environmental laws, Boston is currently reviewing options to dispose of its solid waste. These options include the construction of a resource recovery plant at a site acceptable to the citizens of Boston, a difficult task to say the least.

Wetlands & Shore Protection

Boston has not had to take any extraordinary measures for shore protection since its geologic setting affords natural protection. Historically, high tides from hurricanes and blizzards have been the only concern other than some flooding and wind damage.

However, the coastal areas north and south of Boston have suffered severe damage from hurricanes and blizzards. The famous blizzard of February 1978 caused considerable erosion and flooding of these shoreline areas. The estimated one-hundred-year storm flood elevations were greatly exceeded. Huge amounts of sand from Plum Island, north of Boston in Newburyport, and from beaches to the south, including Cape Cod, have been removed and re-deposited. The normal rate of erosion on many parts of Cape Cod and nearby islands is about 1 m (3 ft) per year. The erosion rate along the Boston Harbor islands is also high.

New Coastal Wetlands Regulations, General Laws Chapter 30, Section 37, which supplemented the Massachusetts Wetlands Protection Act, Chapter 131, Section 40 and the Coastal Zone Management (CZM) Program now prevent, without special permit, any construction activities on shore and beach areas. Extensive tidal marshes still exist along the Neponset River in Dorchester, a section of Boston bordering Quincy to the south, and in East Boston.

The densely urbanized area of the lower Charles River has been exposed to the threat of serious flood damage in the past. Boston and its neighboring cities such as Cambridge previously experienced intensive flooding in severe storms. The Charles River dam was completed in 1910 by the Commonwealth of Massachusetts to prevent tidal flooding along the lower reach of the river and to create a recreational pool covering unsightly and malodorous tidal flats.[2] The pool, also called the Charles River Basin, was modeled on the Alster River Basin of Hamburg, Germany, and it soon became a major recreational and aesthetic feature of Boston. With the growth of the city, the dam became unsuited to the needs of the community. Its sluice gates were no longer adequate to handle floodwater coursing into the basin and its single navigation lock could not accommodate recreational river traffic growing in volume every year.[3] A new Charles River dam was authorized by Congress in 1968 and construction began in February 1973. The dam was completed in 1978. It cost $59 million and is the single largest flood control project in New England. The project won a Presidential design award.

Hazardous Waste

Outside of the hilly areas of the original Boston Peninsula, much of the city was built on reclaimed tidal marsh. These low areas were filled with granular material from sources in and around Boston until the 19th century. With the advent of the industrial revolution, waste

in the form of ash and cinders was used as a fill. Therefore, these materials will be encountered in many excavations in the filled areas of Boston so that it might seem to be *ubiquitous*. Such fill would likely be classified as hazardous waste under our current environmental laws.

Old dumps exist in every section of Boston, some of which have been reclaimed such as the Columbia Point dump, the current site of the John F. Kennedy Library and the Boston campus of the University of Massachusetts. No old dump or landfill in Boston has yet to be classified as a Superfund site.

Boston is now the home of high-tech rather than the smokestack industries that are common elsewhere in the northeast. Only small hazardous waste generators, for the most part, now operate outside the downtown area. Hazardous waste generators are regulated by the Department of Environmental Quality Engineering (DEQE), 310 CMR 30.0, Hazardous Waste Regulations promulgated under Massachusetts General Laws 21C. Some hazardous waste disposal sites are identified on what is called the *Hazardous Waste Site List A,B,C* published quarterly in compliance with Massachusetts General Laws Chapter 21E, Section 3A(b) as amended by Chapter 554, Acts of 1986. They are identified as requiring cleanup or no action.

Under Chapter 21E, a superlien (one that supersedes all others) may be placed on property that is found to contain hazardous waste. The lien allows the state to recover the costs of state-implemented assessments of cleanups. In order to avoid this liability, properties in Massachusetts that are being sold are subjected to a 21E site assessment required by title insurance companies.

The DEQE has not promulgated any regulations describing the criteria for a 21E site assessment. If hazardous waste is found, each site is evaluated on a case-by-case basis.

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REFERENCES

1. Kaye, C.A., "The Geology and Early History of the Boston, Massachusetts Area — A Bicentennial Approach," *U.S. Geological Survey Bulletin 1476*, 1976, 78 pp.

2. Parkman, A., *Army Engineers in New England 1775-1975*, U.S. Army Corps of Engineers, New England Division, 1978, 319 pp.

3. Boston Society of Civil Engineers, "Boston's Charles River Basin: An Engineering Landmark," *Journal of the Boston Society of Civil Engineers Section/ASCE*, 1981, V. 67, N. 4.

4. Metropolitan District Commission, "Metropolitan Water System: Commonwealth of Massachusetts, Boston, Map," 1976.

Major Engineered Structures in Boston

The geologic complexity of the area provides a diverse array of solutions to foundation construction problems.

EDMUND G. JOHNSON

During the early colonial days, over 300 years ago, Bostonians built utilitarian structures that were most often constructed of wood, with simple wood or stone foundations. These buildings were located within the original Boston peninsula. Most of these neighborhoods were eventually leveled by fires and only Paul Revere's house, dating from 1677, remains today from that era.

As Boston prospered during the 1700s, more durable buildings appeared. Of those which survive today — the Old State House built in 1712, the Old South Meeting House in 1729, King's Chapel in 1749 and many edifices on Beacon Hill including Bulfinch's gold-domed State House in 1795 — all were built on solid ground and were probably supported on footings of granite that was quarried largely in Quincy, a town to the south of the Boston. However, construction also pushed toward the waterfront, where Faneuil Hall in 1740 and Quincy Markets in 1824 had to be supported on timber piles driven through the fill and underlying mud.

During the second half of the 19th century, Boston was ready for change and expansion, since it reached the outer limit of its buildable land. Industrialization and the building of railroads greatly improved the current construction capabilities and practices. After the 182 hectares (450 acres) of marsh land that was once Back Bay were filled in, the area was laid out in a grid of streets. Fashionable brick town houses lined these streets as well as several cultural institutions such as the original Museum of Fine Arts, the original Massachusetts Institute of Technology building, Horticultural Hall, Symphony Hall and the Public Library. Also present are the beautiful Trinity Church that faces the public library at Copley Square and the Christian Science Mother Church that lies further to the west. For all of these structures, the standard foundation practice was to support them on untreated timber piles that were typically 7.5 to 12 m (25 to 40 ft) long, and were driven to bearing in the sand layer below the fill and original organic layers. These piles have generally performed well except where the pile cut-offs have been exposed to drying and decay (see "Geotechnical Characteristics of the Boston Area," on pages 53-64).

In the early 20th century there was relatively little new construction. However, in the early 1960s, the city sprang to life and a building boom started that is still in progress. A major start in the Back Bay was the Prudential Center,

which included a 52-story office tower and an adjacent 29-story hotel.[1] The Scollay Square area was demolished to make way for the Government Center which now includes the modernistic City Hall, as well as several major office buildings that house federal and state agencies. This effort was essentially completed by 1968. Subsequently, as the banking and insurance community strengthened their faith in Boston, several major high-rise office buildings have been built in the downtown area between State Street and South Station. There has also been considerable construction activity along the waterfront, and the construction boom has also extended to the renovation and expansion of many older buildings.

An outstanding example of private development of a large urban site is the Christian Science Center which was completed in 1974. Currently, the tallest structure in Boston is the 60-story John Hancock Tower. The structure is totally wrapped with reflective glass and overlooks Copley Square. The link between the Prudential Center area and downtown is gradually being filled in. The Copley Place project includes two hotels (35 and 38 stories), plus offices and retail space. New projects in the Park Plaza area include the State Transportation Building (8 stories), the Four Seasons Hotel (13 stories) and the Heritage on the Garden. The dramatic changes in the Boston skyline during a 28-year period is easily seen in aerial views (see Figures 1 and 2).

Examples of Building Foundations

Many different foundation types are found within the city's limits (see "Geotechnical Characteristics of the Boston Area," on pages 53-64). This assortment of different types is a direct result of the variations in subsurface conditions.

To illustrate the reasons for this variety, the locations of many of the significant buildings constructed during the past 25 years are shown in Figures 3 and 4. The key numbers are identified in Table 1, where the typical geologic sequence found at each site is listed, together with descriptions of the foundations type and other details (see Figures 1 and 2, on pages 54 and 61, respectively, in "Geotechnical Characteristics of the Boston Area").

As the city developed over the years, severe space restrictions have added complexities to foundation design. Where adjacent properties must be protected during deep excavations, tied-back slurry walls have been used, such as those used at Sixty State Street. The John Hancock Parking Garage required the installation of deep drilled-in-caissons within a narrow median strip between depressed roadway sections of the Massachusetts Turnpike Extension. At 53 State Street, the granite facade of the existing Boston Stock Exchange Building was temporarily supported as a new glass-enclosed high-rise was built immediately behind it. Similarly, temporary facade support has been undertaken at the 101 Arch Street and 125 Summer Street construction projects.

In order to provide badly needed parking space, deeper excavations below buildings are often called for. Recent examples include International Place (5 levels), Rowes Wharf (5 levels), 125 Summer Street (5 levels) and 75 State Street (6 levels). The latter three projects utilized the "up-down" construction technique.

Recent developments in equipment and techniques make it possible to install high capacity foundation units below existing structures in low headroom conditions.[2]

In spite of periodic economic slowdowns, the future for building development in Boston is very promising since the demand continues for office as well as hotel and residential space, with each project offering new challenges to engineering geologists and geotechnical engineers.

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REFERENCES

1. Ball, D.G., "Prudential Center Foundations," *Jour-*

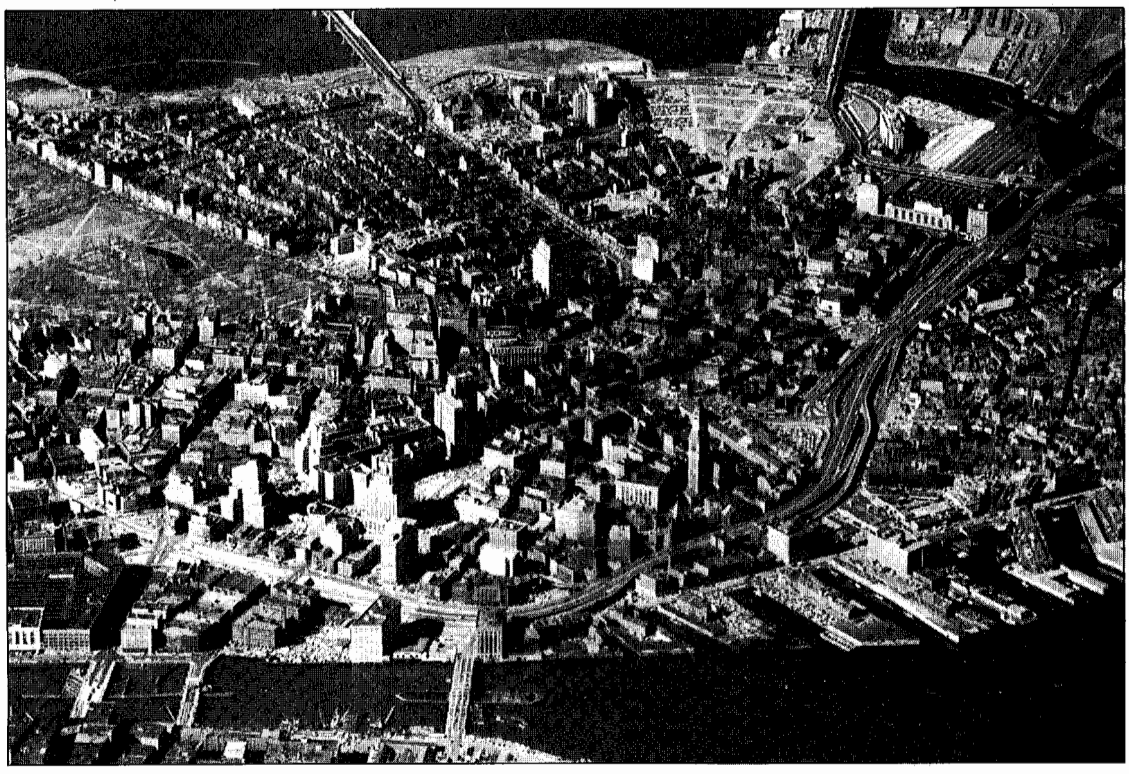


FIGURE 1. Aerial view of downtown Boston in 1959 (Aerial Photos International).

FIGURE 2. Aerial view of downtown Boston in 1987 (Aerial Photos International).

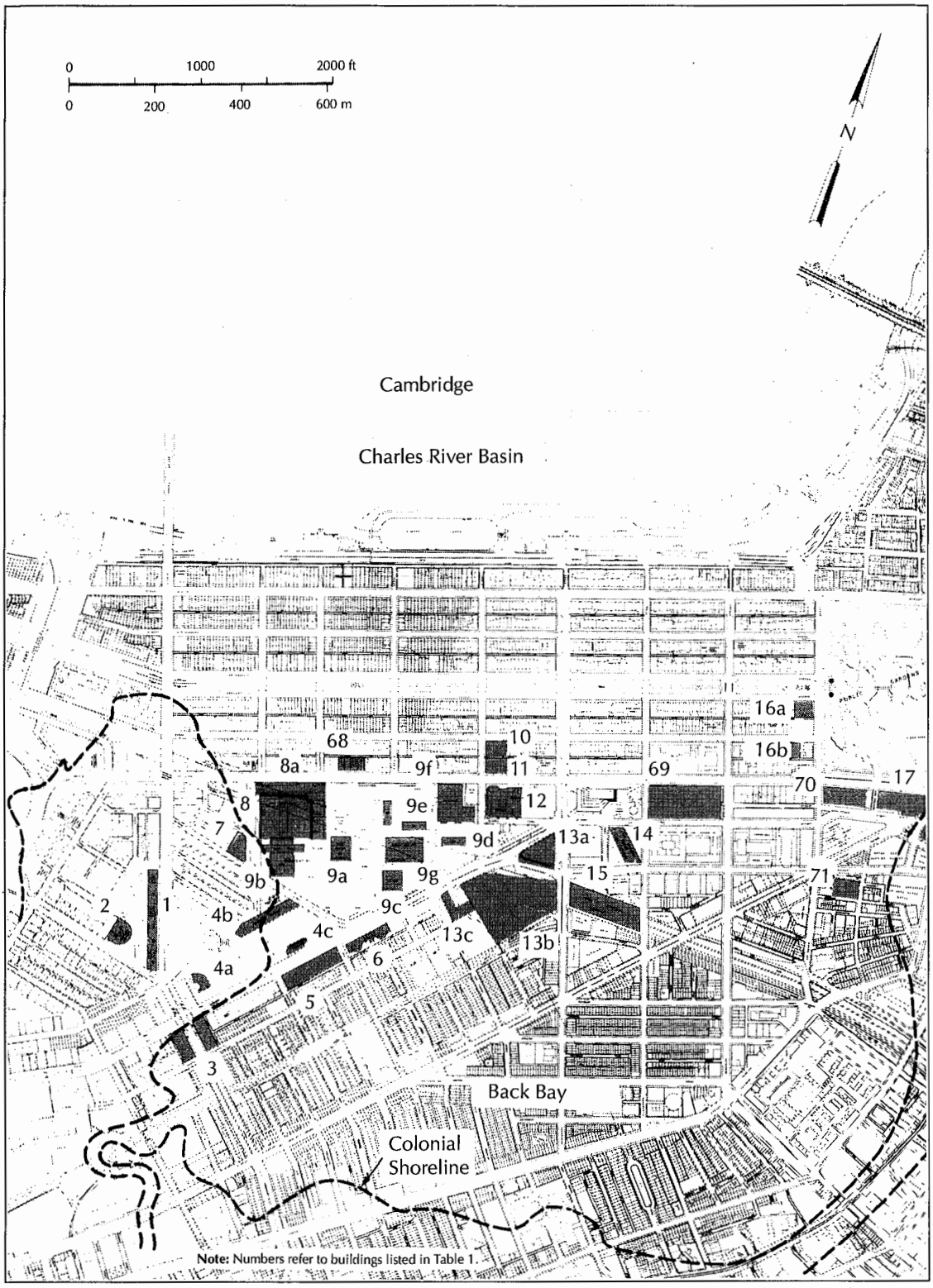


FIGURE 3. Map of the Back Bay district of Boston showing the locations of the major buildings constructed since 1960.

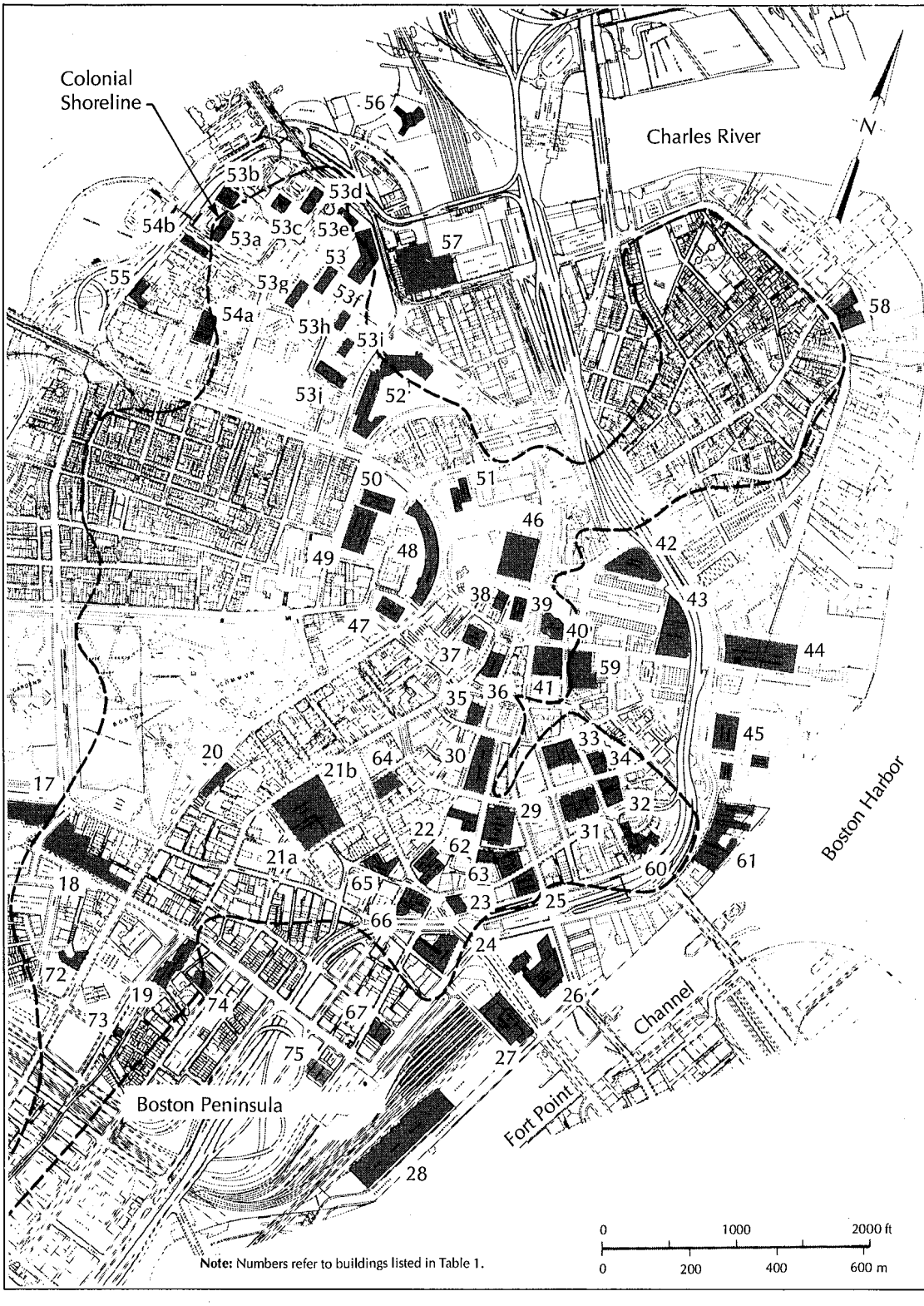


FIGURE 4. Map of the downtown area of Boston showing the locations of the major buildings constructed since 1960.

TABLE 1
Major Modern Buildings in Boston & Their Types of Foundation

Reference Number	Building/Location	Start of Construction	Typical Soil Profile	Soil Unit & Foundation Type	Description
1	Church Park Apartments	1971	A	III-2	10-story apartment building; pressure injected footings in outwash sand.
2	Church Park Garage	1971	A	III-2	6-story parking garage; pressure injected footings in outwash sand.
3	Symphony Towers Massachusetts Ave./ Huntington Ave.	1971	A	III-2	Two 12-story apartment buildings; pressure injected footings.
	Christian Science Church Center, Huntington Ave.				
4a	Sunday School Bldg.	1968	A	III-2	5-story building; pressure injected footings in sand.
4b	Colonnade Bldg.	1968	A	III-2	5-story building; pressure injected footings in sand.
4c	Administration Bldg.	1968	A	VI-3	28-story office building; concrete-filled pipe piles to a depth of 52 m (170 ft)
5	Greenhouse Apartments Huntington Ave.	1981	A	III-2	12-story apartment building; pressure injected footings in outwash sand.
6	Colonnade Hotel Huntington Ave.	1971	A	III-2	12-story hotel; pressure injected footings in outwash sand.
7	Hilton Hotel Dalton St.	1981	A	VI-3	20-story hotel & garage; prestressed concrete piles & pressure injected footings.
8	Hynes Auditorium Boylston St.	1960	A	VI-3	Two-level municipal auditorium & basement; concrete-filled pipe piles to till or rock at depth 60 m (200 ft).
8a	Hynes Convention Center	1985	A	VIII-2 (modified)	Expansion area; drilled piles into argillite (250 tons).
	Prudential Center Huntington Ave./ Boylston St.				
9a	Office Tower	1959	A	VII-2	52-story office tower; drilled-in-caissons, bearing in argillite at depth 66 m (220 ft).
9b	Sheraton Boston Hotel	1962	A	VI-3	29-story hotel; 16-in. diameter concrete-filled pipe piles to till at depth 45 to 60 m (150 to 200 ft).
9c	Southeast Tower	1962	A	VI-3	29-story office tower; 16-in. dia. concrete-filled pipe piles to till at depth of 45 to 60 m (150 to 200 ft).

continued

Reference Number	Building/Location	Start of Construction	Typical Soil Profile	Soil Unit & Foundation Type	Description
9d	Apartment No. 5	1962	A	VI-3	26-story apartment; 16-in. dia. concrete-filled pipe piles to till at depth 45 to 60 m (150 to 200 ft)
9e	Apartments No. 1 & 3	1963	A	VII-2	26-story apartments; drilled-in caissons to argillite at depth 55 m (180 ft)
9f	Lord & Taylor	1966	A	VI-3	5-story retail store; concrete-filled pipe piles to depth 42 m (140 ft)
9g	Sak's Fifth Avenue	1970	A	III-2	5-story retail store; pressure injected footings & wood piles bearing in outwash sand.
10	Exeter Towers Exeter/Newbury Sts.	1980	A	III-2	6-story apartment building; pressure injected footings bearing in outwash sand.
11	One Exeter Place Exeter/Boylston Sts.	1983	A/B	VI-3	13-story office building; prestressed concrete piles to depth 44 m (140 ft)
12	Boston Public Library Addition, Boylston St.	1972	A/B	IV-A-1	4-story library addition; mat foundation bearing on stiff clay & sand.
	Copley Place Project Copley Square				
13a	Westin Hotel	1980	A	VI-3 or VII-1	38-story hotel; prestressed concrete piles driven to till or argillite at depth 30 to 50 m (100 to 165 ft).
13b	Central Area	1981	A	VI-3 or VII-1	Up to 15-story retail, office & parking; prestressed concrete piles driven to till or argillite at depth 30 to 50 m (100 to 165 ft).
13c	Marriott Hotel	1982	A	VI-3 or VII-1	35-story hotel; prestressed concrete piles driven to till or argillite at depth 30 to 50 m (100 to 165 ft).
14	John Hancock Tower St. James/Clarendon Sts.	1969	A	VII-1	60-story office tower; steel H-piles driven to till or argillite.
15	John Hancock Parking Garage, Clarendon/ Dartmouth Sts.	1968	A	VII-2	8-story parking garage; steel H-piles & drilled-in caissons into argillite (built on air rights above Mass. Tnpk. Ext.).
16a	Ritz Carlton Hotel Addition, Arlington St.	1980	B	III-2	18-story hotel; pressure injected footings.
16b	Parking Garage Newbury St.	1980	B	IV-A-2	Prestressed concrete piles (friction) in marine deposits.
17	Four Seasons Hotel Boylston St. (Park Plaza)	1982	B	IV-A-2 (modified)	13-story hotel/condominium; 2 levels parking under; pressure injected footings, to sand layers in marine deposits.
18	State Transportation Building, Park Plaza	1981	B	IV-A-1	8-story building; concrete mat foundation at depth 10.5 m (35 ft), slurry wall around entire perimeter.

continued

Reference Number	Building/Location	Start of Construction	Typical Soil Profile	Soil Unit & Foundation Type	Description
19	Tufts New England Medical Center Washington/Stuart Sts.	1978	B	VII-1	9-story medical facility; piles driven to bedrock to accommodate high column loading
20	Tremont-on-the-Common Tremont St.	1963	B	VI-3	25-story apartment; concrete-filled shell piles to glacial till.
	Lafayette Place Washington St.				
21a	Hotel	1980	B	VI-3	20-story hotel; concrete-filled pipe piles to till, depth 17 m (55 ft).
21b	Parking & Retail	1982	B	IV-A-1	3-story retail; 3 levels parking below; spread footings on sand.
22	Office Building 100 Summer St.	1971	C	VI-A-4	33-story office tower; belled caissons bearing in till at depth 9 m (30 ft).
23	Office Building 175 Federal St.	1973	C	VI-A-4	14-story office tower; belled caissons bearing in till at depth 12 m (40 ft).
24	One Financial Center Atlantic Ave./ Summer St.	1979	C	VI-A-2	46-story office tower; 2 levels below grade; concrete mat on glacial till.
25	Keystone Building 225 Congress St.	1969	C	VI-A-2	17-story office tower; footings on glacial till.
26	Federal Reserve Bank Atlantic Ave/Congress St	1973	C	VI-A-2	35-story office tower; concrete mat on till.
27	Stone & Webster Building Summer St.	1973	B	VI-3	14-story office building; concrete-filled pipe piles to till or rock.
28	South Station Postal Annex Addition Dorchester Ave.	1967	B	VII-1	6-story building; concrete-filled steel pipe piles to bearing in argillite.
29	First National Bank 100 Federal St.	1971	C	IV-A-1	30-story office tower; footings/mat on moraine deposits.
30	Shawmut Bank 1 Federal St.	1975	C	IV-A-1	30-story office tower; footings/mat on moraine deposits.
31	State Street Bank 225 Franklin St.	1966	C	IV-A-1	30-story office tower; footings/mat on moraine deposits.
32	Office Building 265 Franklin St.	1983	C	IV-A-1	20-story office tower; footings on moraine deposits.
33	One Post Office Square Milk/Pearl Sts.	1979	B	VI-3	38-story office tower & parking structure; pressure injected footings to glacial till.
34	Office Building 260 Franklin St.	1983	C	VI-A-2	23-story office tower; spread footings on till at depth 7.5 m (25 ft).
35	Office Building 50 Milk St.	1980	B	IV-A-1	21-story office tower; footings on stiff clay.
36	The Devonshire One Devonshire Place (N. End, Washington St)	1980	C	VI-A-2	40-story apartment & office tower; combined footings on moraine deposits.

continued

Reference Number	Building/Location	Start of Construction	Typical Soil Profile	Soil Unit & Foundation Type	Description
37	Boston Company Building One Boston Place (Washington/State Sts.)	1968	B	—	41-story office tower; core on mat on gravel, heavy corner loads on belled piers in argillite at depth 24 m (80 ft).
38	Office Building One Washington Mall	1968	B	IV-A-1	12-story office building; spread footings at depth 6 m (20 ft).
39	Office Building 28 State St.	1965	B	IV-A-1	40-story office tower; concrete mat bearing at depth 10.5 m (35 ft).
40	Office Building 60 State St.	1976	C	VI-A-2	38-story office tower; 3-level basement; concrete mat on glacial till, also belled caissons below plaza area.
41	Office Building 53 State St.	1982	B/C	VI-A-2 & VI-A-3	40-story office tower; core on deep concrete mat on till, perimeter on concrete-filled pipe piles to till/rock.
42	Public Parking Garage Dock Square	1978	B	VI-3	7-level parking structure; prestressed concrete piles to glacial till.
43	Marketplace Center State St.	1983	B	VI-3	16-story office building; prestressed concrete piles to till.
44	Long Wharf Hotel Atlantic Ave.	1979	B	VI-3	9-story waterfront hotel; prestressed concrete piles to till.
45	Harbor Towers & Garage Atlantic Ave.	1969	B	VI-3	Two 40-story apartment towers & 6-story garage; concrete-filled pipe piles to depth 18 to 27 m (60 to 90 ft).
46	Boston City Hall Government Center	1964	B	VI-3	6-level municipal building; concrete-filled pipe piles to till.
47	Office Building One Beacon St.	1970	C	VI-A-2	40-story office tower; concrete mat on moraine deposits.
48	1, 2 & 3 Center Plaza Cambridge St.	1963	C	VI-A-2	8-story office building; footings on moraine deposits.
49	McCormack State Office Building One Ashburton Place	1969	C	VI-A-2	22-story office building; footings on moraine deposits.
50	Saltonstall State Office Building, Bowdoin St. (Government Center)	1963	C	VI-A-2	16-story office building; concrete-filled pipe piles into till.
51	J.F. Kennedy Office Bldg. Government Center	1966	C	VI-A-2	25-story office building; mat foundation.
52	State Services Center Staniford St. (Government Center)	1970	C	VII	8-story office building; drilled rock-socketed caissons.
	Charles River Park & Longfellow Place				
53a	510 Emerson Place	1960	B	VI-3	15-story apartments; concrete-filled pipe piles.
53b	10 Emerson Place	1960	B	VI-3	23-story apartments; concrete-filled pipe piles.
53c	8 Whittier Place	1962	B	III-2	23-story apartments; pressure injected footings.

continued

Reference Number	Building/Location	Start of Construction	Typical Soil Profile	Soil Unit & Foundation Type	Description
53d	6 Whittier Place	1962	B	III-2	15-story apartments; pressure injected footings.
53e	Amy Lowell House	1974	B	III-2	11-story housing; pressure injected footings.
53f	Parking Garage	1973	B/C	III-4 & VI-A-3	Spread footings & pipe piles.
53g	1 & 2 Hawthorne Place	1973	B/C	III-4 & VI-A-4	15-story apartments; spread footings & caissons.
53h	1 Longfellow Place	1973	C	—	36-story apartments; spread footings on rock.
53i	4 Longfellow Place	1973	C	VI-A-3	36-story apartments; concrete-filled pipe piles.
53j	50 Staniford St.	1973	C	VI-A-2	10-story apartments; spread footings on soil.
	Massachusetts General Hospital Charles St.				
54a	Ambulatory Care Center	1980	B	VI-3	8-story medical facility; concrete-filled pipe/steel shell (composite) piles to glacial till at depth 70 ft.
54b	Cox Building	1979	B	III-2	7-story medical facility; pressure injected footings in sand.
55	Massachusetts Eye & Ear Infirmary Charles St.	1969	B	VI-3	13-story facility; adjacent & above original 5-story wood pile supported building; 150-ton pressure injected footings in till at 80 ft with steel cased shafts.
56	Spaulding Rehab. Ctr. Off Nashua St.	1968/ 1980	B	VI-3	10/6-story nursing facility; concrete-filled pipe piles to till.
57	U.S. Government Services Building Causeway St.	1985	B	VI-3	10-story office building; prestressed concrete piles to till, portion on steel H-piles.
58	U.S. Coast Guard EMS Building Atlantic Ave.	1981	B	III-1	3-story maintenance facility; new structural mat over existing wood piles, plus additional prestressed concrete piles.
59	Office Building 75 State St.	1986	B	VII-2	31-story office tower, 6 levels below; belled & straight shaft caissons in argillite, slurry wall, up-down construction.
60	International Place (Ph. One) High St.	1985	C	VI-A-2	11- to 46-story office towers, 5 levels below; concrete mat & footings on till.
61	Rowes Wharf Development Atlantic Ave.	1985	B	VI-4	15-story office/hotel, 5 levels below; belled caissons in till, slurry wall, up-down construction.
62	Office Building 101 Federal St.	1986	B	VII-2 (modified)	32-story office, 2 levels below; straight shaft caissons socketed in bedrock, concrete under slurry.

continued

Reference Number	Building/Location	Start of Construction	Typical Soil Profile	Soil Unit & Foundation Type	Description
63	Office Building 150 Federal St.	1986	B	VI-A-1	28-story office, 3 levels below; spread footings on marine deposits.
64	Office Building 101 Arch St.	1986	C	VI-A-4	21-story office; belled caissons in glacial till, concreted under slurry.
65	Office Building 99 Summer St.	1986	C	VI-A-2	22-story office tower; footings on glacial till.
66	Office Building 125 Summer St.	1987	C	Special	23-story office tower, 5 levels below; load bearing elements (barrettes) in till & rock, slurry wall, up-down construction.
67	Office Building 745 Atlantic Ave.	1987	B	IV-A-1	10-story office, 3 levels below; concrete mat bearing on sand.
68	Ingalls Building 855 Boylston St.		A	III-2	12-story office; pressure injected footings in sand outwash.
69	Office Building 500 Boylston St.	1986	A	IV-A-1	25-story office towers, 3 levels below; concrete mat bearing on clay.
70	Heritage-on-the-Garden Boylston St.	1986	B	IV-A-1	13-story condominium, 2 & 3 levels below; concrete mat on marine deposits.
71	Bradford Towers West Stuart St.		B	IV-1	4-, 6- &9-story housing; caissons in stiff clay.
72	Bradford Towers East Tremont St.		B	IV-A-1	5- & 7-story housing; spread footings on clay, portion on deep caissons near subway.
73	Elderly Housing Washington St.		B	IV-A-1	Elderly housing; concrete mat on clay.
74	Tufts Health, Science & Education Building Harrison Ave.	1984	B	IV-A-1	9-story medical facility; concrete mat on clay.
75	Wang Laboratories Kneeland St.	1984	A	VII-1	10-story building; prestressed concrete piles to till or bedrock at depth 125 ft.

nal of the Boston Society of Civil Engineers, Vol. 49, 1962, pp. 229-244.

2. Johnson, E.G., & Schoenwolf, D.A., "Foundation Considerations for the Expansion and Renovation of the Hynes Auditorium," Boston Society of Civil Engineers, *Civil Engineering Practice*, Vol. 2, No. 2, 1987, pp. 35-62.

Tunneling Projects in the Boston Area

Tunneling provides an opportunity to hone the geologic knowledge of an area.

DAVID WOODHOUSE

The use of underground space in the Boston area has been very extensive, dating back to the colonial days when wooden water pipes were laid underground. This use became more intensive in the 19th century when pipes for illuminating gas were laid and the first subway in the United States was constructed. Since that time, major underground projects have included the construction of rock tunnels to distribute water from Quabbin Reservoir, sewer interceptors to Deer Island, subaqueous transportation tunnels and subway tunnels (see Table 1).

Tunneling under Boston has been so extensive that only a summary is presented here. Numerous other authors present more detailed studies.[1-18] In addition, many detailed, but unpublished, engineering geological studies have been conducted by the major geotechnical firms in the area.

Subway Tunnels

More than a century has passed since state and local governments in Massachusetts first recognized the need for increasing the speed and capacity of public transportation in the Boston area. The many rapid transit facilities built since that time formed the foundations of today's Red, Orange, Blue and Green rapid transit lines of the Massachusetts Bay Transportation Authority (MBTA) (see Figure 1). This network, now 125 km (77.4 mi) in length, carries an estimated 128 million riders annually. For a city the size of Boston, its rapid transit system is disproportionately large when compared to rapid transit facilities elsewhere in the United States. This reliance on mass transportation is no accident, for Boston has long been a leader in rapid transit development, starting with the first subway in America that opened in Boston in 1897 and remains in daily use.

History. Chapter 478, Acts of 1893, created a three-man Board of Subway Commissioners, subject to the approval of the Boston City Council and appointed by the Mayor.[19] The board was charged by the state legislature to report on the feasibility of building a subway under Boston Common through the deformed till, clay and outwash in order to replace the streetcar traffic on Tremont Street. Work progressed so rapidly that the first section from Park Street to a portal in the Public Garden, near Arlington and Boylston Streets was opened on September 1, 1897. Two additional segments of subway were opened later. The leg under Tremont Street from Boylston Street Station to Pleasant Street (now Broadway) opened

TABLE 1
Major Tunnels in the Boston Metropolitan Area

Name of Tunnel	Purpose	Date Constructed	Length (km)	Lined Diameter (m)	Approx. Depth (m)	Method of Construction	Predominant Lithology
Dorchester Bay Tunnel	Sewer	1880s	3.2	1.0	10.0 - 20.0	Cut-and-cover	Clay/Till
City Tunnel	Water	1947-51	7.7	3.7	70.0 - 137.0	Drill and blast	Argillite, Tillite Sandstone, Diabase, Conglomerates
City Tunnel Extension	Water	1951-56	11.4	3.1	70.0 - 122.0	Drill and blast	Conglomerate Sandstone, Argillite
North Metropolitan Relief Tunnel	Sewer	1950-56	6.3	3.1	85.5	Drill and blast	Argillite
Main Drainage Tunnel	Sewer	1954-59	11.5	3.1 - 3.5	89.0 - 92.5	Drill and blast	Conglomerate, Argillite, Tillites, Sandstone, Shale
Malden Tunnel	Water	1957-58	1.6	3.8	85.5 - 100.0	Drill and blast	Felsite, Argillite, Diabase
Dorchester Tunnel	Water	1968-74	10.2	3.1	30.5 - 61.0	T-BM, Drill and blast	Argillite, Sandstone, Conglomerates, Basalts
Boston Main Drainage Relief Sewer	Sewer	1950	3.8	3.1	70.0 - 80.0	Shield	Clay
Stony Brook Conduit	Sewer	1880s	3.4	5.2 × 4.7	10.0 - 15.0	Open cut	Sand/Gravel Till
MBTA Red Line Extension Northwest Mined Tunnels	Transit	1976-80	2.2	5.8	15.0 - 43.0	Shield (till) Lull fleet	Till, Argillite Sandstone, Diabase
Blue Line Harbor Crossing	Transit	1900-03	2.6	6.2 × 7.0	19.2 - 25.0	Shield compressed air	Clay
Red Line Fort Point Channel	Transit	1915-16	1.0	6.1	18.3 - 23.0	Shield, compressed	Clay, Sand, Gravel
Red Line Subway	Transit	1912	4.0	5.8	7.2	Cut-and-cover	Clay, Sand, Gravel
Orange Line Subway	Transit	1904-06	4.5	Horseshoe/Box	3.9	Cut-and-cover	Clay, Sand, Gravel
Green Line Subway	Transit	1895-1912	5.7	Horseshoe/Box	6.0 - 10.0	Cut cover shield	Clay/Till Fill
Sumner Tunnel	Vehicular	1920s	2.3	Circular	10.0 - 40.0	Shield & compressed	Clay
Callahan Tunnel	Vehicular	1960s	2.3	Circular	10.0 - 40.0	Shield compressed	Clay

on October 1, 1897. On September 3, 1898, operation commenced on the final section from the filled tidal flat area of North Station to Park Street on the edge of Beacon Hill.[20]

Chapter 500, Acts of 1897 amended the original Boston Elevated Railway Act of 1894 that provided for a new transit tunnel under Boston Harbor to East Boston and the Boston segment of a subway to Cambridge. Construction began in 1900 on the first underwater transit tunnel in America. The tunnel was constructed in mainly Pleistocene sediments varying from marine clays under the harbor to the deformed clay and sand around Beacon Hill.[3] The East Boston Tunnel opened for business on December 30, 1904. The tunnel extended from Maverick Square, East Boston, to Court Street Station, near Scollay Square in downtown Boston (now Government Center).

Plans were made to build a subway under Washington Street for the Sullivan-Dudley line. The subway followed the old Shawmut Neck, the original tidal flat area that connected Boston to Roxbury. The tidal flats have long since been filled in. Marine clays underlie the estuarine organic silt and peat that formed the flats. This structure, the Washington Street Tunnel, was opened for public use on November 30, 1908. Removal of high-platform and third-rail equipment from the Tremont Street Subway began at once, and on December 4, 1908, streetcar service fully resumed through all parts of the subway formerly used by the elevated trains.

The need for rapid transit service in the congested Back Bay area led to construction of the Boylston Street Subway through a complex area of clay and fill (see Figure 2).[7] Work began in March 1912 on the two-track tube, which would eventually run from a portal in Governor Square (now Kenmore Square) to a connection with the Tremont Street Subway near the Public Garden Incline. As part of the subway construction, the incline itself would be shifted to the middle of Boylston Street, parallel to its old location. The Boylston Street Subway

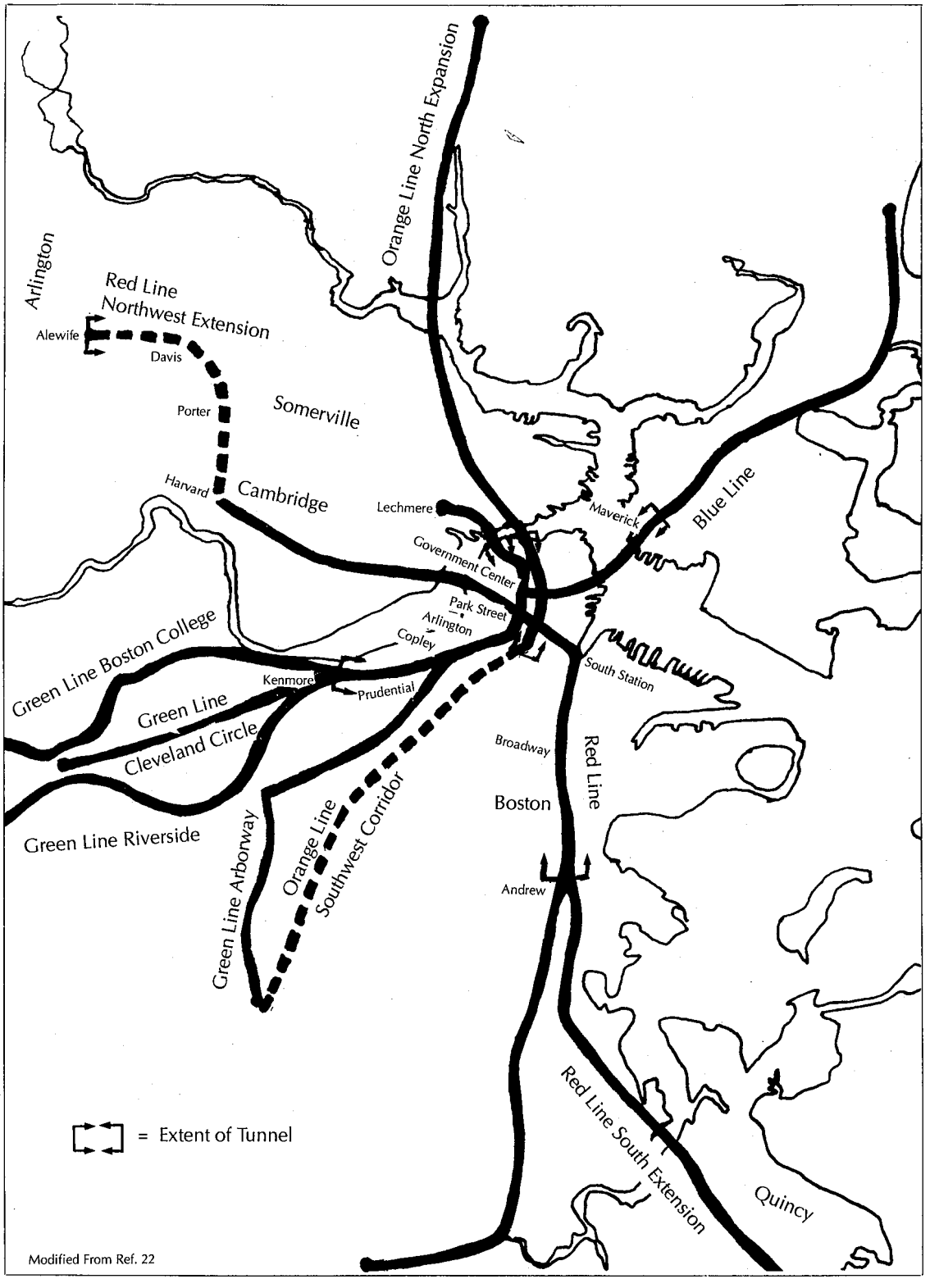


FIGURE 1. Map of the Boston area showing routes of the MBTA's rapid transit system and the extent of tunnels.

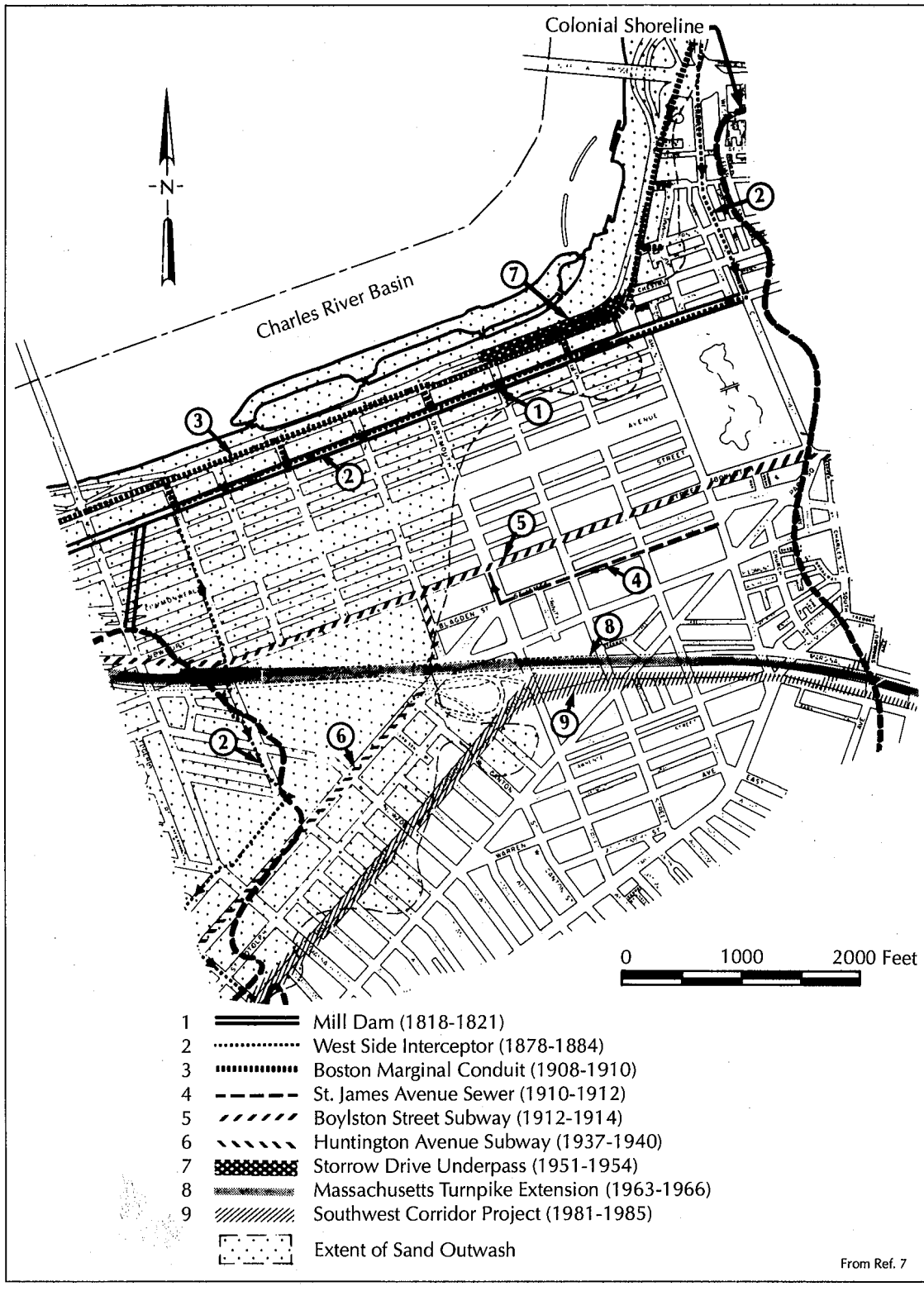


FIGURE 2. Map of the Back Bay area showing the locations of sewers, drains and major transportation routes.

TABLE 2
Geologic Material Along the Boylston Street Subway

Location	Approximate Station	El. Top of Rail	Soil Conditions at Bottom of Subway
Kenmore Street (at Commonwealth)	0+00	16.2	Sand & gravel fill underlain by silt
Charlesgate West (at Commonwealth)	6+00	-8.7	Silt underlain by sand & gravel
Charlesgate East (at Newbury St.)	10+00	-18.9	Sand & gravel; short section of clay
Massachusetts Ave. (at Newbury St.)	19+32	7.5	Sand & gravel fill underlain by silt
Hereford Street	27+15	7.7	Silt over sand & gravel
Gloucester Street	31+55	1.9	Sand & gravel
Fairfield Street	37+15	-4.8	Sand & gravel
Exeter Street	43+75	-6.4	Silt over sand & gravel
Dartmouth Street	49+80	-6.5	Silt over thin peat over thin sand & gravel
Clarendon Street	56+05	-8.8	Silt over thin peat
Berkeley Street	62+25	-13.0	Sand & gravel
Arlington Street	69+10	-14.5	Clay
Hadassah Way	73+35	N -14.0 S -11.5	Fairly hard blue clay (peat between Hadassah Way & Charles Street)
Charles Street	78+00	N -4.2 S -5.0	Blue clay & gravel

Notes: Information was obtained from Boston Transit Commission Plans No. 10219, 10386, 10091, 10418, 11157, 11159, 11161 and 11162 of "Boylston Street Subway." The bottom of subway structure varies from 4 to 5.5 ft. below top of rail. The subway is supported on wood piles from Station 71+82 to Station 76+41.

From Ref. 7

opened on October 3, 1914.

Another extension of an existing subway took place between 1912 and 1916. The East Boston Tunnel, when it originally opened, ran downtown to Court Street Station, a stub-end single-track terminal. Traffic volume through the tunnel increased tremendously over the years, and this single-track turnback became an operational nuisance. The answer was a 2,610-foot extension to Bowdoin Square through the deformed Pleistocene material of Beacon Hill,[21] where an underground station, a loop and a surface incline to Cambridge Street were provided. The loop furnished a quick turnaround for streetcars from East Boston, while the incline permitted cars to travel all the way through from Cambridge to East Boston. The East Boston Tunnel Extension was opened on March 8, 1916.

The Boston Elevated Railway and the Boston Transit Commission shared joint construction responsibility for the Cambridge-Dorchester Tunnel, today part of the MBTA Red Line. Construction began in 1909. The first phase of construction was in an area consisting primarily of outwash sand and clay that extended from Harvard Square, Cambridge, to Park Street in Boston. This line began operation on March 23, 1912. The last tunnel segment, constructed in clay, ran from Broadway Station to Andrew Square, in South Boston and was opened on June 29, 1918.

TABLE 3
Geologic Material Along the Huntington Avenue Subway

Location	Approximate Station	El. Top of Rail	Soil Conditions at Bottom of Subway
Massachusetts Ave.	13+85	-6.0	12 ft. hard packed coarse sand
Cumberland Street	21+50	-10.9	11 ft. hard packed coarse sand & gravel
West Newton Street	26+65	-13.0	7 ft. hard packed sand & gravel
Garrison Street	32+00	-13.1	4 ft. hard packed coarse sand
B&A Railroad Tracks (Mass. Turnpike Extension)	37+50	-13.6	Hard yellow clay (sand pinches out at Station 37+50±)
Blagden Street (& Exeter Street)	41+30	-10.7	4 ft. silt over medium blue clay & sand
Boylston Street (& Exeter Street)	44+50	-6.9	4 ft. peat over 8 ft. fine sand over stiff blue clay

Notes: Information was obtained from City of Boston Transit Department Plans No. 17947, 17943, 17936, 17933 and 17914 of "Huntington Ave. Subway, Plan & Profile." The bottom of the subway structure varies from 3.5 to 6 ft. below top of rail. Footings, pedestrian passageway (Mass. Ave.) and bottoms of catch basins are deeper.

From Ref. 7

In the late 1930s, the Boston Transit Department began the Huntington Avenue Subway through a variety of material (see Figure 2).[7] This subway was a Work Projects Administration (WPA) program and was one of the first examples of major federal funding for local mass transit construction. Prior to the subway, the Huntington Avenue car line ran past Northeastern University on Huntington Avenue to Copley Square and then over Boylston Street as far as the block between Arlington and Charles Streets. Here, the streetcars entered the portal to the Boylston Street Subway. The Huntington Avenue line was the last major surface streetcar route to run through this heavily congested section of the Back Bay, and its diversion to the new subway on February 16, 1941 shortened the running time considerably.

Engineering Geology. Most of the old subways were constructed in a variety of soft ground conditions that vary from marine clay to lodgement till as they passed under the Shawmut Peninsula and the filled areas of the Back Bay and the "Neck." The subaqueous East Boston Tunnel under Boston Harbor was constructed using a shield under pressure. The new Red Line Extension was tunneled through both bedrock and soft ground.

The Boylston Street subway crosses Back Bay from the portal at Kenmore Square near Massachusetts Avenue to Charles Street (see Figures 1 and 2, and Table 2). The bottom of the subway varies from approximately el. +0.9 m (+3.0 ft) at Massachusetts Avenue, to its lowest

TABLE 2
Elevations Along the Southwest Corridor Project

Location	Elevation of Bottom of Structure Subway	Elevation of Bottom of Structure Amtrak	Elevation of Bottom of Deepest Excavation
Gainsborough Street	6.0	3.3	3.6
Massachusetts Avenue	6.0	-6.0	-7.5
Blackwood Street	-8.2	-14.4	-23.0*
West Newton Street	-10.4	-16.7	-24.5*
Harcourt Street	-10.4	-16.4	-24.5*
Yarmouth Street	-4.7	-12.4	-14.4 -18.4**
Dartmouth Street	3.6	-4.0	-5.6 -12.4**
Clarendon Street	3.6	-5.0	-6.4 -9.6**
Berkeley Street	1.6	-5.0	-6.4 -8.4**
Chandler Street (to east end)	-2.1	-3.0	-4.4

Notes: Information was obtained from 1981 and 1982 Massachusetts Bay Transportation Authority construction plans for Contracts No. 097-115 and 097-120.
*Removed organic silt to top of clay stratum.
**Lower elevation for trench excavated for track drain pipe.

From Ref. 7

point of el. -5 m (-19.0 ft) between Arlington Street and Hadassah Way thence to el. -3 m (-10.0 ft) at Charles Street.[7] The soft ground conditions and elevations change along the route of the subway (see Table 2).

The structure was supported on a wide variety of soils including the fill, organic silt, and natural sand and gravel outwash. Where peat was encountered, approximately between Hadassah Way and Charles Street (a distance of 140 m [460 ft]), wood piles were driven to support the structure.[7]

During construction, a temporary drawdown of water levels both in the fill and in the sand and gravel stratum would have occurred.[7] The drawdown in the sand stratum is estimated to have reached el. -3 m (- 10 ft) where the subway route passed opposite to what is

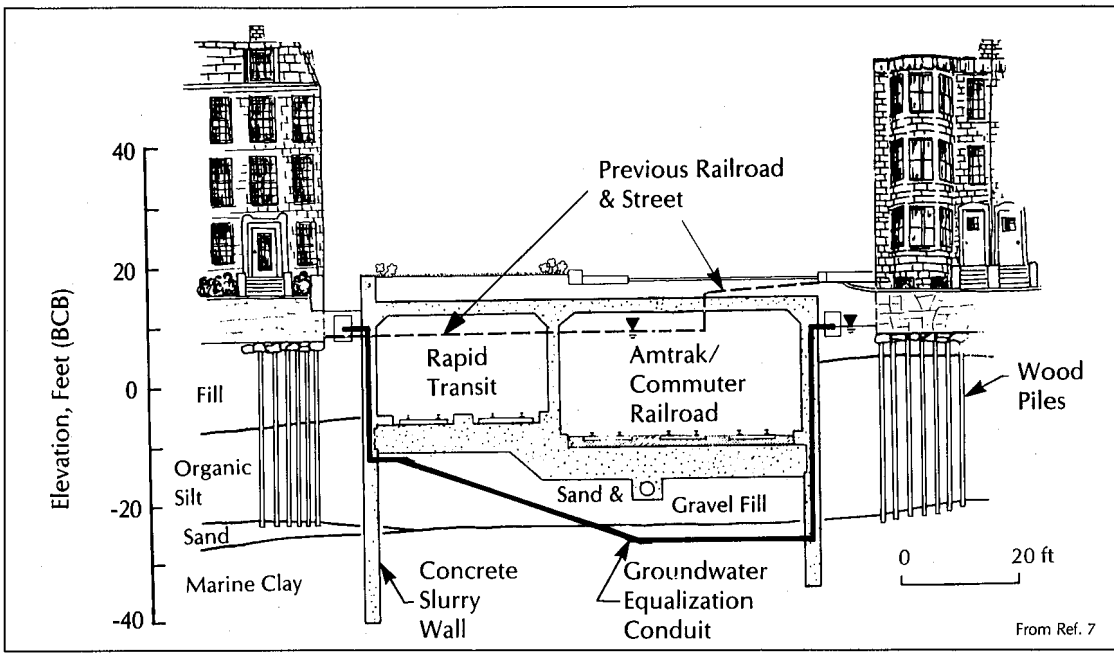


FIGURE 3. Cross-section of the Southwest Corridor tunnel at Follen Street.

now the Prudential Center.

The Huntington Avenue subway, constructed between 1937 and 1940, crosses under Massachusetts Avenue as it enters Back Bay and joins the Boylston Street subway at Exeter Street (see Figure 2 and Table 3). Within this area, the bottom grade of the subway structure varies from -3 m (el. -10 ft) at Massachusetts Avenue down to el. -5.8 m (-19 ft) where the structure passes below the railroad tracks (and under the Massachusetts Turnpike Extension).[7]

Aldrich and Lambrechts noted that:

"The subway was founded on the outwash stratum that extends from 1.5 to 3.7 m (5 to 12 ft) below the bottom of concrete from Massachusetts Avenue to the Turnpike. North of the Turnpike to Boylston Street, the structure bears on clay and organic soils, without piling. During construction, the outwash stratum was dewatered for the entire length of the subway along Huntington Avenue to grades as low as, or even below, el. -6 m (-20 ft). A very significant drawdown of water level occurred over a wide area, for a period of 2 to 3 years. The level in an observation well at Massachusetts and Commonwealth Avenues, 0.6 km (0.4 mi) away, was reported to have dropped from el. 2.1 m (7 ft) to el. 0 in 1939."[7]

Construction for the Huntington Avenue Subway required extensive and prolonged dewatering to levels below any known construction before or since.[7] In addition, drains installed in the tunnels of both subway lines have undoubtedly collected groundwater that leaked into the structure.

Southwest Corridor. A major expansion of the MBTA took place with the construction of the Red Line Extension and the Southwest Corridor which was constructed between 1981 and 1985. It has two tracks for the relocated MBTA Orange Line subway and three tracks for commuter rail and Amtrak service. Through Back Bay the alignment followed parts of two original railroad embankments that were constructed across the tidal basin in the mid-1830s (see Figure 2). From Massachusetts Avenue to Dartmouth Street, the new concrete structure was below ground in a 915 m (3,000 ft) long cut-and-cover tunnel that required excavations as deep as 11.6 m (38 ft). East of Dartmouth Street, the structure extended about 3 m (10 ft) below former grade. Depths of excavations and other data are summarized in Table 4.

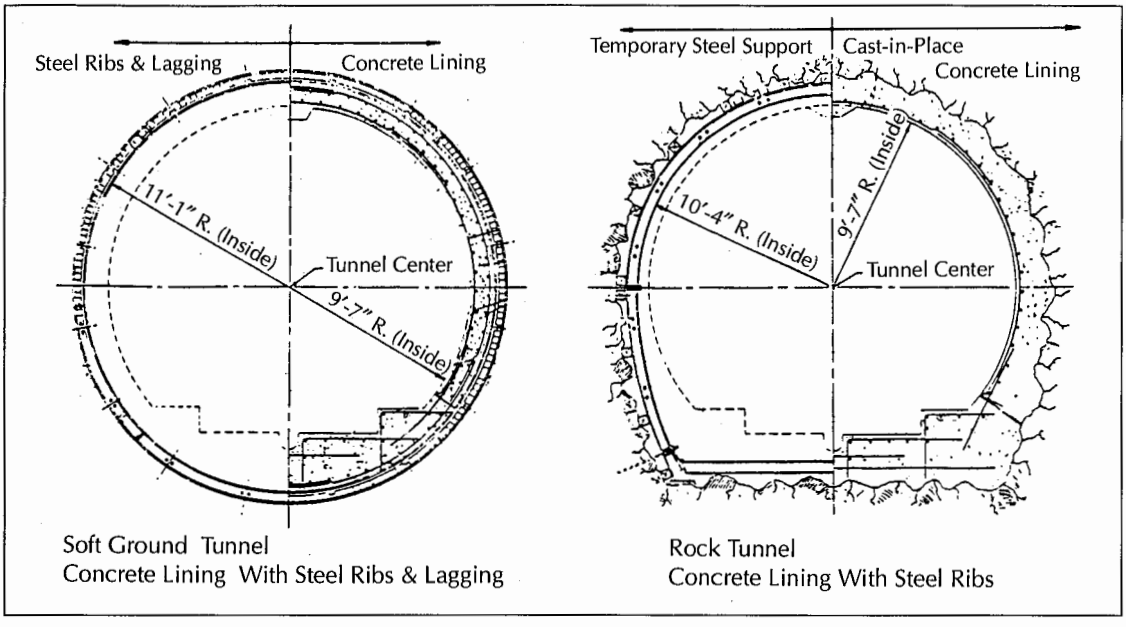


FIGURE 4. Typical tunnel sections.

Aldrich and Lambrechts summarized the engineering geology as follows (see also Figure 3):

"Reinforced concrete slurry walls were used for lateral support of the sides for about 640 m (2,100 ft) of the tunnel excavation. The concrete walls were 1 m (3 ft) thick and penetrated 2.4 to 4.6 m (8 to 15 ft) into the clay stratum. They were used as the tunnel's permanent outside walls. Although water leakage did occur through some of the vertical joints between wall panels, there was no appreciable lowering of groundwater levels in adjacent areas.

"In other deep excavation areas where

TABLE 5
Engineering Properties of the Cambridge Argillite

	Unit Weight (kg/m³; pcf)***				Unconfined Compression f_c (N/m²; psi)***				Tangent Modulus E_{t50} (N/m²; psi)***			
Location & Rock Type	Low	Average	High	No. Tests	Low	Average	High	No. Tests	Low	Average	High	No. Tests
Dorchester Tunnel Argillite	2691.0	2747.0	2810.0	15	34,480	103,430	236,840	15	39,990	62,740	84,120	15
	168.0	171.5	175.4		5,000	15,001	34,350		5,800	9,100	12,200	
MBTA Red Line Extension Argillite	2538.0	2747.0	2844.0	52	41,990	131,420	255,120	50	20,690	47,580	63,430	50
	158.4	171.5	177.5		6,090	19,060	37,000		3,000	6,900	9,200	
MBTA Red Line Extension Melaphyric Dike Rocks*	2606.0	2738.0	2861.0	12	12,620	67,570	166,860	10	14,480	24,130	31,030	10
	162.7	170.9	178.6		1,830	9,800	24,200		2,100	3,500	4,500	
MBTA Red Line Extension Tuff/Trachyte**	2518.0	2739.0	2884.0	6	29,650	103,620	250,980	6	29,650	59,300	74,470	6
	157.2	171.0	180.0		4,300	15,030	36,400		4,300	8,600	10,800	

Notes: * Rock types include diabase, andesite, basalt & altered varieties of these rocks.
** Rock previously identified as dark grey to black tuff; believed to be black trachyte appearing as irregular sill-like intrusion in Porter Square exploration shaft.
*** Metric units are shown above English units on the table grid.

From Ref. 4

TABLE 6
Engineering Properties of Materials in Boston

Stratum	Consolodation Condition	Effective Friction Angle	Total Unit Weight (pcf)	Allowable Bearing Pressure (tsf)
Fill				
Sands distributed along entire project	Loose to medium dense	28-33	110-120	1.0
Sands & Gravels				
Glacial outwash deposits; sands, gravelly sands, silty sands	Medium dense to very dense	32-36	110-125	1.0-2.5
Marine Clay				
Silty Clay	Over-consolidated	24	110-120	2.0-4.0
Till				
Glacially deposited mixture of sand, gravel, cobbles, boulders, silt, clay	Dense to very dense	36	125-140	3.0-5.0
Cambridge Argillite				
Bedrock (slightly indurated)	Medium hard to hard with locally weathered & broken layers	45	165-170	10.0-20.0

From Ref. 16

adjacent structures were further away from the excavation or absent, steel sheet-piling was used for temporary lateral support of the excavation. East of Dartmouth Street, excavations were shallower and soldier piles with wood lagging were used. Water seepage into these excavations temporarily lowered groundwater levels in adjacent areas as much as 3.7 m (12 ft).

"Where concrete slurry walls were used, the tunnel was supported on a thick concrete invert slab bearing on compacted sand and gravel fill that was used to replace unsuitable organic soils. East of this portion of the tunnel, the structure was supported on precast-prestressed concrete piles driven through the clay to end bearing on glacial till or bedrock.

"In order to allow groundwater movement across the corridor structure, a groundwater equalization underdrain system was installed. This system consisted of longitudinal drains placed 0.6 to 1.2 m (2 to 4 ft) below the pre-construction groundwater level on either side of the structure. Where slurry walls formed the tunnel walls, 20 cm (8 inch) diameter header pipes surrounded by crushed stone were connected to 20 cm (8 inch) galvanized steel pipes cast into the walls and connected beneath the invert slab. In other areas, rectangular drains of crushed stone wrapped in filter fabric were constructed beneath the invert slab and up the outside of each wall in order to allow water to flow between longitudinal drains on either side."[7]

Red Line Extension. The 5.0 km (3.1 mi) Northwest Extension of the MBTA Red Line beyond Harvard (see Figure 1) consists of two deep tunnel sections and a cut-and-cover section. The first section, which is 1,342 m (4,400 ft) long, connects the new cut-and-cover Harvard Square Station to the deep rock Porter Square Station. Porter Square Station is, in turn, linked to the cut-and-cover Davis Square Station by the second deep tunnel section, which is 884 m (2,900 ft) long.[16] Beyond the Davis

Square Station, the Northwest Extension continues as cut-and-cover tunnel along a railroad right-of-way to the Alewife Brook Station.

The deep tunnels are twin-bore excavated and 6.7 m (22 ft) in diameter, with the construction access shafts now serving as ventilation and emergency egress shafts at intervals along the alignment. A variety of excavation and support techniques were utilized during the construction of the shafts and tunnels.

Based on the expected subsurface conditions, the minimum support requirements specified for lengths of the tunnels are as follows (see Figure 4):

- Length of tunnel with circular steel ribs and wood lagging for soft ground and mixed-face conditions was 1,479 m (4,850 ft).
- Length of rock tunnels with steel sets specified was 1,085 m (3,560 ft).
- Length of rock tunnels with rock bolt support or support at contractor's option was 1,866 m (6,120 ft).

Engineering properties of the soil and bedrock are summarized in Tables 5 and 6.[3,16]

The primary rock type along the tunnel alignment is the Cambridge Argillite with lesser amounts of intrusive rocks that penetrate the country rock (see Table 5). The argillite, though non-uniform in appearance, does not show any significant variation from a geotechnical standpoint, except where it has faulted or sheared (especially where groundwater has been introduced).

Bedding characteristics in the argillite along the Red Line Extension show extreme variability,[16] although the bedding generally dips gently to moderately (20° to 45°) to the south, representing the northern limb of the Charles River Syncline. The bedrock varies from a massive dark-to medium-gray, fine-grained type to an argillite that exhibits rhythmic bands of light gray layers alternating with medium-to-dark-gray layers. The coarser grained and generally lighter colored layers have well developed bedding structures. These structures include graded sequences, cross bedding and ripple marks. Bedding plane joints are usually spaced 30 to 50 cm (12 to 20 in) apart and their presence or absence does not normally affect the stability of the tunnel opening. The exception exists when nearby faulting, or proximity to the top of the rock, results in the extreme jointing developed parallel to the bedding surfaces. The joint spacing in these cases is usually 10 to 25 cm (4 to 10 in).

The orientation of the alignment is obliquely down- or up-dip in the tangent sections, depending on the direction the heading is being driven. On the curved section from Porter Square Station to Davis Square Station, the alignment swings around almost parallel to the strike to the beds.[16] This gradual change results in a varying response by the bedrock to the tunneling activity that translates into a differently shaped tunnel opening.[4]

The major joint set along the alignment, other than bedding plane joints, generally strike north-northeast, with nearly vertical dips. Jointing parallel to bedding is commonly developed and strikes nearly east-west with gentle to moderate dips, either north or south. A strong set of joints is developed nearly parallel to shear zones, which are usually oriented more east-north-east. These joints tend to die out with increasing distance from the shear zone and have not contributed to major support problems, because most have been oriented nearly normal to the tunnel alignment. Locally, overbreak is experienced parallel to the alignment.[16]

The occurrence of mafic to felsic igneous intrusive bodies along the alignment is not uncommon and the characteristics of each occurrence are unique. They vary in composition, texture, orientation and contact relationships with the country rock. Shear or fault zones without associated clay gouge have not had much effect on the construction aspect of the tunnel other than the variable fracturing effect on the rock. Where gouge is present, however, the stability is affected because of the reduction of the frictional forces that bind blocks of rock together around the opening. The integrity of the tunnel opening is further affected when these zones are nearly parallel to the alignment.[16]

Water Supply & Drainage Tunnels

Bedrock Geology. Water supply and drainage

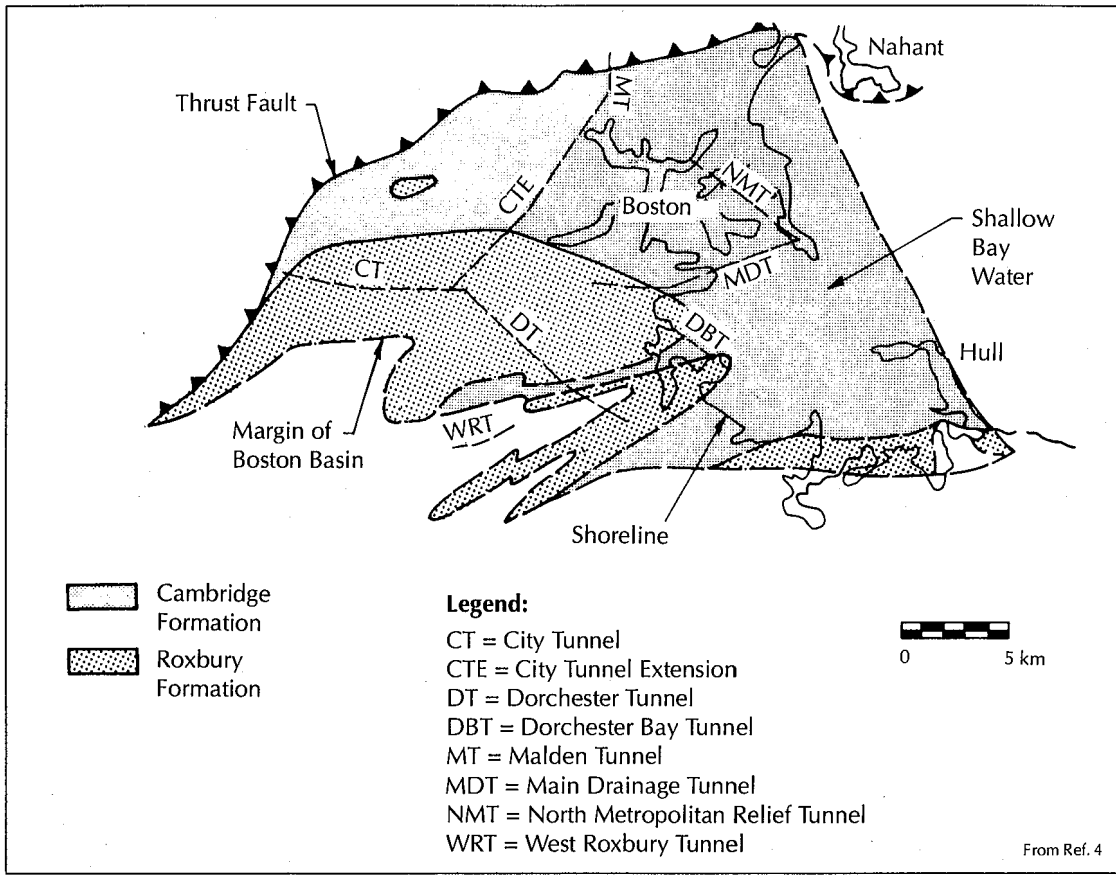


FIGURE 5. Generalized geologic map of the Boston Basin showing water supply tunnels.

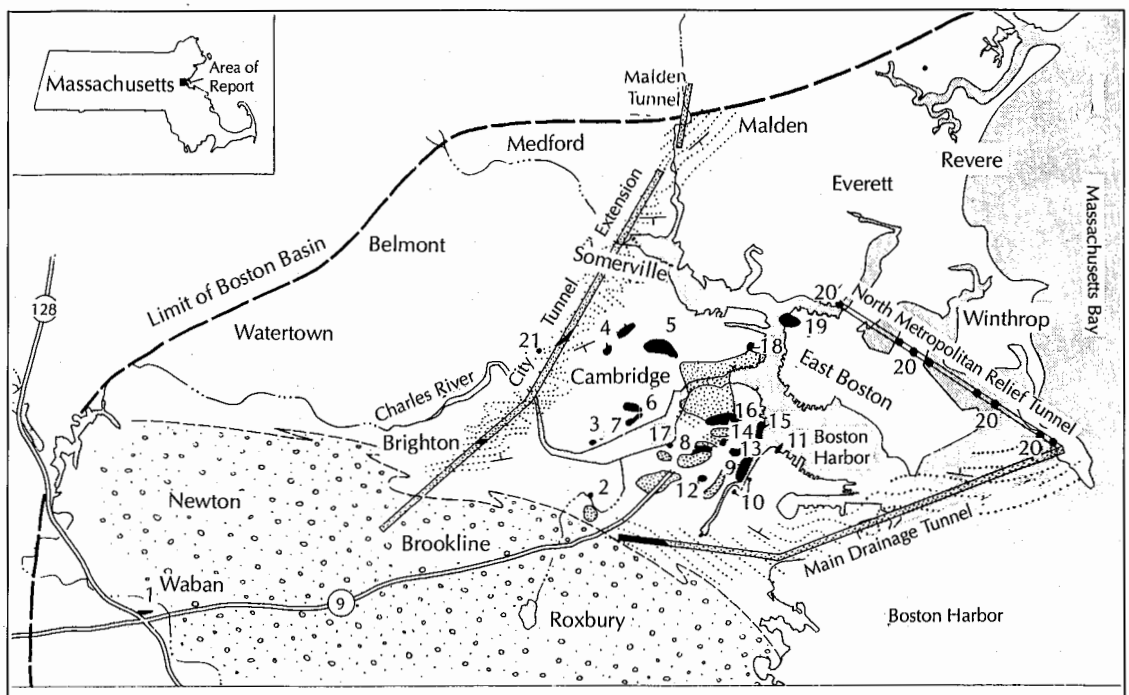
tunnels have been constructed in the region at various times (see Table 1). These tunnels were constructed largely in sedimentary rock (chiefly conglomerate and argillite, but with lesser amounts of sandstone, arkose, shale, melaphyre and diabase). Altered rock was also found in the tunnels.

The tendency of altered rocks to be restricted to certain beds is clearly expressed in the reports by Rahm, and Billings and Tierney, on the tunnels under Boston constructed by the Metropolitan District Commission (see Figures 5 and 6, and the first seven tunnels listed in Table 1).[12,11] In both tunnels studied — the City Tunnel Extension and the Main Drainage Tunnel — soft rock was found, but in both it was limited to certain beds or groups of beds. However, because tunnel observations are limited by the height of the tunnel (about 4 m [13 ft] for the Boston tunnels), the possibility exists that if viewed on a larger scale, the alteration would be seen to cut across planes of stratification.[1]

In the western 1.31 km (4,300 ft) of the Main Drainage Tunnel (see Figures 5 and 6), altered argillite (called "shale" in the tunnel reports) and sandstone are interbedded with massive conglomerate and arkose, some of which appears from the description to be altered. Three diabase dikes and sills are shown on Rahm's map as cutting the soft rocks.[12]

Billings and Tierney found "shale" in two places in the City Tunnel Extension (see Figures 5 and 6).[11] A section about 13 m (40 ft) thick of soft kaolinized argillite, interbedded with thin quartzite beds, purple argillite, sandstone and conglomerate, was found in the tunnel south of the Charles River at a depth of about 69 m (225 ft) below the top of the bedrock surface.

The North Metropolitan Relief Tunnel was completed before a geologist had an opportunity to map it. The geology of this tunnel is based on a study of the cores of exploratory



Legend

Boston Bay Group — Devonian or Carboniferous

Argillite & sandstone; minor conglomerate & volcanic rocks — dotted lines are strike trends based on tunnel data.

Conglomerate, minor argillite & volcanic rocks

Approximate contact between dominantly conglomeratic rocks & fine-grained rocks.

Strike & direction of dip of beds.

5 Known kaolinization area (see key for name of area)

Known unaltered rock (shown only for downtown Boston, Charlestown & tunnels)

Note: Occurrences of kaolinization are in these areas as follows:

1. Intersection of Routes 128 & 9
2. Fenway, in front of Sears Roebuck store
3. Memorial Drive
4. Inner Belt route
5. Boston & Maine freight yards
6. Broad Canal
7. Green Building, MIT
8. Boston Common
9. South Station Postal Annex
10. Gillette Safety Razor plant
11. Pier 2
12. Castle Square
13. Former site of Stone & Webster Building, & the New England Telephone & Telegraph Building
14. West side of Washington Street at intersection of Water Street
15. Atlantic Avenue
16. Government Center
17. Beacon & Clarendon Streets
18. Charlestown Navy Yard
19. East Boston
20. Borings for North Metropolitan Relief Tunnel
21. Memorial Hall, Harvard University

From Ref. 1

FIGURE 6. Map of the Boston area showing the areas of known kaolinization, the bedrock tunnels and the generalized geology of the Boston Bay Group.

borings and on a very rapid survey of the finished and partly-lined bore by Billings.[1] Eight borings were made in badly altered rock, but altered rock was not noticed by Billings in the tunnel, and in all likelihood it was covered by the concrete lining. There are about 40 separate lined sections, and most if not all of these sections may have been built to support altered rock: 24 percent of the tunnel was lined. Unlike the other tunnels, there was an abundance of altered rock in this tunnel and this material was distributed fairly evenly throughout the length of the bore. The length of the lined sections suggests that altered strata ranged from as little as 3 to 30 m (10 to 100 ft) or more in thickness. Between these lined sections are hard argillite and sandstone. Similar rock alterations did not occur in the contiguous western end of the Main Drainage Tunnel.[12]

The deepest recorded occurrence of alteration beneath the surface of bedrock was reported by Billings and Tierney from the City Tunnel Extension under Cambridge.[11] At this point, their profile shows about 91 m (300 ft) of rock overburden.[11] The altered rock obviously extends below the tunnel level to greater depth. No borings yield unequivocal evidence of having reached the base, or maximum depth, of a particular kaolinized zone.

There is one indirect line of evidence that alteration dies out at relatively moderate depths. The distribution of altered rock is much more restricted in the Main Drainage Tunnel and the City Tunnel Extension than it is under the Boston Peninsula and adjoining Cambridge (see Figures 5 and 6). The average altitude of the rock surface in the altered zones is about -30 m (-100 ft), whereas the altitude of the tunnels ranges from -88 to -116 m (-290 to -380 ft). However, because the tunnels do not pass under the highly altered zone of Boston and Cambridge, it cannot be demonstrated that the sparseness of alteration in the tunnels bears on the depth of alteration. In addition, altered rock is abundant at an altitude of -85 (-280 ft) in the North Metropolitan Relief Tunnel.

Engineering Geology. The construction of four bedrock tunnels in the greater Boston area between 1948 and 1960, and under the supervision of the Construction Division of the Metropolitan District Commission (MDC), has greatly added to the geologic knowledge of the area. These four tunnels, totaling slightly more than 32 km (20 mi) in length, are the City Tunnel, the City Tunnel Extension, the Main Drainage Tunnel, and the Malden Tunnel (see Figures 5 and 6). The inner diameter of these tunnels when the concrete lining was emplaced was 3 to 3.7 m (10 to 12 ft).

City Tunnel. The main part of the City Tunnel extends from Riverside, some 8 km (5 mi) northeast of Wellesley, to the Chestnut Hill Reservoir, and is 7,700 m (25,260 ft) long. A branch tunnel 1,043 m (3,422 ft) long extends southeasterly from the east end of the main tunnel to the Chestnut Hill Pumping Station. Seventy-four faults were mapped in the course of explorations.[13] Most of these faults strike northeast and dip steeply to the northwest or southeast. Twenty-nine of the faults have gouge or breccia, or both, ranging in thickness from a smear to 3.4 m (11 ft), but averaging 1 m (3 ft).[8]

No support was needed in the approximately 2,134 m (7,000 ft) of andesites found, nor in the approximately 457 m (1,500 ft) of sandstone exposed in the western part of the City Tunnel.[2,13] Two short sections of sandstone in the Extension required support either because of "excessive splitting parallel to the bedding or to closely spaced joints."[11]

Along 16 faults, the fault-trace separation ranged from several centimeters to 3 m (few in. to 10 ft), averaging 1 m (3 ft); along 6 faults it exceeded 3.7 m (12 ft), the height of the tunnel. Along 52 faults, precise data were not available, but in 36 cases the fault-trace separation was certainly only several centimeters to a meter or less (few in. or ft). Breccia and gouge along 16 of the 52 faults suggest the possibility of greater movement.

Only 16 feet, or 0.0005 percent of the entire tunnel, is supported by structural steel (no bolts were used). The support is just west of a fault and probably indirectly related to it. Seventy-four faults in the City Tunnel did not necessitate support, with one possible exception.[8]

City Tunnel Extension. In the City Tunnel Extension, which is 11,436 m (37,511 ft) long and goes from Chestnut Hill Reservoir to Malden, Billings and Tierney measured 220 faults, exclusive of those along the contacts of dikes or

along bedding planes.[11] These 220 faults are in four rather ill-defined groups:

- a prominent group striking northeast and dipping northwest;
- an equally prominent group striking northwest and dipping northeast;
- a minor group striking northeast and dipping southeast; and,
- a very subordinate group striking northwest and dipping southwest.

The faults range in observed length from 1 m or so (few ft) to a maximum of 82 m (271 ft). The true lengths are always greater than the observed lengths, since the faults have invariably disappeared into the walls or roof of the tunnel.[8] Most of the faults are sharp, tight fractures, at the greatest only several centimeters (few in.) wide. Only a dozen of the faults exceeded a foot in width and the maximum width was 1 m (3 ft). In at least 8 instances several faults were close enough to form fault zones from 1.5 to 59 m (5 to 194 ft) wide. Twenty-one of the faults contained gouge or breccia or both that ranged in thickness from a smear to 1 m (3 ft). Of the 220 faults, 114 were characterized by foliated (slaty) rock several centimeters to 0.3 m (few in. to a foot) thick.[8]

The fault-trace separation was measured on the tunnel walls. Along 84 percent of the faults, the fault-trace separation was only a meter or so (few ft). Along the other 16 percent, it exceeded 3.7 m (12 ft), the height of the tunnel, but was probably only for an extent of several meters (a few tens of ft) at the most. The net slip along the faults was of the same order of magnitude as the fault-trace separation.

Structural steel support with very few bolts was used in 634 linear m (2,103 ft) — 5.6 percent — of the tunnel. Seventy six percent of the support was necessitated by weak shales and argillites, or by badly fractured dikes. Twenty-four percent, equal to 152 m (498 ft) of the support was related to faults. Of this, 29 m (94 ft) was necessitated by faults striking across the tunnel and dipping 25° north-northeast. Associated closely-spaced joints were also a factor. Support in the other 123 m (404 ft) was necessitated by steep shears striking nearly parallel to the tunnel, although in a few cases dikes were a contributing factor. But there were many faults, including some striking parallel to the tunnel, that did not necessitate support. Only 12 of the 220 faults that were recorded were present in areas of support.

Main Drainage Tunnel. The Main Drainage Tunnel, at 1,093 m (3,586 ft) long, extends in a generally easterly direction from near Wentworth Institute in Boston to Deer Island in Boston Harbor. Two short sections of sandstone required support, either because of excessive splitting parallel to the bedding or the joints were too closely spaced.

Rahm mapped 158 faults in the Main Drainage Tunnel.[12] The most abundant had an average strike of N 30° east and dipped 60° northwest. The width ranges from a centimeter to 0.3 m (fraction of an in. to a ft). Gouge was noted along 45 of the faults with thicknesses ranging from a smear to as much as 0.3 m (1 ft). Fault breccia, recorded along 5 of the faults, ranged from a couple of centimeters to 0.3 m (in. to a ft) in thickness. Veins of quartz or calcite, or both, are associated with eight of the faults, generally ranging in thickness from 2 to 9 cm (1 to 6 in), but one is 30 cm (12 in) thick and another is 60 cm (24 in) thick. Rahm states that in the 63 faults for which sufficient data are available, the stratigraphic throw ranges are from 2 cm to 8.5 m (1 in. to 28 ft), with a mean of 0.8 m (2.5 ft) and a median of 1 m (3 ft). The net slip was probably the same order of magnitude as the stratigraphic throw.[12]

Sixty-seven of the faults occur on land between Shaft A at the west end and Shaft B at the edge of the harbor. Seventy-seven percent of this section was supported by structural steel, chiefly because of inherently weak shales and argillites. Forty-six of these faults were in areas supported by structural steel, but this support was related to the lithology, rather than to the faults.[8,12]

Ninety-one of the faults are beneath the harbor — that is, between Shaft B and Shaft C on Deer Island. Only eleven percent of this section was supported by structural steel. This support was associated with diabase dikes or jointed argillite. Some roof bolts were used to stabilize gently dipping strata. None of the faults necessitated support.[12]

In the 915 m (3,000 ft) of conglomerate cut by

the Main Drainage Tunnel,[12] about 12 percent required steel supports, since there was apparently badly altered rock and rock that was faulted and cut by a diabase dike.[2]

Malden Tunnel. The Malden Tunnel, at 1,605 m (5,266 ft) long, extends in a north-south direction through Malden near Malden Square.[10] It crosses the Northern Boundary Fault of the Boston Basin. The Northern Boundary Fault is a large fault that dips northwest, and along which the Lynn Volcanics have been thrust southeasterly over argillite. The stratigraphic throw exceeds 3,000 m (10,000 ft) and the net slip is probably at least 4,570 m (15,000 ft).[10] It is a much larger fault than any of those discussed above. Gouge, breccia, silicification and other features are absent. On the other hand, the argillite for 540 m (1,771 ft) south of the fault and the Lynn Volcanics for 262 m (861 ft) north of the fault were sufficiently jointed to necessitate using structural steel.[10]

Dorchester Tunnel. The Dorchester Tunnel is a high-pressure (200 psi) water supply tunnel that was constructed and operated by the MDC for water delivery to the southern Greater Boston area (see Figure 5). It is the most recent MDC water supply tunnel, having been constructed between 1968 and 1974. The Dorchester Tunnel extends southeastward for about 10.5 km (6.5 mi.) from the Chestnut Hill Reservoir area to Dorchester Lower Mills on the Neponset River estuary. The central construction shaft (Shaft 7C) now serves as an intermediate access point used primarily for water distribution to surface mains. The tunnel is circular in cross-section, and is lined with 0.3 m (1 ft) or more of concrete to provide a 3 m (10 ft) internal diameter. Invert (floor) elevations range from approximately 30 m (100 ft) below sea level at the shaft at Chestnut Hill to 61 m (200 ft) below sea level at the central construction shaft and 64 m (210 ft) below sea level at the shaft at Dorchester Lower Mills. The tunnel was excavated entirely in bedrock. Excavation was by traditional drill-blast methods except for about 1,220 m (4,000 ft) near the main construction shaft, where a mole was used.[17]

The tunnel was mapped by Richardson and travels mostly through the rock of the Boston Basin — mostly through conglomerate, pebbly mudstone and argillite.[15] In addition, the tunnel passes through the Mattapan volcanic rock, principally in the core of the Mattapan anticline and also immediately southeast of the Mount Hope fault.

The central construction shaft was excavated in the argillite of the Roslindale Syncline, and the use of the mole was started in this unit at about 91 m (300 ft) southeast of this shaft. When the mole reached the Mount Hope fault and passed into the Mattapan volcanic rock, it was unable to excavate those rocks satisfactorily, and it was removed from the tunnel. Traditional drill-blast excavation was substituted to complete the tunnel to the shaft at Dorchester Lower Mills.[17] About 884 m (2,900 ft) of volcanic rocks were pierced by the Dorchester Tunnel, of which 46 m (150 ft) required the installation of steel supports due to localized jointing.[15]

The Dorchester Tunnel pierced the entire width of the conglomerate that crops out in Brookline, Jamaica Plain and Roxbury. About 13 percent of the 6,310 m (20,700 ft) of conglomerate traversed by the tunnel required steel support because of soft-rock alteration, which seems to be concentrated along a very wide fault zone.[15,17] Another two percent of supported rock was the result of close jointing, faulting and the effects of a thick diabase dike and interbedded argillite.

Shortly after the tunnel was placed in service in November 1974, the basements of some homes in the area became flooded. Tests made by the owners and the MDC subsequently confirmed that the flooding was the result of water flowing under pressure from the Dorchester Tunnel. Inspection of the tunnel indicated that the lining was significantly cracked, with major water flow occurring between Stations 510 and 523.

MDC engineers and their consultant generally agreed that the internal water pressure in the tunnel was transmitted through the liner and caused compression of the rock.[17,18]

Following investigations, plans and specifications were prepared for pressure grouting to repair the cracked lining. Pressure grouting was performed from within the tunnel. A neat, type III Portland cement grout was used, with a maximum injection pressure of 400 psi. Following the grouting of the primary

stage holes, secondary stage grouting was undertaken at split spacing. Results of water pressure tests, grout take and core borings were used to evaluate the effectiveness of the grouting.[18] After the grouting was completed, a test section of the tunnel lining was instrumented to measure the behavior of the tunnel and adjacent rock during pressure testing. Instrumentation included diametric convergence meters, strain gages, extensometers and piezometers. Over a period of six weeks, the tunnel was water tested under service pressures. In addition to the instrumentation readings, observation well measurements were made to observe the effect of water flow from the tunnel on groundwater levels in the area. Finally, the tunnel was drained, inspected and placed back into service.

Trans-Harbor Highway Tunnels

Two major subaqueous highway tunnels connect downtown Boston with the communities northeast of the harbor, exiting in East Boston (formerly Noddle Island). These tunnels were constructed in two phases. The first tunnel, the Sumner Tunnel, was constructed in the 1920s by conventional shield method, with the muck being removed by hand. To alleviate traffic to Logan Airport in East Boston, a second tunnel, the Callahan Tunnel, was constructed in the 1960s, also using the shield method.

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REFERENCES

1. Kaye, C.A., "Kaolinization of Bedrock of the Boston, Massachusetts, Area," U.S. Geological Survey Professional Paper 575-L, 1967, pp. C165-C172.

2. Kaye, C.A., "Engineering Geologic Framework of the Boston Basin," in Hatheway, A.W., ed., *Engineering Geology in New England*, American Society of Civil Engineers, Annual Meeting, Boston, Preprint 3602, 1979, pp. 1-17.

3. Kaye, C.A., "Bedrock and Quaternary Geology of the Boston Area, Massachusetts," *Geological Society of America, Reviews in Engineering Geology*, Vol. V, 1982, pp. 25-40.

4. Hatheway, A.W., & Paris, W.C., Jr., "Geologic Conditions and Considerations for Underground Construction in Rock, Boston, Massachusetts," in Hatheway, A.W., ed., *Engineering Geology in New England*, American Society of Civil Engineers, Annual Meeting, Boston, Preprint 3602, 1979, 20 pp.

5. Hatheway, A.W., "Hardrock Geologic Mapping for Underground Construction in the Boston Basin," in Farquhar, O.C., ed., *Economic Geology in Massachusetts*, Univ. Massachusetts Graduate School, 1982, pp. 236-246.

6. Aldrich, H.P., Jr., "Back Bay Boston, Part 1," *Journal of the Boston Society of Civil Engineers*, V. 57, N. 1, 1970, pp. 1-33.

7. Aldrich, H.P., Jr., & Lambrechts, J.R., "Back Bay Boston, Part II: Groundwater Levels," Boston Society of Civil Engineers, *Civil Engineering Practice*, V. 1, N. 2, 1986, pp. 31-64.

8. Billings, M.P., "The Significance of Faults in Tunnels," in Farquhar, O.C., ed., *Economic Geology in Massachusetts*, University of Massachusetts Graduate School, 1967, pp. 267-272.

9. Billings, M.P., "Failure of the Concrete Lining in the Dorchester Tunnel (Discussion)," in Farquhar, O.C., ed., *Economic Geology in Massachusetts*, Univ. of Massachusetts Graduate School, 1982, p. 229.

10. Billings, M.P., & Rahm, D.A., "Geology of the Malden Tunnel, Massachusetts," *Journal of the Boston Society of Civil Engineers*, V. 54, 1966, pp. 116-141.

11. Billings, M.P., & Tierney, F.L., "Geology of the City Tunnel Extension, Greater Boston, Massachusetts," *Journal of the Boston Society of Civil Engineers*, V. 57, 1964, pp. 111-154.

12. Rahm, D.A., "Geology of the Main Drainage Tunnel, Boston, Massachusetts," *Journal of the Boston Society of Civil Engineers*, V. 49, 1962, pp. 319-368.

13. Tierney, F.L., Billings, M.P., & Cassidy, M., "Geology of the City Tunnel, Greater Boston, Massachusetts," *Journal of the Boston Society of Civil Engineers*, V. 55, 1968, pp. 60-96.

14. Aldrich, H.P., & Dugan, J.P., "Investigation of a Cracked Pressure Tunnel Lining in Rock," Transportation Research Board, 58th Annual Meeting,

Washington, D.C., 1979, Abstracts.

15. Richardson, S.M., "Geology of the Dorchester Tunnel, Greater Boston, Massachusetts," *Journal of the Boston Society of Civil Engineers Section/ASCE*, V. 63, 1977, pp. 247-269.

16. Cullen, T.R., Young, L.W., Jr., & Keville, F.M., "Tunneling Through the Cambridge Argillite," in Farquhar, O.C., ed., *Economic Geology in Massachusetts*, Univ. of Massachusetts Graduate School, 1982, pp. 197-211.

17. Ashenden, D.D., "Geologic Factors Affecting Failure of the Dorchester Tunnel, Boston," in Farquhar, O.C., ed., *Economic Geology in Massachusetts*, Univ. of Massachusetts Graduate School, 1982, pp. 213-219.

18. Dugan, J.P., Jr., "Pressure Grouting to Repair the Dorchester Water Tunnel, Boston," in Farquhar, O.C., ed., *Economic Geology in Massachusetts*, Univ. of Massachusetts Graduate School, 1982, pp. 221-228.

19. MBTA, *The Boston Rapid Transit System: A History of Progressive Growth*, date unknown.

20. Whitehill, W.M., *Boston: A Topographical History*, 2 ed., Belknap Press of Harvard University, 1973, 249 pp.

21. Kaye, C.A., "Beacon Hill End Moraine, Boston — New Explanation of an Important Urban Feature," in Coates, D.R., ed., *Urban Geomorphology*, Geological Society of America Special Paper 174, 1976, pp. 7-20.

22. Keville, F.M., & Pizzo, C.D., "Community Participation in Public Works Projects," Boston Society of Civil Engineers, *Civil Engineering Practice*, V. 1, N. 1, 1986, pp. 23-42.

Acknowledgements for Boston Geology Articles

The articles included in this issue from page 33 to page 117 are selected from and are based on a larger treatise on the geological and geotechnical conditions under the City of Boston entitled "Geology of the City of Boston, Massachusetts." The editors of the larger work wish to acknowledge William Pitt, of Geotechnical Engineers, Inc., and Henry Russell, of Parsons Brinckerhoff Quade and Douglas, who are additional contributing authors to the work.

The authors wish to thank the New England Section of the Association of Engineering Geologists for their support in preparing the manuscript. They also thank the following individuals for their technical reviews of various sections of the manuscript: A.W. Hatheway, M.H. Peaaase, W.H. Dennen, H.P. Aldrich, R. Hirschfeld, E.L. Krinitzsky, D. Brownlee and J. Engels.

The authors extend their appreciation to the Boston Society of Architects and Harl P. Aldrich for permission to use their figures in the History and Tunnelling articles.

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